UNIVERSIDAD POLITÉCNICA DE MADRID
ESCUELA TÉCNICA SUPERIOR DE INGENIEROS DE
CAMINOS, CANALES Y PUERTOS

ACCEPTABLE LIFE SAFETY RISKS ASSOCIATED WITH THE EFFECTS OF GAS EXPLOSIONS ON REINFORCED CONCRETE STRUCTURES

TESIS DOCTORAL

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Tribunal nombrado por el Magnífico y Excelentísimo Sr. Rector de la Universidad Politécnica de Madrid, el día de de 2017 para juzgar la Tesis Doctoral arriba citada, compuesto de la siguiente manera:

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Los Vocales

Madrid a de de 2017

To my father

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Abstract

Accidental actions on structures may be characterized as low probability - high consequence events. On one hand, their occurrence during the envisaged design working life of the structure is unlikely. On the other, if not appropriately accounted for, the associated effects on structures might entail significant damage. Since moreover such effects are subjected to high uncertainties, decision-making related to structural safety accounting for accidental actions is generally difficult and prone to be based on irrational grounds. Among such actions, gas explosions, are a good example. Despite the continuous modernization of gas installations and appliances, available statistics from different countries show that the occurrence rate of such explosions in buildings does not seem to decrease in a significant way. While the hazard potential is known and recognized, and although dealt with in many design codes, gas explosions are seldom accounted for in the design and evaluation of ordinary building structures. The low occurrence probability evokes reluctance to allocate resources to mitigate the associated risks, which, as a consequence, are often ignored and sometimes consciously accepted. The question if “doing nothing” is a justified practice cannot be easily answered however, since under the implicit approach adopted in everyday practice for verification of structural safety the risks are not quantified nor are acceptable risk levels established.

On this background, the study aims at exploring methods and tools for the practical application of explicit risk analysis in connection with gas explosions in buildings. A procedure is established for quantification of implicitly acceptable structure-related risks to persons, based on the probability of structural collapse and the consequences of such a failure in terms of loss of human life. The procedure adopted is applied to a representative set of building structures with RC members (beams and columns), which is obtained by varying the parameters with the greatest effect on design within reasonable limits. Following their identification, the most relevant hazard scenarios to these members are represented in terms of limit state functions (LSF). Based on the established LSF's, a strict design ($E_d = R_d$) according to a consistent set of codes is carried out, so that structural member performance complies exactly with the safety requirements that reflect *current best practice*. The basic variables involved in the LSF are stochastically characterized, where special attention is paid to the dynamic effects associated with the explosion-induced high loading rates on the members, such as the contribution of inertia forces, energy dissipation and strain rate-sensitive material behaviour.

Quantification of these effects is addressed in a deterministic dynamic analysis where the explosion load is represented as an idealised pressure-time function, compatible with simplified models. Under consideration of dynamic material properties, member flexural response is obtained assuming a single degree of freedom system, whereas the reaction forces, representative for the shear forces, are determined from the dynamic equilibrium formulation applied to the members themselves. For the beams, a comparative study is conducted, where the deployed simplified models are validated by means of non-linear finite element analysis. The analysis of the columns under dynamic bending moment-axial force interaction requires a specific solution algorithm that accounts for the axial force dependent formulation of structural resistance under consideration of both the material- and geometrical non-linearities involved.

In the subsequent reliability analysis of the structural members, the mentioned algorithm is coupled to a purpose-developed FOSM-based iterative procedure in order to obtain the most likely failure point for the established LSF. Taking account of the occurrence probability of a gas explosion event, implicitly acceptable structural failure probabilities for both columns and beams are derived and analyzed in the light of target ceilings demanded by structural codes. The findings suggest significant scope for a more rational formulation of design rules for accidental situations related to gas explosions.

For the estimation of the structural failure consequences, a regression model is developed from previously compiled and statistically evaluated data on explosion-induced structural collapse scenarios in buildings. The model delivers estimations for the number of fatalities as a function of the area affected by structural collapse and the occupancy rate of this area. Reasonable hypothesis are adopted in order to account for the possibility of system collapse given a local member failure.

Subsequently, implicitly acceptable risk profiles are obtained for each of the representative building structures where account is taken of the fact that, in addition to the considered accidental load scenarios, certain member failure modes might be triggered by persistent load arrangements associated with normal building use conditions. Acceptance criteria for structure-related life safety risks are deduced from the findings. Such criteria facilitate the adoption of rational decisions on both, the need and the appropriate choice of risk-reduction measures to counteract the effects of gas explosions in buildings. The design of key elements, upon which depends the stability of the structure, or a large part of it, may be one of these strategies. For this purpose, acceptable risks are translated into target failure probabilities for individual structural members, defined as a function of the potential failure consequences. In spite of their notional character, such target values provide a rational basis for the calibration of the implicit rules in structural codes and standards for verification of structural safety in relation to gas explosions.

Resumen

Las situaciones accidentales sobre estructuras se pueden considerar como eventos poco frecuentes, pero de importantes consecuencias. Por un lado, su probabilidad de ocurrencia durante el periodo de vida útil previsto de la estructura es remota, por otro, los efectos asociados sobre la estructura pueden acarrear importantes daños si no se han tenido en cuenta de forma apropiada. Como además estos efectos están normalmente afectados por un alto grado de incertidumbre, la adopción de decisiones relacionadas con la seguridad estructural en conexión con acciones accidentales es generalmente difícil y propensa a carecer de una base racional. Entre tales acciones, las explosiones de gas son un buen ejemplo. A pesar de la modernización continua de los equipamientos e instalaciones para el suministro de gas en los edificios, estadísticas disponibles de diferentes países muestran que estas explosiones se siguen produciendo con cierta frecuencia. Mientras su potencial de amenaza para la seguridad estructural es objetivamente conocido, la consideración de explosiones de gas en el dimensionado y la evaluación de estructuras de edificación es muy poco frecuente. La baja probabilidad de ocurrencia provoca rechazo a la hora de asignar recursos destinados a la mitigación de los riesgos asociados, que, por consecuencia, a menudo se ignoran o se aceptan conscientemente. Sin embargo, la respuesta a la pregunta de si “no hacer nada” es justificado no es evidente, puesto que, según el enfoque implícito adoptado en la práctica habitual para la verificación de los requisitos de seguridad estructural, los riesgos no se cuantifican ni se establecen los niveles de riesgo aceptables.

Ante este trasfondo, el presente estudio se centra en el desarrollo de unos métodos y herramientas para el análisis explícito de los riesgos asociados con las explosiones de gas en edificios residenciales. Se establece un procedimiento para cuantificar los riesgos implícitamente aceptados para las personas, asociados con las estructuras. Este procedimiento está basado en la determinación de las probabilidades de colapso de los elementos resistentes del sistema analizado y de las pérdidas esperadas de vidas humanas en caso de producirse estos fallos. El procedimiento se aplica a un conjunto representativo de estructuras de edificación con elementos de hormigón armado (vigas y pilares), que se obtiene variando los parámetros característicos para su dimensionado dentro de unos límites razonables. Tras la identificación de los escenarios de riesgo más relevantes para estos elementos, dichos escenarios se representan en términos de las funciones de estado límite (FEL). Basado en estas FEL, los elementos estructurales se dimensionan de forma estricta ($E_d = R_d$) de acuerdo con los requisitos de seguridad estructural que se ajustan a la *mejor práctica actual*. Las variables básicas que intervienen en la FEL se caracterizan estocásticamente con énfasis en los efectos dinámicos asociados con las elevadas velocidades de carga a las que se someten los elementos en una explosión de gas, entre ellos la contribución de fuerzas de inercia, disipación de energía y la alteración de las propiedades de los materiales.

La cuantificación de estos efectos se abarca en el marco de un análisis dinámico determinista donde la carga explosiva se representa a través de una función presión-tiempo idealizada, compatible con modelos dinámicos simplificados. Teniendo en cuenta las propiedades de materiales dinámicas, la respuesta en flexión se analiza mediante un sistema equivalente de un grado de libertad, mientras que las reacciones de apoyo, representativas para los esfuerzos de cortante, se determinan a partir de las ecuaciones

de equilibrio a nivel de elemento. Para las vigas, se lleva a cabo un estudio comparativo que permite validar los modelos simplificados mediante un análisis no lineal por elementos finitos. El análisis de los pilares sometidos a interacción de momentos flectores y esfuerzos axiales dinámicos requiere un algoritmo de solución específico que tiene en cuenta la dependencia de la resistencia estructural del nivel del axil, así como el comportamiento no lineal de los materiales y geométrico.

En el subsiguiente análisis de fiabilidad, el mencionado algoritmo se implementa en un procedimiento de análisis iterativo basado en el método FOSM, desarrollado para la determinación del punto de fallo más probable en la FEL establecida. Teniendo en cuenta la probabilidad de ocurrencia de una explosión de gas, se obtienen las probabilidades de fallo implícitamente aceptables tanto para los pilares como para las vigas. Los resultados sugieren un margen de mejora significativo para una formulación más racional de las reglas de dimensionado en situaciones accidentales caracterizadas por explosiones de gas.

Para la estimación de las consecuencias de fallos estructurales se desarrolla un modelo de regresión a partir de datos previamente recopilados sobre colapsos totales o parciales de edificios inducidos por explosiones. El modelo proporciona el número de víctimas mortales en función del área afectada por el colapso, así como de la ratio de ocupación de dicha área. Se adoptan hipótesis razonables para contemplar la posibilidad de un colapso de sistema a partir de un fallo local de un elemento estructural.

A continuación, se obtienen los perfiles de riesgo implícitamente aceptado para cada una de las estructuras representativas. Se considera la posibilidad de que ciertos modos de fallo puedan desencadenarse tanto como consecuencia de un escenario accidental como también por escenarios persistentes, en condiciones de uso normales de los edificios. A partir de los resultados obtenidos, se deducen unos criterios de aceptación de los riesgos para las personas, asociados con las estructuras de edificación. Estos criterios facilitan la adopción de decisiones racionales sobre la necesidad de adoptar medidas para mitigar los efectos de explosiones sobre estructuras y la elección adecuada de estas medidas. El dimensionado de los denominados elementos clave ("key elements"), de los que depende la estabilidad de una estructura, o de una gran parte de ella, podría ser una de estas estrategias. Para este fin, los riesgos aceptables asociados con las estructuras se traducen en probabilidades de fallo admisibles para elementos individuales, que se definen en función de las potenciales consecuencias del fallo. A pesar de su carácter nominal, estos valores admisibles proporcionan una base racional para la calibración consistente de las reglas implícitas en los códigos estructurales para la verificación de la seguridad estructural en relación con las explosiones de gas.

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Symbols

Latin upper case letters

A	: Area; Building net room area
A_{av}	: Average building net room area
A_{bw}	: Building area affected by the blast wave
A_{col}	: Building area affected by the collapse
$A_{col,j}$	: Building area affected by the collapse associated with hazard scenario j
$A_{col,min}$	: Building area affected by the collapse below which a specific design solution would be acceptable without need for any intervention regarding explosion-related structural safety
A_d	: Design value of an accidental action
A_{EX}	: Tributary area for a member under explosion load exposure
A_{G+Q}	: Tributary area for a member under gravitational load exposure
A_{hu}	: Net room area of a housing-unit
$A_{hu,av}$	: Average net-room area of a housing-unit
A_i	: Average net room area A per person i
A_l	: Loaded area
A_s	: Cross-sectional area of reinforcement; Total cross-sectional area of reinforcement in columns ($A_s = A_{s1} + A_{s2}$)
$A_{s,av}$	: Average net-room area of a building storey
$A_{s,req,acc}$	: Cross-sectional area of reinforcement layer demanded by accidental design situation
$A_{s,req,per}$	: Cross-sectional area of reinforcement layer demanded by persistent design situation
A_{sw}	: Cross-sectional area of shear reinforcement
$A_{sw/s}$	: Cross-sectional area of shear reinforcement per unit length
$(A_{sw/s})_{req,acc}$	: Cross-sectional area of shear reinforcement per unit length demanded by accidental design situation
$(A_{sw/s})_{req,per}$	: Cross-sectional area of shear reinforcement per unit length demanded by persistent design situation
$(A_{sw/s})_{min}$	: Minimum cross-sectional area of shear reinforcement per unit length
A_{s1}	: Cross-sectional area of tensile reinforcement layer
A_{s2}	: Cross-sectional area of compression reinforcement layer
A_v	: Area of venting panels
A_v/V	: Venting coefficient
A_0	: Reference area
B	: Building width B
BLS	: Buckling limit state
B_C	: Failure domain associated with exceedance of BLS due to M-N interaction, characterized by yielding of compression reinforcement
B_T	: Failure domain associated with exceedance of BLS due to M-N interaction, characterized by yielding of tensile reinforcement
C	: Consequence; Constant
CoV	: Coefficient of variation
$CoV_{p_{EX,peak}}$	: Coefficient of variation for explosion peak pressure (population)

$CoV_{p_{EX,peak,s}}$	: Coefficient of variation for explosion peak pressure (sample)
C_b	: Constant accounting for location of collapse-triggering beam
C_c	: Constant accounting for location of collapse-triggering column
C_i	: System component
$C_{ij,k}$	: Consequence of type k associated with event j due to hazard i
D	: Damage (local), Distribution type; Number of disappeared persons
DIF	: Dynamic Increase Factor taking account of strain rate effects
DIF_{Ec}	: Dynamic Increase Factor for concrete modulus
DIF_{fc}	: Dynamic Increase Factor for concrete compressive strength
DIF_{ft}	: Dynamic Increase Factor for reinforcing steel tensile strength
DIF_{fy}	: Dynamic Increase Factor for reinforcing steel yield strength
DIF_{fyws}	: Dynamic Increase Factor for transverse reinforcing steel yield strength
DIF_m	: Dynamic Increase Factor for material property m
$DIF_{\epsilon c1}$	: Dynamic Increase Factor for concrete compressive strain at maximum stress
DLF	: Dynamic Load Factor taking account of inertia forces and energy dissipation
DLF_E	: Dynamic Load Factor associated with action effect E
DLF_M	: Dynamic Load Factor associated with bending moments M
DLF_N	: Dynamic Load Factor associated with axial forces N
DLF_V	: Dynamic Load Factor associated with shear forces V
E	: Action effect; Modulus of elasticity; Energy
$\mathbf{E}$	: Action effect vector (in the bending moment - axial force space)
$\mathbf{E}^*$	: Action effect vector (in the bending moment - axial force space) corresponding to most likely failure point on the limit state surface
EI	: Cross-sectional stiffness
EI_I	: Cross-sectional stiffness in the uncracked state
EI_{II}	: Cross-sectional stiffness in the cracked state
ESS	: Explained sum of squares
EX	: Explosion
$E(N)$	: Expected number of fatalities associated with an undesired event
E_c	: Concrete modulus of elasticity
$E_{c,stat}$	: Concrete modulus of elasticity under static loading conditions
$E_{c,dyn}$	: Concrete modulus of elasticity under dynamic loading conditions
E_d	: Design value of action effect E
E_{dyn}	: Dynamic action effect due to explosion loading (including model uncertainty)
E_{EX}	: Dynamic action effect due to explosion loading (excluding model uncertainty)
$E_{EX,peak}$	: Action effect due to static application of explosion peak pressure $p_{EX,peak}$
E_{ij}	: Event j caused by hazard i
E_k	: Kinetic energy
E_s	: Reinforcing steel modulus of elasticity (under static and dynamic loading conditions)
F	: Frequency; Force; F distribution by Fisher; Cut-off value corresponding to F distribution by Fisher
F^*	: F-statistic corresponding to F distribution by Fisher
$FORM$	: First Order Reliability Method
$F(n)$	: Frequency of occurrence of an event with $N \geq n$ fatalities

$F(n)_{\text{adm}}$	: Admissible frequency of occurrence of an event with $N \geq n$ fatalities
$F(n)_{\text{EX}}$	: Frequency of a collapse scenario with $n \geq N$ fatalities associated with explosion load arrangements
$F(n)_{\text{PER}}$	: Frequency of a collapse scenario with $n \geq N$ fatalities associated with persistent load arrangements
$F(n)_t$	: Target value for frequency $F(n)$
$F(n)_{\text{TOT}}$	: Total frequency of a collapse scenario with $n \geq N$ fatalities associated with all kind of load arrangements potentially affecting the structure in question
$F(n)_{t,\text{PER}}$	: Target value for frequency $F(n)_{\text{PER}}$
$F(n)_{t,\text{TOT}}$	: Target value for total frequency $F(n)_{\text{TOT}}$
$F_X(x)$	: Cumulative distribution function
$F(1)$	: Frequency of occurrence of an event with $N \geq 1$ fatalities
$F(1)_t$	: Target value for frequency $F(1)$
F_c	: Force resisted by concrete in compression
F_{eq}	: Equivalent force (SDOF system)
$F_{\text{eq,peak}}$	: Equivalent peak force (SDOF system)
F_k	: Characteristic value of an action
F_{peak}	: Peak force
F_y	: Yield force of reinforcing steel
F_s	: Force resisted by the reinforcement layer
G	: Gumbel distribution; Permanent action
G_f	: Fracture energy of concrete in tension under static loading conditions
G_k	: Characteristic value of a permanent action
H	: Hazard; Building height
$\mathbf{H}$	: Hat matrix (relates the vector of observations Y_i to the vector of model predictions $\hat{Y}_i$)
H_i	: Hazard i related to a specific system
H_0	: Hypothesis for testing significance of a regression model and its parameters
I	: Resulting inertia force; Second moment of area; Impulse; Non-cracked state; Number of injuries
II	: Cracked state prior to yielding
III	: Post-yield state
I_{col}	: Number of injuries due to structural collapse
I_{sev}	: Number of severe injuries
$I_{\text{sev,col}}$	: Number of severe injuries due to structural collapse
I_{sli}	: Number of slight injuries
$I_{\text{sli,col}}$	: Number of slight injuries due to structural collapse
K	: Reduction coefficient for the resistance of the compression strut taking account of the stress state in the compression chord
K_i	: Transformation factor
K_{IL}	: Inertia-load factor
K_L	: Load factor
K_{LM}	: Load-mass factor
K_M	: Mass factor
L	: Building length L
LN	: Lognormal distribution

Ln	: Natural logarithm
LQI	: Life Quality Index
LSF	: Limit State Function
M	: Bending moment; Safety margin; Ratio between observed and predicted value
$M_{(+)}$	: Positive bending failure scenario of beams
$M_{(-)}$	: Negative bending failure scenario of beams
MDOF	: Multi degree of freedom
MLSC	: Marginal Life Saving Cost
MSE	: Mean square error
MSR	: Regression mean square
M_E	: Applied bending moment (action effect)
M_E^*	: Applied bending moment (action effect) corresponding to FORM design point
M_{Ed}	: Design bending moment (action effect)
$M_{Ed,I}$	: First order contribution to design bending moment (action effect)
$M_{Ed,II}$	: Second order contribution to design bending moment (action effect)
$M_{Ed,acc}$	: Design bending moment (action effect) for accidental situation
$M_{Ed,per}$	: Design bending moment (action effect) for persistent situation
$M_{E,dyn}$	: Applied bending moment due to dynamic actions
$M_{E,dyn,I}$	: Applied first order bending moment due to dynamic actions
$M_{E,dyn,II}$	: Applied second order bending moment due to dynamic actions
$M_{E,stat}$	: Applied bending moment due to static actions
$M_{E,stat,I}$	: Applied first order bending moment due to static actions
$M_{E,stat,II}$	: Applied second order bending moment due to static actions
$M_{E,stat,d}$	: Contribution of static actions to design bending moment M_{Ed}
M_{EX}	: Bending moment at the mid-span cross-section of the beams due to the vertical explosion load $\tilde{p}_{EX}$; First order bending moment at the intermediate cross-section of the columns due to the horizontal explosion load $\tilde{p}_{EX,h}$
$M_{EX,d}$	: Design value for first order bending moment at the intermediate cross-section of the columns due to the horizontal explosion load $\tilde{p}_{EX,h}$
M_{Fc}	: Moment resisted by concrete in compression
M_{Fs}	: Moment resisted by the reinforcement layer
M_G	: Bending moment due to permanent actions
M_{NEX}	: First order bending moment at the intermediate cross-section of the columns due to the eccentrically applied axial force N_{EX}
M_Q	: Bending moment due to variable actions
M_R	: Resisting bending moment
M_R^*	: Resisting bending moment corresponding to FORM design point
$M_{R,b}$	: Resisting bending moment corresponding to the balance point
$M_{R,cr}$	: Cracking moment
M_{Rd}	: Design value for resisting bending moment
$M_{Rd,u}$	: Design value for ultimate moment
$M_{Rd,y}$	: Design value for yield moment
$M_{R,dyn}$	: Resisting bending moment available to sustain dynamic actions
M_{Rm}	: Mean value for resisting bending moment
$M_{R,u}$	: Ultimate moment

$M_{R,y}$	: Yield moment
N	: Number of fatalities; Axial force; Normal distribution
NA	: Numerical analysis
N_{col}	: Number of fatalities due to structural collapse
N_E	: Applied axial force (action effect)
N_E^*	: Applied axial force (action effect) corresponding to FORM design point
N_{Ed}	: Design axial force (action effect)
$N_{Ed,acc}$	: Design axial force (action effect) for accidental situation
$N_{Ed,per}$	: Design axial force (action effect) for persistent situation
$N_{E,dyn}$	: Applied axial force due to dynamic actions
$N_{E,stat}$	: Applied axial force due to static actions
$N_{E,stat,d}$	: Contribution of static actions to design axial force N_{Ed}
N_{EX}	: Axial force due to explosion load
$N_{EX(-),(+)}$	: Accidental load arrangements for columns
N_G	: Axial force due to permanent actions
N_{Gf}	: Axial force due to permanent loads of the building floor systems
N_{G+Q}	: Axial force due to gravitational loads
N_j	: Number of fatalities associated with hazard scenario j
N_{max}	: Maximum number of fatalities associated with an undesired event (e.g. a structural collapse scenario) in a system (e.g. a building structure)
N_{Q1}	: Axial force due to leading variable action
N_{Q2}	: Axial force due to accompanying variable action
N_R	: Resisting axial force
N_R^*	: Resisting axial force corresponding to FORM design point
$N_{R,b}$	: Resisting axial force corresponding to the balance point
N_{Rd}	: Design value for resisting axial force
$N_{Rd,max,acc}$	: Design value for maximum resisting axial force in the accidental situation
$N_{R,dyn}$	: Resisting axial force available to sustain dynamic actions
$N_{R,max}$	: Maximum resisting axial force (corresponding to $e/h = 0$)
$N_{R,u}$	: Resisting axial force corresponding to the ultimate moment M_{Ru}
$N_{R,y}$	: Resisting axial force corresponding to the yield moment M_{Ry}
Ocu	: Total building occupancy
Ocu_{col}	: Occupancy of the area affected by the collapse A_{col}
$Ocu_{hu,av}$	: Average occupancy per housing unit
P	: Probability, Representative value of a prestressing action
PER	: Persistent
PRM	: Predictive residual mean
$P(D H)$	: Conditional probability of (local) damage in case the hazard occurs
$P(E_{ij})$	: Occurrence probability of event j due to a set of hazards i
$P(E_{ij} H_i)$	: Conditional probability of event j due to hazard i given occurrence of hazard
$P(H)$	: Occurrence probability of a hazard
$P(H_i)$	: Occurrence probability of hazard i
$P(S D)$	: Conditional probability of a partial or total system failure in case of (local) damage
P_f	: Accumulated failure probability of a structural member or a structural system

$P_{f,adm}$	: Implicitly admissible, accumulated failure probability
Q	: Variable action
Q_k	: Characteristic value of a variable action
R	: Risk; Societal risk associated with a technical system (e.g. a structure); Resistance
$\mathbf{R}$	: Resistance vector (in the bending moment - axial force space)
R^*	: Resistance corresponding to most likely failure point on the limit state surface (FORM design point)
$\mathbf{R}^*$	: Resistance vector (in the bending moment - axial force space) corresponding to most likely failure point on the limit state surface
R^2	: Coefficient of determination
$\bar{R}^2$	: Adjusted coefficient of determination
RC	: Reinforced concrete
RF	: Response function
RSS	: Residual sum of squares
$R(\delta)$	: Resistance function
R_{adm}	: Implicitly acceptable risks to persons (in terms of the expected number of fatalities) associated with a particular structure
$R_{adm,m}$	: Mean value of implicitly acceptable risks to persons associated with a particular structure
$R_{adm,5\%}$	: 5%- fractile value of implicitly acceptable risks to persons associated with a particular structure
$R_{adm,95\%}$	: 95% - fractile value of implicitly acceptable risks to persons associated with a particular structure
R_d	: Design value of resistance R
$R_{d,req}$	: Required design resistance
$R_{d,req,acc}$	: Required design resistance for accidental situations
$R_{d,req,per}$	: Required design resistance for persistent situations
R_{dyn}	: Effective resistance to sustain dynamic actions
$R_{dyn,eq}$	: Equivalent effective resistance to sustain dynamic actions
$R_{dyn}(\delta)$	: Effective resistance function to the applied dynamic loading
R_{eq}	: Equivalent resistance (SDOF system)
$R_{eq,y}$	: Equivalent yield resistance (SDOF system)
R_j	: Societal risk to persons associated with hazard scenario j
R_k	: Risk associated with consequence component k
$R_{k,adm}$	: Admissible risk associated with consequence component k
R_m	: Mean resistance
R_{prov}	: Provided resistance
R_{req}	: Required resistance
R_{stat}	: Resistance corresponding to static loads
R_u	: Ultimate resistance
$R_{u,dyn}$	: Effective ultimate resistance to sustain dynamic actions
R_y	: Yield resistance
$R_{y,dyn}$	: Effective yield resistance to sustain dynamic actions
S	: System failure
SA	: Simplified analysis

SDOF	: Single degree of freedom
SEE	: Standard error of estimate
SLS	: Serviceability limit state
SM	: Safety Margin in the M-N space
T	: Natural period
TSS	: Total sum of squares
T_{ref}	: Reference period
$U_1 - U_5$	: Failure domains associated with exceedance of UMLS due to M-N interaction
ULS	: Ultimate limit state
UMLS	: Ultimate moment limit state
V	: Volume of enclosure or compartment where explosion occurs; Reaction force; Shear force
V_c	: Shear failure scenario characterized by crushing of the compression strut
V_{cu}	: Contribution of concrete in compression to shear resistance $V_{R,s}$
V_E	: Applied shear force (action effect)
V_{Ed}	: Design shear force (action effect)
$V_{Ed,acc}$	: Design shear force (action effect) for accidental situation
$V_{Ed,per}$	: Design shear force (action effect) for persistent situation
$V_{E,dyn}$	: Applied shear force due to dynamic actions
$V_{E,stat}$	: Applied shear force due to static actions
V_{EX}	: Shear force due to explosion loading
V_G	: Shear force due to permanent actions
V_Q	: Shear force due to variable actions
V_R	: Resisting shear force
V_R^*	: Shear resistance corresponding to FORM design point
$V_{R,c}$	: Shear resistance limited by failure of the compression strut
$V_{R,c}^*$	: Shear resistance limited by failure of the compression strut corresponding to FORM design point
V_{Rd}	: Design value for resisting shear force
$V_{Rd,c}$	: Design value for resisting shear force limited by crushing of the compression strut
$V_{Rd,s}$	: Design value for resisting shear force limited by failure of the tension tie
$V_{R,dyn}$	: Resisting shear force available to sustain dynamic actions
V_{Rm}	: Mean value for resisting shear force
$V_{Rm,c}$	: Mean value for resisting shear force limited by crushing of the compression strut
$V_{Rm,s}$	: Mean value for resisting shear force limited by failure of the tension tie
$V_{R,s}$	: Shear resistance limited by failure of the tension tie
$V_{R,s}^*$	: Shear resistance limited by failure of the tension tie corresponding to FORM design point
V_s	: Shear failure scenario characterized by failure of the tension tie
V_{su}	: Contribution of the shear reinforcement to shear resistance $V_{R,s}$
$V_{su,d}$	: Contribution of the shear reinforcement to the design shear resistance $V_{Rd,s}$
W	: Section modulus
W_s	: Strain energy
$W_{s,BL}$	: Strain energy dissipated under assumption of a bilinear resistance function
$W_{s,M}$	: Strain energy dissipated until bending failure

$W_{s,ML}$	: Strain energy dissipated under assumption of a multilinear resistance function
$W_{s,Vs}$	: Strain energy dissipated until shear failure due to failure of the tension tie
$\mathbf{X}$	: Vector of basic variables X_i
X_i	: Basic variable; Explanatory variable of a regression model
$\mathbf{X}_R$	: Vector of basic variables X_i involved in structural resistance
Y	: Observed value; Dependant variable of a regression model
$\hat{Y}$	: Predicted value of variable Y
$\bar{Y}$	: Sample mean of variable Y
$\mathbf{Z}$	: Vector of standardized variables Z_i
Z_i	: Standardized basic variable

Latin lower case letters

a_c	: Integration coefficient determining the resulting concrete compressive force F_c under assumption of a parabola-rectangle diagram
a_{cc}	: Coefficient taking account of long-term effects on the concrete compressive strength
a_{nom}	: Nominal value of a geometrical property
b	: Mid-span cross-section width
b_i	: Regression coefficient (represents the sample estimate of the corresponding population parameter β_i)
b_{min}	: Minimum mid-span cross-section width
b_w	: Support cross-section width
$b_{w,req,acc}$	: Support cross-section width demanded by accidental design situation
$b_{w,req,per}$	: Support cross-section width demanded by persistent design situation
c	: Factor which accounts for the dependency between different failure events; Constant depending on the curvature distribution along the member
d	: Effective cross-section depth
d_1	: Concrete cover
e	: Eccentricity; Exponential function
e/h	: Normalized eccentricity (for cross-section depth h)
$(e/h)_d$	: Normalized design eccentricity (for cross-section depth h)
$(e/h)_{d,acc}$	: Normalized design eccentricity (for cross-section depth h) corresponding to the accidental design situation
e_d	: Design eccentricity
e_E	: Applied eccentricity
e_i	: Regression model residual
e_{min}	: Minimum eccentricity
e_R	: Failure point eccentricity
$e_{R,b}$	: Balanced eccentricity (corresponding to balance point)
e_{tot}	: Nominal eccentricity
e_y	: Yield eccentricity
e_I	: First order eccentricity
e_{II}	: Second order eccentricity
f	: Failure; Material strength
$f(t)$	: Load-time function
f_c	: Concrete compressive strength

f_{cd}	: Design concrete compressive strength
$f_{c,dyn}$	: Concrete compressive strength under dynamic loading conditions
f_{ck}	: Characteristic concrete compressive strength
f_{cm}	: Mean concrete compressive strength
$f_{c,stat}$	: Concrete compressive strength under static loading conditions
f_{ct}	: Concrete tensile strength
$f_{ct,dyn}$	: Concrete tensile strength under dynamic loading conditions
$f_{ct,stat}$	: Concrete tensile strength under static loading conditions
f_k	: Characteristic value of a material strength
f_s	: Strength of reinforcement (f_y or f_t)
$f_{s,dyn}$	: Strength of reinforcement under dynamic loading conditions
$f_{s,stat}$	: Strength of reinforcement under static loading conditions
f_t	: Tensile strength of reinforcement
$f_{t,dyn}$	: Tensile strength of reinforcement under dynamic loading conditions
f_{tk}	: Characteristic tensile strength of reinforcement
$f_{t,stat}$	: Tensile strength of reinforcement under static loading conditions
$f_X(x)$	: Probability density function for variable X
$f_X(x)$	: Joint probability density function for vector of basic variables X_i
$f_{XY}(x,y)$	: Joint probability density function for variables X and Y
f_y	: Yield strength of reinforcement
f_{yd}	: Design yield strength of reinforcement
$f_{y,dyn}$	: Yield strength of reinforcement under dynamic loading conditions
f_{yk}	: Characteristic yield strength of reinforcement
$f_{y,stat}$	: Yield strength of reinforcement under static loading conditions
f_{yw}	: Yield strength of shear reinforcement
$f_{yw,dyn}$	: Yield strength of shear reinforcement under dynamic loading conditions
f_{ywk}	: Characteristic yield strength of shear reinforcement
$f_{yw,stat}$	: Yield strength of shear reinforcement under static loading conditions
$f_Z(z)$	: Probability density function for variable Z
$f_Z(z)$	: Joint probability density function for vector of standardized variables Z_i
$g(\dots)$	: Function of (...)
$g(\mathbf{X})$	: Structural performance function in terms of basic variables X_i
g_f	: Uniformly distributed permanent loads of floor system, excluding self-weight of the principle beams (force/area)
$g_{f,k}$	: Characteristic value for uniformly distributed permanent loads of floor system, excluding self-weight of the principle beams (force/area)
g_r	: Uniformly distributed permanent loads of roof system, excluding self-weight of the principle beams (force/area)
$g_{r,k}$	: Characteristic value for uniformly distributed permanent loads of roof system, excluding self-weight of the principle beams (force/area)
$\tilde{g}_c$	: Self-weight of the principle beams (force/length)
$\tilde{g}_{c,k}$	: Characteristic value for self-weight of the principle beams (force/length)
$\tilde{g}_f$	: Uniformly distributed permanent loads of floor system, excluding self-weight of the principle beams (force/length)

$\tilde{g}_{f,k}$	: Characteristic value for uniformly distributed permanent loads of floor system, excluding self-weight of the principle beams (force/length)
$\tilde{g}_r$	: Uniformly distributed permanent loads of roof system, excluding self-weight of the principle beams (force/length)
$\tilde{g}_{r,k}$	: Characteristic value for uniformly distributed permanent loads of roof system, excluding self-weight of the principle beams (force/length)
h	: Cross-section depth
h_u	: housing unit
h_c	: Column height
h_{ii}	: Leverage of observation Y_i (i^{th} diagonal element of hat matrix $\mathbf{H}$)
h_s	: Storey height
i	: Number; Person; Distributed inertia forces
j	: Number; Hazard scenario (collapse scenario of a structural member)
k	: Number; Consequence type, Stiffness; Ratio between tensile- and yield strength of reinforcing steel; Member failure mode
k_c	: Coefficient for determining the depth of the resulting concrete compressive force F_c under assumption of a parabola-rectangle diagram
k_{dyn}	: Effective stiffness (during response to dynamic actions)
$k_{\text{dyn,eq}}$	: Equivalent effective stiffness (during response to dynamic actions)
k_{eq}	: Equivalent stiffness (SDOF system)
$k_{M,\text{dyn}}$	: Effective stiffness (during response to dynamic actions) in terms of moment/displacement
l	: Beam span (= distance between frames in longitudinal sense of the building); Member load arrangement
l_{EX}	: Tributary width for a member under explosion load exposure
l_{G+Q}	: Tributary width for a member under gravitational load exposure
l_{ij}	: Lethality rate for person i due to hazard scenario j
l_0	: Effective length of a column (buckling length)
m	: Mass; Material property; Slope
$\bar{m}$	: Mass per unit length
m_i	: Ratio between observed value Y_i and model prediction $\hat{Y}_i$
m_{dyn}	: Material property under dynamic loading conditions
$\mathbf{m}_{\text{dyn}}$	: Set of material properties under dynamic loading conditions
m_{eq}	: Equivalent mass (SDOF system)
m_R	: Slope of the plastic response stage of the moment curvature function
m_{stat}	: Material property under static loading conditions
$\mathbf{m}_{\text{stat}}$	: Set of material under static loading conditions
m_{II}	: Slope (at $\chi = \chi_y$) of step-function $M_E(\chi)$ representing action effect M_E as function of χ
n	: Number of fatalities
n_{hu}	: Number of housing units
$n_{hu,bw}$	: Number of housing units affected by the blast wave
$n_{hu,col}$	: Number of housing units affected by the collapse
n_i	: Number of risk-exposed persons
n_j	: Number of hazard scenarios
n_k	: Number of failure modes

n_l	: Number of load arrangements
n_m	: Number of structural members
n_s	: Number of storeys (excluding ground floor and roof)
$n_{s,bw}$	: Number of storeys affected by the blast wave
$n_{s,col}$	: Number of storeys affected by the collapse
$n_{s,f}$	: Number of storeys where member failure occurs
p	: Pressure; Probability
$p(EX)$	: Occurrence probability of a gas explosion in a gas-supplied housing-unit during reference period T_{ref}
p_d	: Nominal explosion pressure
$p_{d f}$	: Conditional probability of death of a person in the event of failure
p_e	: Member exposure probability
p_{EX}	: Explosion pressure
$p_{EX,peak}$	: Explosion peak pressure
$p_{EX(-),(+)}$	: Accidental load arrangements for beams
p_f	: Probability of failure
$p_{f,adm}$	: Implicitly admissible failure probability
$p_{f,adm EX}$	: Implicitly admissible conditional probability of member failure provided that a gas explosion has occurred and the associated effects influence the member loading conditions.
$p_{f,adm,EX}$	: Implicitly admissible failure probability due to an explosion load arrangement
$p_{f,adm,PER}$	: Implicitly admissible failure probability due to a persistent load arrangement
$p_{f EX}$	: Conditional probability of failure provided that a gas explosion has occurred and the associated effects influence the member loading conditions
$p_{f,EX}$	: Probability for a structural member to fail (in a certain failure mode) under an explosion load arrangement
$p_{f,kl}$	: Probability for a structural member to fail in mode k under load arrangement l
$p_{f,kl EX}$	: Conditional probability of member failure in mode k provided a gas explosion occurred and the associated effects influence the member loading conditions l
$p_{f,PER}$	: Probability for a structural member to fail (in a certain failure mode) under a persistent load arrangement
$p_{ft,code}$	: Nominal target probability of failure according to current design codes
$p_{ft EX}$	: Target value for conditional probability of failure $p_{f EX}$
$p_{ft,EX}$	: Target value for failure probability $p_{f,EX}$
$p_{ft EX,code}$	: Nominal target value for conditional probability of failure $p_{f EX}$ according to current design codes
$p_{ft EX,IR}$	: Individual risk-based target value for conditional probability of failure $p_{f EX}$
$p_{ft EX,max}$	: Maximum target value for conditional probability of failure $p_{f EX}$
$p_{ft EX,min}$	: Minimum target value for conditional probability of failure $p_{f EX}$
$p_{ft EX,SR}$	: Societal risk-based target value for conditional probability of failure $p_{f EX}$
$p_{ft EX,SR,max}$	: Maximum societal risk-based target value for conditional probability of failure $p_{f EX}$
$p_{ft EX,SR,min}$	: Minimum societal risk-based target value for conditional probability of failure $p_{f EX}$
$p_{ft,IR}$	: Target probability of failure based on individual risk
$p_{f,TOT}$	: Total probability for a structural member to fail (in a certain failure mode) under all kind of load arrangements potentially affecting the member in question
$p_{ft,PER}$	: Target value for failure probability $p_{f,PER}$

$p_{ft,SR}$	: Target probability of failure based on societal risk
$p_{ft,TOT}$	: Target value for total failure probability $p_{f,TOT}$
p_j	: Occurrence probability of hazard scenario j
p_{key}	: Nominal pressure for design of key elements against unidentified accidental actions
$p_{N f}$	: Conditional probability of $N \geq n$ fatalities in the event of failure
p_{stat}	: Static failure pressure of venting panels
$\tilde{p}$	: Uniformly distributed load (force/length)
$\tilde{p}_d$	: Nominal explosion load (force/length)
$\tilde{p}_{EX}$	: Uniformly distributed explosion load (force/length)
$\tilde{p}_{EX,h}$	: Uniformly distributed explosion load in horizontal direction (force/length)
$\tilde{p}_{EX,peak}$	: Uniformly distributed explosion peak load (force/length)
$\tilde{p}_N$	: Equivalent, uniformly distributed explosion load on a column taking account of eccentric axial force (force/length)
q_i	: Uniformly distributed imposed loads on building floors (force/area)
$q_{i,k}$	: Characteristic value for uniformly distributed imposed loads on building floors (force/area)
q_s	: Uniformly distributed snow load on roof (force/area)
$q_{s,k}$	: Characteristic value for uniformly distributed snow load on roof (force/area)
$\tilde{q}_i$	: Uniformly distributed imposed load on building floors (force/length)
$\tilde{q}_{i,k}$	: Characteristic value for uniformly distributed imposed load on building floors (force/length)
$\tilde{q}_s$	: Uniformly distributed snow load on roof (force/length)
$\tilde{q}_{s,k}$	: Characteristic value for uniformly distributed snow load on roof (force/length)
s	: Number; Distance between stirrups along the longitudinal member axis; building storey
s_{b_i}	: Standard deviation of regression coefficient b_i
r_i	: Individual risk to person i
$r_{i,adm}$	: Admissible individual risk
$r_{i,EX}$	: Individual risk to person i due to collapse scenarios associated with explosion load arrangements
r_{ij}	: Individual risk to person i associated with hazard scenario j
$r_{i,PER}$	: Individual risk to person i due to collapse scenarios associated with persistent load arrangements
$r_{i,TOT}$	: Total individual risk to person i due to collapse scenarios associated with all kind of load arrangements potentially affecting the structure in question
$r_{it,PER}$	: Target value for individual risk $r_{i,PER}$
$r_{it,TOT}$	: Target value for total individual risk $r_{i,TOT}$
r_Q	: Coefficient of correlation associated with a quantile plot
$r_{Q,\alpha}$	: Critical value for coefficient of correlation associated with a quantile plot
t	: Time; t distribution by Student; Cut-off value corresponding to t distribution by Student
t_{b_i}	: t -statistic corresponding to t distribution by Student
t_i	: Regression model studentized residual
t_{cr}	: Time at cracking of concrete
t_{dec}	: Time at decompression of cross-section
t_{max}	: Time at maximum response
t_p	: Explosion pressure pulse duration

t_r	: Explosion pressure rise time
t_u	: Time at ultimate deflection
t_v	: Time at maximum shear force
t_y	: Time at yield
t_σ	: Time required to attain a specific stress state σ
x	: Neutral axis depth, coordinate of the longitudinal axis of a beam
$\mathbf{x}$	: Specific realization of vector $\mathbf{X}$
$\mathbf{x}^*$	: Design point vector
$\mathbf{x}^{*T}$	: Transposed design point vector
x_i	: Specific realization of basic variable X_i
x_i^*	: Design point coordinates
x_{id}	: Code-based design value
$x_{pEX,peak,p}$	: p-fractile value of explosion peak pressure
$x_{pEX,peak,p,est}$	: Estimated p-fractile value of explosion peak pressure
$\mathbf{x}_R$	: Specific realization of vector $\mathbf{X}_R$
y	: Year
$\mathbf{z}$	: Specific realization of vector $\mathbf{Z}$
$\mathbf{z}^*$	: Design point vector in standardized normal space
z_i	: Specific realization of standardized basic variable Z_i
z_i^*	: Design point coordinates in standardized normal space

Greek upper case letters

ΔM^*	: Difference between bending moments M_E^* and M_R^* corresponding to FORM design point
ΔM_E	: Increment of bending moment M_E
ΔM_{EX}	: Increment first order bending moment at the intermediate cross-section of the columns due to the horizontal explosion load $\tilde{p}_{EX,h}$
$\Delta M_{E,I}$	: Increment of first order bending moment $M_{E,I}$
$\Delta M_{E,II}$	: Increment of second order bending moment $M_{E,II}$
ΔM_{NEX}	: Increment first order bending moment at the intermediate cross-section of the columns due to the eccentrically applied axial force N_{EX}
ΔN^*	: Difference between axial forces N_E^* and N_R^* corresponding to FORM design point
ΔN_E	: Increment of axial force N_E
Δa	: Deviation of a geometrical property a from its nominal value
Δh	: Increment of cross-section depth h
Δs	: Load step size
Δt	: Time step size
Φ	: Standard normal distribution function
$\Pi_{d,req}$	: Required value for design parameter
Ψ	: Ratio between the design values for the ultimate and the yield moment

Greek lower case letters

α	: Risk aversion exponent in F-n criterion; Angle between the shear reinforcement and the longitudinal member axis; Constant model parameter in a regression model
α_A	: Reduction factor for imposed loads on beams
α_i	: Sensitivity factor
α_n	: Reduction factor for imposed loads on columns
β	: Reliability index
β_c	: Coefficient taking account of shear crack inclination
β_i	: Model parameter of a regression model
β_{req}	: Required reliability index
$\beta_{req,EX}$	: Required reliability index associated with explosion scenario
$\beta_{req,PER}$	: Required reliability index associated with persistent scenario
β_t	: Target reliability index
$\beta_{t,code}$	: Nominal target reliability index according to current design codes
χ	: Curvature
χ_{cc}	: Curvature due to concrete creep
χ_{cr}	: Cracking curvature
χ_{stat}	: Curvature due to static actions
χ_{tot}	: Nominal curvature
χ_u	: Ultimate curvature
χ_y	: Yield curvature
δ	: Displacement; Deflection; Resulting deflection at mid-span of the columns
$\dot{\delta}$	: Velocity
$\ddot{\delta}$	: Acceleration
δ_{dyn}	: Displacement due to dynamic actions
δ_{max}	: Maximum displacement; Maximum deflection
$\delta_{max,dyn}$	: Maximum displacement (deflection) due to dynamic actions
δ_{stat}	: Displacement of the intermediate cross-section of the columns due to static actions
δ_u	: Ultimate displacement; Ultimate deflection
$\delta_{u,dyn}$	: Ultimate displacement due to dynamic actions
δ_y	: Yield displacement; Yield deflection
$\delta_{y,dyn}$	: Yield displacement due to dynamic actions
ε	: Strain
ε_c	: Concrete compressive strain
$\varepsilon_{c,stat}$	: Concrete compressive strain due to static actions
ε_{ct}	: Strain in the most loaded concrete cross-section fiber in tension
$\varepsilon_{ct,stat}$	: Strain in the most loaded concrete cross-section fiber in tension due to static actions
ε_{cu}	: Ultimate concrete compressive strain
$\varepsilon_{cu,dyn}$	: Ultimate concrete compressive strain under dynamic loading conditions
$\varepsilon_{cu,stat}$	: Ultimate concrete compressive strain under static loading conditions
ε_{cv}	: Strain in the concrete compression strut at time of maximum shear ($t=t_v$)
$\varepsilon_{cv,stat}$	: Strain in the concrete compression strut due to static actions

$\varepsilon_{c,y}$	: Concrete compressive strain at yield of the longitudinal reinforcement ($t=t_y$)
ε_{c1}	: Concrete compressive strain at maximum stress
$\varepsilon_{c1,dyn}$	: Concrete compressive strain at maximum stress under dynamic loading conditions
$\varepsilon_{c1,stat}$	: Concrete compressive strain at maximum stress under static loading conditions
ε_s	: Strain of reinforcement
ε_{sv}	: Strain in the transverse reinforcement close to the member support cross-section at time of maximum shear ($t=t_v$)
$\varepsilon_{sv,stat}$	: Strain in the transverse reinforcement close to the member support cross-section due to static actions
ε_{s1}	: Strain of tensile reinforcement layer
ε_{s1}^*	: Strain of tensile reinforcement layer corresponding to FOSM design point
$\varepsilon_{s1,stat}$	: Strain of tensile reinforcement layer due to static actions
$\varepsilon_{s1,y}$	: Strain of tensile reinforcement layer at yield ($t=t_y$)
ε_{s2}	: Strain of compression reinforcement layer
ε_{s2}^*	: Strain of compression reinforcement layer corresponding to FOSM design point
ε_u	: Strain of reinforcement at maximum load
$\varepsilon_{u,dyn}$	: Strain of reinforcement at maximum load under dynamic loading conditions
ε_{uk}	: Characteristic value for strain of reinforcement at maximum load
$\varepsilon_{u,stat}$	: Strain of reinforcement at maximum load under static loading conditions
ε_y	: Yield strain of reinforcement
$\varepsilon_{y,dyn}$	: Yield strain of reinforcement under dynamic loading conditions
ε_{yk}	: Characteristic value for yield strain of reinforcement
$\varepsilon_{y,stat}$	: Yield strain of reinforcement under static loading conditions
$\varepsilon_{\sigma,dyn}$	: Strain corresponding to a specific stress-state σ under dynamic loading conditions
$\dot{\varepsilon}$	: Strain rate
$\dot{\varepsilon}_c$	: Strain rate of concrete in compression
$\dot{\varepsilon}_{c,av}$	: Average strain rate of concrete in compression
$\dot{\varepsilon}_{ct,av}$	: Average strain rate of concrete in tension
$\dot{\varepsilon}_{cv,av}$	: Average strain rate in the concrete compression strut
$\dot{\varepsilon}_{c0}$	: Quasi-static strain rate of concrete in compression
$\dot{\varepsilon}_{m,av}$	: Average strain rate
$\dot{\varepsilon}_{m,av}$	: Set of average strain rates
$\dot{\varepsilon}_s$	: Strain rate of reinforcing steel
$\dot{\varepsilon}_{s,av}$	: Average strain rate of reinforcing steel
$\dot{\varepsilon}_{s0}$	: Quasi-static strain rate of reinforcing steel
ϵ	: Error or disturbance term in a regression model
φ_{MII}	: Ratio between the dynamic, second order moment $M_{E,dyn,II}$ and the total dynamic bending moment $M_{E,dyn}$
$\varphi_{MII,d}$	: Ratio between the second order design moment $M_{Ed,II}$ and the total design bending moment M_{Ed}
$\phi(x)$	: Ordinate of the shape function of a structural member at distance x
γ	: Confidence level
γ_c	: Partial safety factor for concrete

γ_{Ed}	: Partial safety factor covering uncertainty in the action and/or action effect model
γ_F	: Partial safety factor covering uncertainty in the action and/or action effect model and in the corresponding model
γ_f	: Partial safety factor for actions or action effects
γ_M	: Partial safety factor covering uncertainty in the material properties resistance and in the resistance model
γ_m	: Partial safety factor for material properties
γ_{Rd}	: Partial safety factor associated covering uncertainty in the resistance model
γ_s	: Partial safety factor for reinforcing steel
η	: Policy factor
η_c	: Reduction factor for concrete cracked in shear
λ	: Slenderness ratio; Probability of an explosion event per unit of time
μ	: Mean value
$\mu_{f_{m,dyn}}$	: Mean value of dynamic material strength
$\mu_{f_{m,stat}}$	: Mean value of static material strength
$\mu_{m_{dyn}}$	: Mean value of dynamic material property
$\mu_{m_{stat}}$	: Mean value of static material property
$\mu_{p_{EX,peak}}$	: Mean value of peak pressure (population)
$\mu_{p_{EX,peak,s}}$	: Mean value of peak pressure (sample)
μ_X	: Mean value of variable X
μ_δ	: Displacement ductility ratio
$\mu_{\delta,dyn}$	: Effective displacement ductility ratio during dynamic response
$\mu_{\delta,dyn,eq}$	: Equivalent effective displacement ductility ratio during dynamic response
μ_χ	: Curvature ductility ratio
$\mu_{\xi_{E,dyn}}$	: Mean value of model uncertainty coefficient for determining action effect E due to dynamic actions
v_{acc}	: Ratio between the nominal explosion load $\tilde{p}_d$ and the total nominal load supported by the beams
θ	: Angle between the concrete compression strut and the longitudinal member axis
ρ	: Correlation coefficient, reinforcement ratio
ρ_c	: Material density (reinforced concrete)
ρ_l	: Longitudinal reinforcement ratio
ρ_{XY}	: Correlation coefficient between variables X and Y
ρ_w	: Shear reinforcement ratio
σ	: Standard deviation; Stress
σ^2	: Variance
σ_c	: Axial stress state in the concrete cross-section ($\sigma_c > 0$ for compression) taking account of the force resisted by the longitudinal reinforcement
$\sigma_{p_{EX,peak}}$	: Standard deviation of peak pressure (population)
$\sigma_{p_{EX,peak,s}}$	: Standard deviation of peak pressure (sample)
σ_X	: Standard deviation of variable X
σ_X^2	: Variance of variable X
$\sigma_{X,Y}$	: Covariance between variables X and Y

$\sigma_{\xi_{E,dyn}}$	: Standard deviation of model uncertainty coefficient for determining action effect E due to dynamic actions
ξ	: Model uncertainty coefficient
$\xi_{E,dyn}$	: Model uncertainty coefficient for determining action effect E due to dynamic actions
$\xi_{M_{E,dyn}}$	: Model uncertainty coefficient for determining bending moments due to dynamic actions
$\xi_{M_{E,stat}}$	: Model uncertainty coefficient for determining bending moments due to static actions
ξ_{M_R}	: Model uncertainty coefficient for determining resisting bending moments
$\xi_{N_{E,dyn}}$	: Model uncertainty coefficient for determining axial forces due to dynamic actions
$\xi_{N_{E,stat}}$	: Model uncertainty coefficient for determining axial forces due to static actions
ξ_{N_R}	: Model uncertainty coefficient for determining resisting axial forces
$\xi_{V_{E,dyn}}$	: Model uncertainty coefficient for determining shear forces due to dynamic actions
$\xi_{V_{E,stat}}$	: Model uncertainty coefficient for determining shear forces due to static actions
$\xi_{V_{Rc}}$	: Model uncertainty coefficient for determining the shear resistance limited by crushing of the compression struts
$\xi_{V_{Rs}}$	: Model uncertainty coefficient for determining the shear resistance limited by yielding of the shear reinforcement
ψ_0	: Factor for combination value of a variable action
ψ_1	: Factor for frequent value of a variable action
ψ_2	: Factor for quasi-permanent value of a variable action
ω	: Tensile reinforcement index

Chapter 1 Introduction

1.1 Background

1.1.1 Risks and their treatment in structural engineering

In everyday life risk is a common term used interchangeably with words like chance, likelihood and probability to indicate that there is uncertainty about the state of the activity, item or issue under discussion [1]. In a technical context, and particular when referring to the field of decision making problems in engineering, risk is to be understood as a mathematical expectation of the consequences of an undesired event [2], e.g. a structural failure. Considering a single event, the risk (R) is expressed by (1.1) as the probability (P) that this event will occur multiplied with the possible consequences (C) it might entail. Since probabilities are dimensionless quantities, the risk (R) is given in the units of the consequences (C), e.g., number of fatalities, monetary units, environmental losses, etc.

$$R = P \cdot C \tag{1.1}$$

Risk, as given per (1.1), is a measure for the magnitude of a hazard which can be defined as a set of circumstances with the potential for causing undesired events within a given system [2], as for instance an abnormal action or an insufficient resistance. Figure 1.1 adopted from [3], is an attempt to schematically categorize the hazard potential associated with a specific technical system, e.g. a building structure. Naturally, only objectively known hazards, i.e., which have been appeared somewhere before, can be subjectively recognised as a threat for structural safety. Subjectively realised hazards might be taken into account by counteracting, suitable safety measures. Alternatively, they might be consciously accepted, in case the corresponding risk is judged sufficiently small. Along with the hazards stemming from human errors, which might have their origin either in unknown, unrealised or simply ignored hazards, or as well in erroneous safety measures, the consciously accepted risks constitute the so-called residual risks, with which, as Schneider [3] points out, «...we have to live – whether we like it or not...» and which are the cause of failures in daily practice. Schneider's analysis [3] of about 800 structural failures published in [4] reveals that in around 75% of these incidents or accidents some sort of human error was the triggering event, rising to even 85% when considering only those cases where casualties were involved (Figure 1.1). These numbers allude to scope for improvement, especially with regard to efficiency of control and checking mechanisms [5-7]. Moreover, they imply that around 25% of the studied 800 accidents and incidents, and approximately 15% of those where casualties were among the consequences, may be attributed to consciously accepted risks, what merits further attention.

Figure 1.1 Categorization of hazard potential associated with a technical system; adapted from [3]

The consciously accepted risks associated with a structural system are the natural consequence of uncertainties involved in the demands on the system, in its geometrical or material properties or in modeling of both action effects and structural performance. While, as shown before, risk cannot be avoided, it can be managed in the public interest [8]. Risk management concerns the analysis, assessment and decision-making in regard to the risks involved in a certain problem. It includes the joint consideration of all uncertainties prevailing in a particular problem and all possible consequences. In this process, a central role for engineers is to provide a basis for decision-making in regard to the cost efficient safeguarding of personnel, environment and assets [1]. The notion *cost efficient safeguarding* alludes thereby to the fact that each safety measure, or equivalently, risk-reducing measure, has a certain cost and that the available financial resources for this purpose are limited and, consequently, must be aimed at being allocated with the highest possible efficiency.

In daily structural design practice, the treatment of safety is based on an implicit approach where the risks associated with a certain structural solution are not explicitly quantified and the question of their acceptability is judged on the base of prescriptive, codified rules. These rules, mainly based on experience and knowledge gained in the past, are deemed reasonable for the type of structure and problem concerned [2]. They represent the state of best practice and provide a reasonable basis for the design of most type of structures under normal loading, operational and environmental conditions [1]. Nevertheless, the prescriptive character inherent to the implicit treatment of safety entails scope for improvement in certain aspects. For instance, in everyday practice many objectively known hazards remain undetected, as reflected by the significant influence of human errors among the residual risks (Figure 1.1). Moreover, the implicitly (and consciously) accepted risks associated with structures designed to current standards diverge widely for a number of reasons, among them the lack of a fully consistent calibration of the standards themselves [9, 10], what alludes to a certain margin for a more efficient allocation of the available resources for reduction of these risks.

A growing acknowledgement of the consequences of such shortcomings has prompted the inclusion of explicit risk analysis in some of the more recent structural design codes, as for instance the Eurocodes [11]. As the notion *explicit* refers to, in such an analysis the risks associated with a certain structural

system or solution are explicitly quantified and compared with safety targets in terms of maximum acceptable risks [2]. Often, such an approach is referred to as performance-based, since design or evaluation is based on performance-based requirements rather than a closed set of prescriptive rules. In comparison to the implicit approach adopted in daily practice, an explicit analysis and assessment of risks entails a number of significant advantages [12]. For instance, a systematic qualitative risk analysis might contribute to detection of relevant hazards and thereby to prevent structural failures. Moreover, on the grounds of a quantitative risk analysis, suitable acceptance criteria may be derived which provide a rational basis for decision-making in structural engineering [9, 10, 13]. For instance, they may constitute the basis for a consistent calibration of the semi-probabilistic models used in the current standards. Other applications where explicit risk analysis provides significant advantages are the assessment of existing structures [14] or the forensic analysis of failed or collapsed structures [15], where site-specific data obtained from the structure may be included in an adequate manner in the evaluation procedure. Moreover, such methods may contribute to the justification of innovative technologies and the optimization of certain solutions, which are substantial advantages of performance-based design codes.

1.1.2 Risks associated with accidental design situations

Another field where explicit analysis and assessment of structural risks delivers significant benefits concerns the accidental design situations which are governed by so-called accidental or abnormal actions, e.g. due to fire, explosion or impact. Accidental actions, which may be considered as the most common causes of structural failure [16], may be characterised as low probability - high consequence events [8]. The meaning of this is that it is unlikely that they will occur during the envisaged design working-life of the structure, but that in case they do, they might entail considerable load effects, which, if not appropriately accounted for, might lead to significant consequences.

A number of structural design codes, among them Eurocode EN 1991-1-7 [17], provide specific guidance concerning design for accidental actions. Given the aforementioned, specific nature of such actions, the general design philosophy differs from the usual approach followed for normal actions in persistent design situations. Design for accidental situations according to EN 1991-1-7 is in particular implemented to avoid structural catastrophes [18]. This is reflected by the general performance requirement stated in the Eurocodes, that the *structure shall be designed and executed in a way that it will not be damaged by events such as fire, explosion, impact and the consequences of human errors, to an extent disproportionate to the original cause* [11, 17]. As a result of this principle, local failure, in most cases identified as a member or component failure, might be acceptable in accidental design situations provided that neither the whole structure nor an important part thereof will collapse [18]. Other strategies for risk-control or mitigation when designing for accidental actions according to [17] are:

- Preventing the action from occurring or reducing the probability and/or magnitude of the action to a reasonable level (e.g., by fire alarm or sprinkler systems).
- Protecting the structure against the action (e.g. by traffic bollards).
- Designing the structure to resist the action (or designing key elements, on which the structure would be particularly reliant).
- Applying prescriptive design/detailing rules which provide a minimum level of robustness (e.g., tri-orthogonal tying for resistance to explosions, or minimum level of ductility of structural elements subject to impact).
- Mitigating the consequences of structural failure.

In practice, the appropriate choice of one or several of the aforementioned strategies will depend on the particular circumstances of each individual design problem at hand. Regardless of this, the practical implementation of these strategies might be associated with certain difficulties. For instance, in [18] the limited effect of preventing or protective measures is mentioned for they depend on factors which, over the life span of the structure, might be outside the control of the structural design process. On the other hand, providing specific local resistance to members might not be justified measure from an economical point of view taking account of the low probability of occurrence and the high uncertainties associated with accidental actions. The fact that for purpose of an economical design the nominal values for some of the loads laid down in [17] are kept usually unrealistic small, such as in the case of impact loads due to vehicles [16], may neither be considered a solution on rational grounds, even if in a context of a linear-elastic structural analysis the design might nevertheless be conservative [19]. In that sense it might be more rational to focus on controlling the consequences of local damage as suggested, for instance, by [20]. However, the tolerance of local failure in view of preserving general structural system performance might also be associated with problems regarding practical implementation. For instance, Ellingwood highlights that this concept may not be attractive to certain stakeholders [8], what is especially the case when risk for human life is involved.

The foregoing considerations indeed underline that, despite the significant advances made in the past, a rational design of structures for accidental actions remains a challenging issue [16]. Vrouwenvelder points out that the current European design standard [17] should be seen as a first step only, which offers scope for improvement from both the theoretical and practical point of view [21]. The advantages that an explicit risk analysis and assessment procedure might offer in this context are numerous:

A systematic, qualitative risk analysis might contribute to the detection of objectively known, but difficult to recognize accidental actions (Figure 1.1).

- Out of all potential hazards to a structure, it should be focused on those that lead to an unacceptable increase in risk. To identify those hazards, a quantitative risk analysis based upon rational acceptance criteria constitutes an essential decision tool. Thereby, the aforementioned specific characteristics inherent to accidental actions, such as low occurrence probabilities, on one hand, and high uncertainties and failure consequences, on the other, might be put into perspective and judged in a rational manner, in terms of risk.
- The availability of rational risk acceptance criteria is moreover helpful in view of devising appropriate strategies and measures for risk mitigation, which should be given high priority in design for accidental actions [18]. For instance, the efficiency of preventive measures could be judged within the scope of an explicit risk analysis. The same is true for structural measures, like specific design of certain structural elements to withstand the action. In addition, decisions on the acceptance of a certain amount of damage could be judged in a suitable manner by means of explicit risk analysis, provided rational acceptance criteria are at hand.
- Explicit risk-analysis enables global decision making regarding treatment of technical risks in general, and in structural engineering in particular. In this respect, the quantification of risks associated with accidental actions would enable a direct comparison to risks associated with other design situations, such as the persistent design situations [11]. Moreover, comparison between risks associated with different types of structures (buildings, bridges, etc.) might be established. In summary, the explicit methods provide a base for a unified safety concept and, consequently, for an efficient allocation of resources designated for risk-reduction.

The benefits provided by methods geared to risk based decision making in the field of structural engineering, in general, and related to accidental actions in particular, are generally recognized [1, 12, 16, 22]. In the last years, significant advances have been made with respect to development and application of such methods. However, for the time being the existing regulations establish only a general framework for this purpose. The lack of practical tools and methods, as well as administrative obstacles, still hinders the implementation of explicit approaches in routine practice to date. A key issue in this context is the lack of rational risk acceptance criteria. The Eurocode on accidental actions [17], for instance, identifies the need for defining the accidental actions to be taken into account in design, as well as the corresponding design strategies, as a function of the acceptable level of risk, among others. However, acceptable risk levels are not explicitly established and it is up to authorities, owners or other stakeholders to prescribe these [23].

On this background, the present thesis aims to contribute to the development of practical models and criteria designated for the analysis and acceptance of structural risks associated with accidental actions. In particular, the work focuses on risks due to gas explosions in building structures.

1.1.3 Risks due to gas explosions in building structures

Gas explosions account for a substantial number of accidental actions in buildings [24]. The most famous case is without doubt the accident in an apartment on the 18th floor of the 22-storey Ronan Point building (London) in 1968. A gas explosion blew out an external load-bearing wall what made the upper floor slab fall onto the floor below, initiating a progressive collapse of one corner of the whole block [25], as shown in Figure 1.2 (left) [26]. Five people died and 16 were injured. The incident alerted experts in the field of structural engineering to the problem of progressive collapse [27] and finally let to modification of building codes and regulations. Since then, and despite the continuous modernization of gas installations and appliances, the occurrence rate of gas explosions in buildings across different western countries does not seem to decrease in a significant way. In Spain, for instance, such incidents caused at least 132 fatalities in the last decade [28], many among them attributable to structural collapse. As an example, Figure 1.2 (right) shows the complete collapse of a residential building in Palencia, in 2007, involving 9 fatalities and more than 30 injured [29].

Figure 1.2 Partial collapse of Ronan Point building (London), in 1968 (left) [26] and total building collapse in Palencia (Spain) in 2009 (right) [29].

Gas explosions are dealt with in the Eurocodes [17]. Given their well-known hazard potential for structural safety, they are classified as identified accidental actions, and demanded to be accounted for “in

the design of all parts of the building (or other engineering works) where gas is burnt or regulated...". In some other part of [17] it is specified that for buildings classified as consequence class CC2, such as residential buildings [11], the design for gas explosions should be based on the design of key elements "to resist the action by either using an analysis based upon static equivalent load models or by applying prescriptive design/detailing rules". Specific guidance on practical applications is provided in [30].

In spite of their, at least, apparent importance with regard to structural safety, and despite the specific consideration in the Eurocodes [17], it is felt that in the vast majority of ordinary buildings designed nowadays in Spain, and probably elsewhere, risks associated with gas explosions are not counteracted by any kind of measures. In other words, such risks are often ignored and sometimes consciously accepted (Figure 1.1). Among possible reasons therefore, one might quote the occurrence rate of explosion events, by most engineers probably perceived as very low, and the reluctance to allocate funds to mitigate such kind of low-probability future events [8]. Whatever the reasons are, the question that rises is if "doing nothing" is a justified practice, or if, on the contrary, certain risk reduction measures would be appropriate. A knowledgeable answer to this question cannot be easily given however, since according to the implicit approach in current practice the risks are not quantified nor are the acceptable risk levels established. Indeed, addressing these issues would contribute to a more rational and transparent treatment with regard to decisions on both the need and the appropriate choice of risk-reduction measures to counteract the effects of gas explosions in buildings. These subjects are being addressed in the present thesis. The particular objectives and the scope of the work will be outlined in advance.

1.2 Scope

As stated before, equation (1.1) describes, in general terms the risk associated with a certain undesired event. Generally, any type of structural failure or behaviour might constitute such an event. However, in the context of accidental actions, the most relevant, undesired event is a structural collapse scenario and consequently the study focuses thereon only. Under consideration of an abnormal design situation, such as a gas explosion, the probability (P) for structural collapse can, in general terms, be itemized as expressed per (1.2):

$$P = P(H) \cdot P(D|H) \cdot P(S|D) \quad (1.2)$$

where H represents the hazard, D the local damage and S a failure scenario related to system failure [31]. Referring to the hazard *gas explosion in building structures*, probability P(H) represents the occurrence probability of such an explosion, P(D|H) the conditional probability of a member failure in case the explosion occurs and P(S|D) the conditional probability of a partial or total collapse of the structure, given the local damage. The idealized representation of the collapse probability given by equation (1.2) provides the basis for the current treatment of load combinations for abnormal loads in ASCE Standard 7-10 [32], as reported by [33], and is generally used within the context of a risk-based approach for evaluation of structural robustness [34], [35].

Probability P(H) can be estimated from statistics and is essentially independent of any structural design strategy adopted to increase safety, whereas P(D|H) and P(S|D) fall in the purview of the structural engineer [22]. The probability of member failure given the hazard, P(D|H), can be assessed by means of structural reliability methods for which purpose suitable probabilistic models for both action and resistance variables are required. Finally, probability for system failure P(S|D) depends on the characteristics of the entire structure and demands an analysis in the damaged state where load-carrying mechanisms not normally considered in building design (e.g., membrane action of floors, large deformations, significant non-linear action in the post-peak range) are mobilized [22]. Such an analysis is difficult and

impractical for design, although it might naturally be justified for research purpose, as for instance in [34]. In the present work the explicit analysis of a structural system in the damaged state, and consequently the explicit estimation of the conditional system failure probability $P(S|D)$ is not included. Consequently, the quantification of the effectiveness of prescriptive design or detailing rules, affecting system behaviour rather than member behaviour, such as providing continuity through tie forces, is neither addressed in the study.

Concerning the other main constituent of risk, as defined per (1.1), the consequences (C), it is to note that a gas explosion might entail loss of life, injury or significant monetary losses, among others. In the present study, representation of consequences will be simplified by consideration of fatalities only. This is justified since risks to personal integrity generally prevail in (civil) engineering, for both ethical and legal reasons [9]. Although according to the new international standard about general principles on reliability of structures [36], there is no need for defining acceptable levels of risk to persons, in the opinion of social scientists it is unlikely that the public would accept higher failure rates than those associated with current best practice [13], even if they are based on rational acceptance criteria such as the Marginal Life Saving Cost principle (MLSC), demanded by [36]. Indeed, the aforementioned standard [36] states that “an activity which is found to be acceptable should be assessed in regard to the corresponding absolute level of life safety risk”, and that the practical implementation of the MLSC principle by using the Life Quality Index (LQI) might require the specification of “absolute values of the acceptable life safety risks”. The LQI principle itself is not addressed in the work.

Finally, the scope will be limited with respect to building types and their structural systems. Gas explosions may occur in any type of building where gas is burnt for purpose of energy supply. However, the by far highest numbers of explosions occur in ordinary, residential buildings. In Spain, the vast majority of residential building structures are made of reinforced concrete (RC). The most common type of structural system is shown in Figure 1.3, constituted essentially by beams and columns resisting to both gravitational and wind loads. Consequently, the scope of the study is limited to such type of reinforced concrete structures, designated for residential use. Other construction materials or structural typologies, although they might be generally more vulnerable to explosions, such as non-reinforced masonry [37] or concrete panel structures [38], are not considered.

Figure 1.3 Typical building structure with reinforced concrete members

The present study will be based on the Spanish building codes (CTE) on basis of design and actions while for structural resistance reference will be made to the national standard EHE. To a large extent, these codes are compatible with the respective Eurocodes.

1.3 Objectives and approach

The main *objectives* of the present thesis are the following:

- To deduce and establish models for estimation of both the probabilities and the consequences of a gas explosion-induced collapse in ordinary, residential RC building structures.
- To employ these models in order to determine the implicitly accepted risks to persons associated with such structures.
- To deduce rational acceptance criteria that may be employed for the assessment of explosion related structural risks within the context of an explicit risk analysis, or serve as a basis for a risk-based calibration of semi-probabilistic models used in daily practice.

The notion “implicitly accepted risks” refers thereby to the inherent risk set out in existing structural standards, which represent best practice (section 1.1.1) and which are regarded as intrinsically acceptable. Risk acceptability therefore depends on the degree of reliability implicitly required by such standards. Taking account of the fact that structural design codes are based predominantly on the design of structural members or the consideration of individual member failure modes [39], this degree is directly associated with the implicitly accepted probability of cross-section or member failure. Strictly speaking, this is true for normal loading conditions (persistent design situations), since the design philosophy for accidental actions, as exposed before (section 1.1.2), differs from the traditional design approach for normal actions. Nevertheless, the Eurocode [11] offers, in compliance with the persistent design situations, the possibility to base the verification of structural safety in relation with gas explosions on cross-section or member resistance (section 1.1.2), through application of the accidental design load combination. Although in daily practice member design for explosion loading might not be an economical solution (section 1.1.2), and disregarding the fact that such a design, in contrast to the requirements in persistent design situations, is often neglected in practice (section 1.1.3), the failure probabilities associated with cross-sections or members designed according to the accidental load combination, and by extension the corresponding risks, might indeed be referred to as intrinsically acceptable. Following this approach, the risks associated with gas explosions could be directly compared to the risks corresponding to persistent design situations, which besides of being intrinsically accepted reflect general praxis. As stated before, one of the main advantages of explicit risk analysis and assessment is precisely that they provide a base for a unified, coherent safety concept including all type of situations to which a structure might be exposed.

In light of these considerations, the following *tasks* have to be addressed in order to achieve the above-defined objectives:

The determination of structural failure probabilities requires the former definition of models in order to characterize the uncertainties associated with each of the basic variables involved in the problem at hand. Of special importance in the present context are those variables that refer to both the dynamic loading and resistance parameters. For instance, a probabilistic model for the gas explosion pressure is required. Moreover, dynamic effects on the structural behaviour, such as the contribution of inertia forces, energy dissipation by means of non-linear deformations or the strain rate sensitivity of materials should be accounted for in the reliability calculations, what calls for a previous dynamic analysis. For

purpose of an efficient data processing, and because excessive accuracy is not imperative in the present context, such an analysis should be based on simplified dynamic models.

The estimation of the consequences of gas explosions in buildings demands for a suitable model. Since gas explosions occur with certain frequency (section 1.1.3), and since these, at least when fatalities are involved, are generally reported in press, a reasonable approach to obtain such a model from a statistical evaluation of data on real explosion-induced collapse events. Since the study focusses on risks to persons, the model should deliver estimations for the number of fatalities as a function of some parameter characterizing the magnitude of structural damage induced by the explosion. Since the explicit analysis of damage to a structural system, provided a failure of one of its members, is out of scope (section 1.2), reasonable assumptions should be established regarding this issue.

For the second of the established objectives, the estimation of the implicitly acceptable risks to persons, a procedure has to be defined which, for sake of a subsequent comparison, should be consistent with the approach adopted in former studies [9, 10, 13]. In broad terms, this procedure implies determining implicitly acceptable collapse probabilities of a representative set of structural members, strictly designed according to a consistent set of code rules, so that structural member performance complies exactly with the safety requirements that reflect *current best practice*. In a second step, the consequences of explosion-induced member collapse are to be estimated by means of the previously deduced consequence models. Finally, the implicitly accepted risks to persons associated with a representative set of RC building structures should be determined as a function of both acceptable probabilities of structural failure and the associated consequences.

Subsequently, risk acceptance criteria suited to practical applications should be deduced from the findings. Thereby, risk acceptance from both the individual and the societal perspective should be approached. The results should be related and compared to previous developments associated with normal loading conditions [9, 10, 13]. Finally, the study should close with specific recommendations regarding the acceptability of risks to persons due to gas explosions in building structures.

1.4 Outline of the document

Chapter 1: Introduction

The present chapter offers an introduction to the background of the present thesis. The concept of risk is introduced and their treatment in structural engineering is highlighted, with particular emphasis on accidental actions. It is argued that, in comparison with the implicit treatment of risks in daily practice, an explicit approach offers a variety of significant advantages that are all related to more rational decision making and hence to a more efficient allocation of resources designated for risk-reduction in practice. Specific reference is made to gas explosions in buildings. The need to quantify the associated risks and to deduce rational acceptance criteria for these is identified. On these grounds, the objectives and the scope of the present thesis are defined.

Chapters 2 and 3: State of the art

Given the different thematic areas to be dealt with in the present study, the state of the art will be treated in two different chapters. Chapter 2 addresses the basic concepts of risk and reliability. The main steps of an explicit risk analysis procedure comprising the establishment of risk acceptance criteria are explored. The principles of structural reliability are then presented and related to the treatment of risks in current design standards. Some consideration on the challenging issue of system reliability are made.

The chapter closes with a brief summary of the complex circumstances involved in the reliability analysis of RC elements under moment-axial force interaction, since such calculations are addressed in the study.

In Chapter 3, the occurrence and the physical backgrounds of gas explosions in building structures are explored. The behaviour of RC elements exposed to this type of action is described. Simplified approaches for modeling of this behaviour, employed in the presented study, are presented.

Chapter 4: Approach and basic developments

Chapter 4 addresses key issues regarding the approach to the study and the methodology (mathematical framework and procedure) followed in order to attain its objectives. Moreover, it contains basic developments upon which the following chapters are grounded. Among these are the definition of the representative set of structures and structural members, the establishment of the hazard scenarios to these structures and the translation of these scenarios into limit state functions (LSF). Moreover, based on these LSF, the strict design of the defined structural members is addressed. Finally, models are established for purpose of a stochastic characterization of the basic variables involved in the defined LSF.

Chapter 5: Dynamic Analysis

As stated before, the reliability analysis of explosion-induced member response may not obviate a specific dynamic analysis, what will be addressed in Chapter 5. In particular, the objectives of this analysis are the determination of dynamic load and increase factors, which account for certain dynamic influences on the structural behaviour of the members when exposed to the effects of an explosion. Moreover, second-order effects will be addressed in the analysis. The analysis procedure, based on simplified dynamic models is exposed, followed by a description and discussion of the results. Finite element calculations are carried out for purpose of the validation of the simplified models used.

Chapter 6: Implicit reliability level

Chapter 6 addresses procedural aspects and the results of the FORM analysis carried out to establish the reliability level implicitly required by the employed design codes. A specifically developed reliability procedure for the analysis of column reliability under M-N interaction is described. The results for both beams and columns are then presented and discussed in the light of current target ceilings. Finally, Chapter 6 investigates into the accumulated failure probabilities of structural members, taking account of the fact that, in addition to the accidental load combination, their failure might be triggered by persistent load arrangements.

Chapter 7: Consequence models

The deduction of models to estimate the consequences is outlined in Chapter 7. The procedure followed for data collection from press releases is exposed. Subsequently, the gathered data is statistically evaluated and complemented by means of information from the National Statistics Institute (INE). A multi-linear regression analysis is then performed. Different models are discussed on the background of statistical criteria in order to obtain the best possible solution. Comparisons to results in former studies [9, 10, 13] are provided.

Chapter 8: Implicitly acceptable risks and requirements

Once the probabilities for structural failure (Chapter 6) and the corresponding consequences (Chapter 7) are being established, the implicitly accepted risks are deduced for the representative set of building structures and compared to former studies [9, 10, 13]. The statistically evaluated results will provide the basis for the deduction of acceptance criteria for risks to persons associated with building structures.

Chapter 9: Conclusions

The thesis closes with a summary of the most relevant findings. The main contributions of the study are highlighted and the possible subjects to be addressed in the future identified.

Chapter 2 Risk and reliability

2.1 Principles of risk-informed decision making

2.1.1 Introduction

Engineering facilities such as buildings, bridges or power plants are generally intended to provide benefit to the members of society. A beneficial engineering facility is normally understood as [1]:

- Assuring safety to persons.
- Being economically efficient.
- Minimizing the adverse effects on the environment.

In order to achieve these requirements, the ultimate task of civil engineers is to make decisions, or to provide a rational basis for decision makers (authorities, politicians, etc.), such that the benefits of the engineering facility will be as large as possible. Thereby, the uncertainties affecting all the circumstances or events to which the facility may be subjected must be dealt with in order to enable suitable estimations of the probability for these events to occur. Moreover, the potential consequences of these events must be addressed. As outlined in the introduction of the present thesis (section 1.1), the joint consideration of probabilities and consequences describes the risk (1.1), which provides a suitable measure for decision making in engineering.

In the next section, a generally accepted and frequently applied framework for risk-informed decision analysis will be exposed and its constitutive individual steps described. In the following, section 2.1.3, addresses a crucial point for the assessment of risks - the setting of suitable risk acceptance criteria. In keeping with the scope of the thesis, the most relevant criteria for life safety risks will be outlined, from the perspective of both the individual and the societal risk acceptability.

2.1.2 Explicit risk analysis procedure

A generic representation of the steps comprising a risk-informed decision analysis is shown in Figure 2.1. The procedure is based on the Australian New Zealandic Code 4369 [40] and has found acceptance in many other countries. It is generic in a sense that it principally applies for any type of engineered facility [1].

The procedure is divided into two principal stages - qualitative and quantitative risk analysis. The former primarily intends to identify the hazards to which a system may be subjected and to combine them logically to establish possible hazard scenarios. The latter aims at delivering a quantitative evaluation of those scenarios, in terms of their occurrence likelihood and their potential consequences, and to assess the risks with regard to previously established acceptance criteria. In the following, the individual steps characterizing the risk analysis procedure will be exposed according to the descriptions in [1, 2, 41, 42].

Figure 2.1 Generic representation of a risk-based decision making procedure, adapted from [40].

Definition of context and scope

The definition of the context of a decision problem concerns the identification of the decision makers and all other stakeholders with interests in the decisions to be made (society, groups or individual risk receptors, clients, authorities, etc.). Moreover, it implies to detect and define all the constraints to the decision making process, irrespective of their nature (political, legal, social, financial, cultural, etc). A crucial step of the context definition is the setting of acceptance criteria, which, as Faber [1] points out, can be considered as a decision problem itself.

In addition to the context, the scope of the analysis to be performed should be defined at the beginning of the procedure. This includes the clear definition of the system boundaries and the description of the assumptions regarding the system representation and idealization, what will have significance for the level of detail to be addressed in the analysis.

Identification of hazards and hazard scenarios

The qualitative identification of hazards and hazard scenarios is one of the most important steps within the risk analysis procedure [2]. Once the potential hazards and their possible combinations are recognized, appropriate measures might be adopted to overcome their consequences. On the other hand, any unidentified relevant hazard necessarily introduces a bias in the decisions adopted during the subsequent evaluation, as a result of which misleading conclusions could be drawn.

As introduced before (section 1.1.1), in the context of a risk analysis, a hazard describes a set of circumstances with the potential for causing undesired events within a given system. In general terms, it might be distinguished between natural and man-made hazards [2, 3]. Examples for the former are earthquakes, landslides, hurricanes, snow, etc., whereas the latter refer to influences like fire, explosions, vehicle impacts or similar events. However, for purpose of the discussion of counter measures, such a distinction, which is not even straightforward in all cases, is of less importance. Special attention should be paid however to hazards due to human errors, such as design errors, material flaws, construction errors, etc. [7] for they are the most common cause of failures in practice (section 1.1.1).

Several hazards may concur in space and time, giving rise to what is known as a hazard scenario. Such situations normally generate higher risks than any individual hazard separately, what in the case of structures may be considered the rule rather than the exception [3]. Generally, each scenario can be characterized by a combination of leading and accompanying actions and influences.

A variety of methodologies have been developed for identification of hazards (e.g. FMEA, PHA, HAZOP) and for modeling of relevant scenarios (e.g., fault tree, event tree, decision trees, causal networks) [2]. Detailed descriptions of these methodologies are beyond the scope of this thesis, but may be found in the specific literature [42, 43].

Estimation of probabilities

Probability is, generally speaking, the likelihood or degree of certainty of a particular event occurring during a specified period of time [2]. Different interpretations of the probability concept are described in the literature, including the classical, the frequentistic or the Bayesian approach [1, 3]. In many problems related to mechanical or electronic engineering, probability estimations may be based on frequentistic information, e.g. when the probability of failure of massively produced, in principle identical components, such as light bulbs or valves, is considered, where large amount of data on observed failure rates is available. However, in the field of structural engineering this is, as a rule, not the case, since the conditions characterizing both loading and resistance of a structure are generally unique and, consequently, failure rate information is virtually nonexistent. In such cases the Bayesian interpretation of probability is far more appropriate [1]. The basic idea behind this approach is that lack of knowledge should be treated by probabilistic reasoning. In reality, decisions have to be made despite the lack of knowledge and probabilistic tools are a great help in that process. Modern structural reliability and risk analysis is based on this approach [1].

Assuming that a considered technical system or one of its components may be exposed to mutually exclusive hazard situations H_i , and an undesired scenario E_{ij} (such as a structural failure) given situation H_i occurs with the (conditional) probability $P(E_{ij}|H_i)$, then the probability of occurrence of the undesired event, $P(E_{ij})$, is given by [2]:

$$P(E_{ij}) = \sum_i P(H_i) \cdot P(E_{ij}|H_i) \quad (2.1)$$

The conditional failure probabilities $P(E_{ij}|H_i)$ can be determined by means of reliability methods whereby account can be taken of the uncertainties involved in the problem. This will be addressed in more detail in sections 2.2.1 to 2.2.3. For now it should be mentioned that these probabilities should be understood as nominal (or notional) values, i.e. they do not reflect the true probability of failure for a number of reasons, among them the fact that they do not account for human error [1, 2]. Regardless of their notional character, which precludes any interpretation in absolute terms, failure probabilities obtained by means of reliability methods have been proven to be very useful when interpreted in comparative sense only, for instance as operational values for the purpose of code-calibration and comparison of reliability levels of structures [44].

Estimation of consequences

In general terms, consequences are possible outcomes of an undesired event that may be expressed to define the extent of human fatalities and injuries, environmental damage or economic losses [2]. The consequences to be expected in a certain failure scenario depend on the specific characteristics of both the hazard and the system where this hazard occurs. A distinction may be drawn between direct and

indirect consequences, such as proposed in the JCSS risk assessment framework [45]. Direct consequences are related to damage of individual system constituents, whereas indirect consequences are understood as any consequences beyond the direct consequences. For instance, when a civil engineering structure is referred to, the direct consequences may describe the losses due to local structural damage, e.g. a member failure, whereas the latter, also referred to as follow-up consequences, describe the losses associated with the system failure, for instance a collapse scenario (S) given the local damage (D) (section 1.2).

A systematic procedure to estimate the consequences of undesired events in a technical context is called consequence analysis. The consequence types k to be considered in such an analysis are necessarily the same as those contained in the risk acceptance criteria, specified within the definition of the context of the risk analysis procedure (for example number of fatalities, number of injured or monetary consequences expressed in a certain currency). Following the established nomenclature, $C_{ij,k}$ describes the consequence of type k associated with event j due to hazard i [2].

Estimation of risks

Having performed the analysis of the probabilities of occurrence of an undesired event, such as a structural failure, and its consequences, the corresponding risks may be determined as a function of both measures (1.1). Following the above introduced definitions of probabilities and consequences, the total risk R_k , expressed in terms of consequence component k , can be formulated as the sum over all hazards (H_i) and undesired events (E_{ij}) these hazards might induce:

$$R_k = \sum_{i,j} (C_{ij,k} \cdot P(E_{ij}|H_i)P(H_i)) \quad (2.2)$$

In cases where it is possible to deal with a single consequence type only, subscript k in equation (2.2) may be omitted and hence the analysis will be simplified. Moreover, it should be mentioned that (2.2) does not account for differentiation of direct and indirect consequences.

Risk assessment

Risk assessment refers to a mere comparison of the estimated risks R_k to previously established admissible values, $R_{k,adm}$. Simply speaking, if equation (2.3) is fulfilled, the risks associated with the considered system are considered acceptable and no further actions are required. However, as mentioned before, the mayor difficulty lies in the definition of suitable and rational acceptance criteria, not in the comparison itself. Section 2.1.3 deals with this subject.

$$R_k \leq R_{k,adm} \quad (2.3)$$

Risk treatment

Should the risks exceed the bounds of the specified acceptable values, different options for risk treatment have to be analysed. Possible strategies for risk treatment measures can be of technical or administrative nature, and might be associated with one or more of the following strategies [1]:

- Risk mitigation or avoidance can be achieved by conceptual changes affecting the system and its components. An example would be the use of non-corrosive reinforcement in order to mitigate risk of corrosion damages in concrete structures.

- Risk reduction may be implemented by reduction of the consequences and/or the probability. In practice, risk reduction is normally performed by a physical modification of the considered system or its component, such as providing an adequate capacity.
- Risk transfer may be performed, e.g., by insurance or other financial arrangements where a third party takes over the risk. Therefore, risk transfer is normally associated with a cost.
- Risk acceptance might be an option if the risks do not comply with the risk acceptance criteria and other approaches for risk treatment are not effective. This may e.g. be the case when the costs of risk mitigation, reduction or transfer are higher than the desired risk reduction.

Risk communication

Risk-informed decision making usually affects a high number of stakeholders with different interests regarding the decisions to be made. A fluid and efficient (risk) communication between the different parties involved is therefore crucial. A mayor problem related to risk communication is that these parties usually have different understandings about the concept of risk. The definition of risk in a technical context, as the product of probabilities and consequences (1.1), is unfamiliar to most of the involved stakeholders, and thus the perception of risks by experts and others may be completely different [2]. Hambly points out that the way in which the public perceives the seriousness of risks can be very different from the reality [46], what might be, at least partially, attributable to the way media report on accidents and hazardous activities. An example for this is the collapse of the Ronan-Point building (Figure 1.2), which, involving 5 fatalities made the news during weeks, contributed to the public's perception towards high-rise accommodation [2] and finally entailed a modification of certain building codes. On the other hand, an everyday car accident with up to 5 fatalities, is in the most cases not even newsworthy and the corresponding life safety risk is generally assumed by the public although it is definitely higher than the risk associated with a building collapse.

Monitoring and review

Risk analysis is a living process involving a constant feedback of information from the considered system to the analysis procedure. Therefore, monitoring and other data acquisition during construction or use of the system is useful. Whenever new information is obtained, the risk analysis may be updated and used for optimizing the system performance in regard to the specified acceptance criteria [1].

2.1.3 Acceptance criteria

The establishment of rational risk acceptance criteria is one of the most important and challenging issues in the context of risk-management in modern societies. The subject involves addressing questions like "How safe is safe enough?" and "How much are we willing to pay to reduce the risks to acceptable levels?" [3, 46]. As stated in the introduction of the thesis (Chapter 1), it is essential to recognize that risk-reduction implies a certain cost and that the funds for this purpose are generally limited. Hence, the establishment of risk acceptance criteria should, as a rule, pursue the most efficient allocation of the available resources for risk reduction. As stated before, the present section will focus on acceptance criteria for risks to persons. A key issue is the distinction between individual and societal risk criteria [2, 47-50]:

- *Individual risk criteria* aim at limiting the risk exposure for individual persons.

- *Societal risk criteria* pursue the goal to limit the occurrence frequencies of events affecting groups of persons. This principle tries to capture the socio-political aversion to large-scale accidents with particularly severe consequences.

The need for approaching risks to persons from both perspectives, the individual and the societal, can be explained in a simple manner by means of Figure 2.2, adapted from [48]. It distinguishes, schematically, two situations, A and B, describing the risk exposure of a certain population to a risk source, for instance a liquid petrol gas (LPG) station, situated at a certain distance from the population. Due to the higher population density in Situation B, an accidental explosion of the LPG station could cause very severe consequences in terms of loss of human life. Since society shows generally an aversion towards accidents implying many fatalities [51], the decision whether Situation B would be acceptable or not should be based on an acceptance criterion for societal risk, where the frequency of occurrence of an explosion should be assessed in conjunction with the consequences such an explosion might entail. On the contrary, in situation A, the population density and hence the potential n° of fatalities in case of an explosion of the LPG station is relatively small, and thus the risk might be judged acceptable from the societal point of view. However, such a decision might be in gross contradiction with the preferences of the individual population members exposed to the risk in situation A, who might, irrespective of the fact that for society the risk would be assumable, plead to the safeguarding of the basic human rights for individuals [1]. Indeed, while the societal risk is smaller in situation A, the risk for an individual population member is the same in both situations, A and B. This clearly underlines the need for a distinction in risk criteria for respectively, individuals and the society. In the following, the commonly used approaches for defining both types of criteria will be exposed.

Figure 2.2 Schematic illustration of individual (IR) and societal (SR) risk; adapted from [48]

Individual risks

In the context of the present study, the annual probability of a person being killed describes the risk to an individual. In order to derive acceptance criteria related to the individual risk, it is often proposed to take a look at the implicitly accepted risks by individuals, revealed in accident statistics. The fact that the individual risk levels associated with different activities show statistical stability over the years and are approximately equal for different Western countries, indicates a consistent pattern of preferences [49]. The probability of losing one's life in normal daily activities such as driving a car (Figure 2.3) appears to be one or two orders of magnitude lower than the overall individual risk, which is of the order of 10^{-3} y^{-1} for a person of 30 years of age [52]. Only purely voluntary activities, such as, e.g. mountaineering, entail higher risk (Figure 2.3). The fact that the public tolerates greater risks from voluntary than from involuntary activities is commonly known. Moreover, the benefit obtained from the activity is a

decisive influence factor for the acceptability of risks to individuals. In general, the higher the benefit obtained from the activity, the higher is the willingness to accept the associated risks [49] (Figure 2.3).

Figure 2.3 Approximate individual risks for selected hazardous activities deduced from accident statistics and schematical representation of voluntariness and benefit on risk acceptance; adapted from [49]

Taking the average death rate from risk-neutral activities, such as car driving, of about 10^{-4} y^{-1} as a reference, Vrijling et al. [49] and Diamantidis [50] suggested to define the acceptable individual risk $r_{i,adm}$ as a function of a so-called policy factor η . This factor accounts for both voluntariness of the activity and the benefit obtained, and ranges from 0.01, for involuntary activities with no kind of compensation, to 100 for completely voluntary- and beneficial activities (Figure 2.3).

$$r_{i,adm} = \eta \cdot 10^{-4} \quad (\text{y}^{-1}) \quad (2.4)$$

Risks associated with structural collapse are invariably involuntary [47] and benefit is comparatively small. Consequently, the corresponding policy factor η should adopt small values, of about 10^{-2} to 10^{-1} , and thus reasonable values for the admissible individual risk due to structural collapse would be of the order of 10^{-6} to 10^{-5} y^{-1} . Indeed, this is consistent with the criteria laid down in standards, guidelines and technical papers on the safety requirements for built systems in general and structures in particular. The former version of standard ISO 2394 entitled “General principles on reliability for structures” [53], for instance, cites an average individual risk associated with structures of 10^{-6} y^{-1} as admissible. The Dutch Ministry of Housing, Spatial Planning and the Environment (VROM) recommendations for industrial activities distinguish between new and existing facilities, defining $r_{i,adm}$ to be 10^{-6} y^{-1} for the former and 10^{-5} y^{-1} for the latter [49]. The Swiss code on the assessment of existing structures [54] also defines the admissible level of individual risk to be 10^{-5} y^{-1} . That figure was likewise adopted by Steenbergen and Vrouwenvelder [55] to calculate the target reliability levels for existing structures. Similar developments by Sykora and Holicky [56] are based on $r_{i,adm} = 10^{-6} \text{ y}^{-1}$, further to the ISO standard [53]. It is also noteworthy that admissible individual risks for structures of the cited order of magnitude are consistent with the available, worldwide statistics for structural failure, mainly associated with extreme events like earthquakes, storms, landslides and flooding. Although these are inconclusive for several reasons, they allow for setting a rough fatality rate estimate of 10^{-6} to 10^{-7} per year for direct deaths and one order of magnitude larger for serious injuries [47].

Taking into account these considerations, it is possible to define a criterion for an individual risk-based target probability of structural failure ($p_{f,IR}$). As suggested by several authors [47, 49, 55, 56], this can

be achieved by considering the conditional probability of death of a person present in or around a structure at the time of collapse ($p_{d|f}$) in the following manner:

$$p_{ft,IR} = \frac{r_{i,adm}}{p_{d|f}} \quad (2.5)$$

The available estimations for the conditional probability $p_{d|f}$ are widely dispersed. Based on the qualitative definition of the consequence classes (CC) according to [11], Steenbergen and Vrouwenvelder [55] estimated conditional probabilities of 0.03 for CC2 and 0.3 for CC3. The statistical analysis of the databases on structural failures by [57], reported in [56], reveals lower probabilities $p_{d|f}$, especially what CC3 is concerned: 0.01-0.03 for CC2 and 0.03-0.05 for CC3. Finally, in a recent publication [58] the following values are proposed: $p_{d|f} = 0.05$ for CC2 and 0.2 for CC3. In general, a need for improving the estimates for $p_{d|f}$ is identified.

Societal risks

It seems generally accepted that frequency-consequence curves, also termed F-N curves, are a fairly accurate description of the societal risk involving multiple fatalities [51]. They show the relationship between the annual frequency F of events with N or more fatalities and are usually shown in a log-log plot [47]. Some examples are given in Figure 2.4. In the left part, taken from [52], some early proposals to the acceptance of risks stemming from nuclear reactor accidents in the United States are plotted and compared to frequency-consequence curves associated with different natural hazards. It was argued that the risks associated with a nuclear incident would be acceptable if they were of the same order as risks from a meteorite impact [52]. More “daily-life risks” are represented in the right part of Figure 2.4 [49] which shows the frequency - consequence relationships for different hazardous activities and installations in the Netherlands, among these, living in the surroundings of the Schiphol airport (Amsterdam) in regard to a possible plane crash, living or being close to a LPG filling station or the risk associated with traffic accidents causing larger number of fatalities, like bus accidents. Moreover, Figure 2.4 (right) contains the F-N acceptance criterion put forward by the Dutch authorities intended to limit the societal risks associated with particular technical facilities or activities. Note that it should serve as a guideline, though in [49] it is alluded to the fact that not all hazardous activities and installations comply with this criteria, as can be observed in the figure.

Figure 2.4 Examples for F-N criteria from [52] (left) and [49] (right).

In practical applications, it is usual to define the risk-acceptance criterion by two F-N curves, as shown in Figure 2.5 [50]. The upper curve represents the just tolerable limit above which risks are not acceptable, whereas the lower defines the limit below which risks might be accepted without further measures. The zone in between is called the ALARP zone (*As Low As Reasonably Possible*), where risk reduction measures should be considered and judged on an economical basis [2]. This principle has been applied in the past to various hazardous technical facilities, including the chemical and the transportation industries [50].

Figure 2.5 Schematic representation of F-N criteria illustrating the ALARP principle [50].

The mathematical definition of the F-n criterion is given by equation (2.6), where $F(1)$ and α are constants:

$$F(n) = p(N \geq n) \leq F(1) \cdot n^{-\alpha} \quad (2.6)$$

An exponent α greater than 1 accounts for social aversion to events implying greater consequences. Usually employed values are $\alpha = 1$ to 2 [2]. The Dutch criterion (Figure 2.4, right), for instance, limits the societal risk by a line that is inversely proportional to the square of the number of fatalities ($\alpha=2$), expressing lower accepted frequencies with increasing potential number of deaths.

The constant $F(1)$ represents the frequency of occurrence of an event with $N \geq 1$ fatalities. As a general rule, its value should be consistent with the reference system to which the F-n criterion is applied. As contended by Vrijling et al. [49], it seems preferable to start from a societal risk criterion defined on the national scale, taking all hazardous systems and activities into consideration. In other words, both constants, $F(1)$ and α , should reflect national safety policy [2]. Risk criteria for specific activities or locations should be established relative to this global criterion what can be achieved by determining the percentage of constant $F(1)$ applicable to the activity or location at issue [2].

The former version of international standard ISO 2394 [53] identifies the F-n criterion (2.6) as a possible requirement for structural safety, where the objective is to prevent accidents which may involve a large loss of human life. The standard recommends numerical values for constants $F(1)$ ($= 0.01$ or 0.1) and α ($= 2$) which, however, must be understood as merely indicative, for it specifies no reference system to which the F-n criterion applies.

The definition of an F-n acceptance criterion should be consistent with the expected number of fatalities $E(N)$ associated with the system and the reference period considered, i.e., the area under the F-n acceptance line (2.7). If this circumstance is not taken into consideration, the result may be irrational decision-making [59].

$$E(N) = \int F(n) \, dn \quad (2.7)$$

Further to Vrouwenvelder et al. [2], assuming a risk aversion exponent (α) greater than 1, expression (2.8) provides an approximate solution for integral (2.7). $N_{\max}$ is the upper integration limit, i.e., the maximum number of fatalities associated with an undesired scenario that may arise in the system at issue. The lower integration limit is set at $N_{\min} = 1$, although F-n criteria are often being defined for $N_{\min} \geq 10$ only (see for instance Figure 2.4), since more frequent accidents with a relatively small number of fatalities (for instance car accidents) are more suitably addressed in the context of individual risk acceptance [2]. The expression may be further simplified, as in (2.9), by defining $N_{\max} = \infty$, since the formulation is insensitive to variations in the upper integration limit.

$$E(N) = F(1) \cdot \frac{\alpha}{\alpha-1} \cdot \left(1 - \frac{1}{\alpha} \cdot N_{\max}^{1-\alpha}\right) \quad , \alpha > 1 \quad (2.8)$$

$$E(N) = F(1) \cdot \frac{\alpha}{\alpha-1} \quad , \alpha > 1, N_{\max} = \infty \quad (2.9)$$

For a risk neutral approach ($\alpha=1$), the solution for $E(N)$ can be expressed as follows:

$$E(N) = F(1) \cdot (\ln(N_{\max}) + 1) \quad , \alpha = 1 \quad (2.10)$$

Finally, in compliance with equation (2.5), it would be desirable to deduce a target failure probability based on the societal risk to persons. To that end, an admissible failure frequency $F(n)_{\text{adm}}$, expressed by (2.6), can be converted to a target probability ($p_{ft,SR}$) with equation (2.11), where subscript SR means societal risks and $p_{N|f}$ describes the conditional probability of $N \geq n$ fatalities in the event of failure [13].

$$F(n)_{\text{adm}} \cong p_{ft,SR} \cdot p_{N|f} \quad (2.11)$$

It should be noted that when applying this equation, care should be taken for keeping the analysis consistent in regard to the reference system to which the $F(n)$ criteria are suited for. Since failure probabilities refer generally to a particular failure mode of structural elements, frequencies $F(n)_{\text{adm}}$ should consistently refer to a system consisting of a single structural member only [13].

Relation between individual and societal risks

The above-described relations may also be used to establish a relation between the societal risks, on one hand, and the individual risks, on the other. As stated in several studies, for instance [2, 48, 60], both risk types may be related to each other by the number of expected fatalities $E(N)$. Based on the definition of individual risk r_i , $E(N)$ could, in most general terms, be obtained from equation (2.12) [48], where $m(x,y)$ is the population density at a considered location (x,y) and $r_i(x,y)$ represents the individual risk to persons present at this location. A drastic simplification of (2.12) is given by (2.13), where it is assumed that the totality of persons present at the location ($N_{\max}$) are exposed to the same individual risk r_i [60].

$$E(N) = \iint r_i(x,y) \cdot m(x,y) \, dx \, dy \quad (2.12)$$

$$E(N) \approx N_{\max} \cdot r_i \quad (2.13)$$

2.2 Principles of structural reliability

2.2.1 Uncertainties

Most of the decision problems in engineering are subject to uncertainties. Usually, it is differentiated between uncertainties due to inherent natural variability, model uncertainties and statistical uncertainties [1, 61].

Uncertainties due to inherent natural variability are also denoted physical or aleatory uncertainties. They are associated with the loading environment, the geometry of the structure, the material properties and the repair qualities [1]. According to ISO 2394 [53] they might be subdivided into those which can, and cannot, be affected by human activities. For instance, environmental loading characteristics, such as the snow load on a ground or wind speeds, belong to the latter category. The first category concerns, for example, the uncertainties associated with material- or geometrical characteristics of a structure. Such kind of uncertainties can be reduced by use of more advanced production and quality control methods, which on the other hand, may imply an increase in costs. Hence, up to a certain limit, the level of uncertainty can be chosen with regard to economic consequences. Therefore, the distinction between the two mentioned categories may be important [53].

The model uncertainties account for the uncertainty associated with the idealised mathematical descriptions used to approximate the actual physical behaviour of the structure. They must be considered in relation with models for both action effects and structural resistance [62]. The model uncertainty can be assessed as an auxiliary variable ξ (Figure 2.6), based on comparisons between the observed value Y (e.g. a test result) and the values predicted by a particular model, $\hat{Y}$ [63]:

$$\xi = \frac{Y}{\hat{Y}} \quad (2.14)$$

Finally, statistical uncertainties arise due to incomplete statistical information. They may result from different sources, such as, for example, a limited number of test results or neglecting possible correlations between certain variables. Statistical uncertainties can normally reduced by increasing test- and observational efforts [53].

Figure 2.6 Schematic illustration of estimation of model uncertainty statistics from a number of tests [62].

2.2.2 Basic variables and their stochastic characterization

Modern methods of reliability- and risk analysis allow for a very general representation of the above described uncertainties ranging from non-stationary stochastic processes- and fields to time-invariant

random variables [43]. In most cases it is sufficient to model the uncertain quantities by random variables with given cumulative distribution functions and distribution parameters estimated on the basis of statistical and/or subjective information [1]. The fundamental random variables, that define and characterise both actions on a structure and its resistance, are termed basic random variables, or just basic variables. Usually they correspond to the variables employed in conventional structural analysis and design, such as loads, geometrical dimensions or the basic mechanical material properties. The compressive strength of concrete, for instance, would be considered a basic variable though it can be related to more fundamental variables such as cement content, water-to-cement ratio, etc. [43, 61]. Naturally, the model uncertainties (Figure 2.6) must also be represented by basic variables.

The appropriate statistical characterization of basic variables by means of probability distributions and their parameters depend on the available knowledge. In many cases, distributions and parameters can be adjusted on the basis of statistical data obtained from observations of the properties of interest and/or subjective information [1]. In other cases this might not be possible and deterministic “point estimates” might be used instead [41].

When characterizing basic variables, it is convenient to resort to one of the well-defined statistical distributions. The normal distribution, for instance, is generally employed to represent the uncertainties associated with geometrical properties, densities or permanent loads. The resistance of materials is well represented by lognormal distributions, whereas variable loads, such as life loads in buildings or loads due to climatic influences, like snow or wind, are commonly represented by extreme-value distributions such as Gumbel or Weibull. A detailed description of these and other distributions and their properties is given in the literature, for instance [43], and will be omitted here.

In general, terms, there are two ways to represent a (continuous) statistical distribution (Figure 2.7): by means of a probability density function, $f_X(x)$, or its integral, the so-called cumulative distribution function, $F_X(x)$, related to each other by (2.15).

$$F_X(x) = P(X \leq x) = \int f_X(x) dx \quad (2.15)$$

Figure 2.7 Probability density function $f_X(x)$ (left) and cumulative distribution function $F_X(x)$ (right)

Probability distributions may be defined in terms of their parameters or, alternatively, by their moments. While some distributions (e.g., the normal distribution) are fully described by the first two moments only, i.e. the mean μ_X (1st moment) and the variance σ_X^2 (2nd moment), others, such as extreme-value distributions, might require higher moments too for an adequate characterization of the inherent distribution properties, such as the skewness (3rd moment) and the kurtosis (4th moment). However, in most practical applications only the first two moments are used, which are mathematically defined as:

$$\mu_X = \int_{-\infty}^{+\infty} x \cdot f_X(x) dx \quad (2.16)$$

$$\sigma_X^2 = \int_{-\infty}^{+\infty} (x - \mu_X)^2 \cdot f_X(x) dx \quad (2.17)$$

For purpose of a simplified representation, the distribution type (D) for a certain basic variable X_i is commonly designated by capital letters (e.g., N: Normal, LN: Lognormal, G: Gumbel, etc.) followed by the moments in parentheses:

$$X_i \sim D(\mu_{X_i}, \sigma_{X_i}^2) \quad (2.18)$$

In some practical problems, it might be necessary to consider dependencies between random variables. For this purpose, the continuous distribution functions corresponding to the basic variables X_i of an n -dimensional vector $\mathbf{X}$ can be “joined together” to a so-called joint probability density function $f_X(\mathbf{x})$. For the case of only two variables, this function is represented in Figure 2.8. The probability density functions of the random variables, $f_X(x)$ and $f_Y(y)$, are called marginal density functions since they can be represented by probability density functions on the margin. The joint probability density function $f_{X,Y}(x,y)$ shows a “hump” represented here by contour lines. From the shape and the direction of these lines, possible correlations can be detected [3]. Analytically, the correlation between variables X and Y can be expressed by the correlation coefficient $\rho_{X,Y}$ (2.20) as a function of the covariance $\sigma_{X,Y}$ (2.19):

$$\sigma_{X,Y} = \iint_{-\infty}^{+\infty} (x - \mu_X) \cdot (y - \mu_Y) f_{X,Y}(x,y) dx \cdot dy \quad (2.19)$$

$$\rho_{X,Y} = \frac{\sigma_{X,Y}}{\sigma_X \cdot \sigma_Y} \quad (2.20)$$

The variables are fully positive- or negatively correlated if ρ is 1 or -1, respectively, and independent of each other if $\rho = 0$. Attention should be paid to the fact that the correlation coefficient only recognises linear correlation whereas for correlations of higher order the analysis is more complex [3].

Figure 2.8 Joint probability density function $f_{X,Y}(x,y)$ and marginal density functions $f_X(x)$ and $f_Y(y)$.

Finally, it should be stated that in certain practical applications it might be convenient to mathematically combine certain basic variables. Consider for instance, the yield strength of a reinforcing steel bar, f_y , and its cross-sectional area, A_s , which in a form of a product could be merged to the yield force F_y . For purpose of a subsequent reliability analysis, it could then be desirable to determine both the distribution function and the corresponding moments of the resulting variable F_y . Some important computational rules for stochastically independent variables can be found in [3, 43], for instance. Among them, the

central limit theorem provides useful information on the shape of the probability density distributions for sums and products of independent variables. Provided that none of the variables dominates, this theorem says that the distribution of the *sum* of n arbitrary random variables X_i approaches the normal distribution with increasing n , independent of the distribution types of the variables. Moreover, it states that the distribution of the *product* of n arbitrary random variables X_i approaches the log-normal distribution with increasing n , independent of the distribution types of the variables [3].

2.2.3 Probability of failure and reliability index: FOSM and FORM concepts

The state of any structure may be classified as either satisfactory or unsatisfactory, where distinct conditions separating both are called limit states [64]. Limit states can be associated with a certain structural performance criteria. In accordance with [53], it can be assumed that such a performance criteria can be brought into the form of a function $g(\mathbf{X})$, where $\mathbf{X}$ is the n -dimensional vector of basic variables X_i that describe both the problem and the requirements for a particular basis of assessment. Based on this assumption and limiting the scope to problems related to structural safety- or failure, the safety requirement or desirable state for a structure can be expressed as:

$$g(X_1, X_2, \dots, X_n) > 0 \quad (2.21)$$

The distinction between the desirable (safe) and undesirable (failure) state or domain is given by the limit state function (2.22). The limit state concept is illustrated in Figure 2.9 [53], for the case of two basic variables, X_1 and X_2 .

$$g(X_1, X_2, \dots, X_n) = 0 \quad (2.22)$$

Figure 2.9 Illustration of limit state concept for the case of two basic variables [53].

Based on the above-exposed definitions of limit states, the structural failure probability p_f may be described, in its most general form, by means of equation (2.23),

$$p_f = P(g(\mathbf{X}) < 0) = \int_{g(\mathbf{X}) \leq 0} f_{\mathbf{X}}(\mathbf{x}) d\mathbf{x} \quad (2.23)$$

where $f_{\mathbf{X}}(\mathbf{x})$ is the joint probability density function (section 2.2.2) to be integrated over the failure domain, defined by the condition $g(\mathbf{X}) \leq 0$. Various methods for the non-trivial solution of the integral (2.23) have been proposed including direct integration, numerical integration, simulation techniques, such as Monte Carlo, or approximate, second-moment and/or transformation methods [43]. For they are very efficient and for they are used in the present thesis, the focus will be directed on the latter.

As an introduction to the problem, the basic reliability problem is considered, defined by a single load effect E and the corresponding resistance R [3, 43]. The limit state function is given by:

$$g = R - E = 0 \quad (2.24)$$

The corresponding failure probability can be expressed as:

$$p_f = P(R \leq E) = \iint f_{RE}(r, e) dr de \quad (2.25)$$

The volume of the joint probability density function $f_{RE}(r, e)$ of R and E is 1.0 and its contours are concentric curves (Figure 2.10). As explained before (section 2.2.2), the variables R and E are plotted as marginal probability density functions on the r and e axes, respectively. The limit state equation (2.24) separates the safe- from the failure region and divides the volume into two parts. The volume located in the failure domain corresponds to the failure probability. Moreover it should be noted that the point on the limit state equation where the joint probability density is greatest corresponds to the most likely point of failure, commonly referred to as *design point* (r^*, e^*) .

If R and E are independent variables, the joint probability function $f_{RE}(r, e)$ corresponds simply to the product of the marginal probability density functions $f_R(r)$ and $f_E(e)$ (Figure 2.10). The former can be substituted, by applying (2.15), in terms of the cumulative distribution function, $F_R(x)$, representing the probability that R is smaller than a given value x . The latter can be substituted for the probability that $E = x$, given by $f_E(x)$. Hence, (2.25) can be simplified into:

$$p_f = P(R \leq E) = \int_{-\infty}^{+\infty} F_R(x) \cdot f_E(x) dx \quad (2.26)$$

Figure 2.10 Joint probability density function $f_{RS}(r, s)$ and limit state surface for basic reliability problem

The integral expressed by (2.26) is known as the convolution integral. Only for a few distributions of R and E it is possible to obtain an analytical solution of this integral [43]. The most notable example is when both are normally distributed, with means μ_R , μ_E and variances σ_R^2 , σ_E^2 , respectively. Following this assumptions, the safety margin $M = R - E$, likewise normally distributed according to the central limit theorem (section 2.2.2), can be defined with a mean and variance given by well known rules for addition (or subtraction) of normal random variables [43]:

$$\mu_M = \mu_R - \mu_E \quad (2.27)$$

$$\sigma_M = \sqrt{\sigma_R^2 + \sigma_E^2} \quad (2.28)$$

The failure probability is then given by the probability that $M < 0$ (2.29), i.e. the integral of the probability density function of M , $f_M(x)$, between $-\infty$ and the origin (Figure 2.11).

$$p_f = P(R \leq E) = \int_{-\infty}^0 f_M(x) dx = \frac{1}{\sigma_M \sqrt{2\pi}} \int_{-\infty}^0 e^{-\frac{(x-\mu_M)^2}{2\sigma_M^2}} dx \quad (2.29)$$

 Figure 2.11 Probability density function $f_M(x)$ and reliability index β

By standardizing $z = (x - \mu_M)/\sigma_M$, expression (2.29) can be reformulated in terms of the standard normal distribution function Φ :

$$p_f = P(R \leq E) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{-\frac{\mu_M}{\sigma_M}} e^{-\frac{z^2}{2}} dz = \Phi\left(-\frac{\mu_M}{\sigma_M}\right) \quad (2.30)$$

The ratio μ_M/σ_M in (2.30) defines the so-called reliability index β according to Cornell [65]. It expresses how often the standard deviation σ_M may be placed between zero and the mean value μ_M (Figure 2.11). Failure probability p_f and reliability index β are hence related by:

$$p_f = \Phi(-\beta) \quad (2.31)$$

The above ideas for the specific case of two normally distributed random variables can be readily extended to the case where a linear limit state function (LSF) consists of n normal variables [43]. In most practical problems, however, the LSF is not linear and the first two moments are not easily obtainable. In this case, a suitable approach is to linearize the LSF by expanding it as a first-order Taylor series about some point x^* . The corresponding equations are given, for example, in [43].

Once the non-linear LSF has been linearized, the corresponding, first two moments (μ and σ^2), and hence β (and p_f), can be obtained as shown before. However, it was early recognized that this way of proceeding entailed a significant shortcoming. Precisely, it was found that the solution of β was sensitive to the formulation of the LSF, i.e., different, but mathematically equivalent formulations of the LSF led to different β , what was called the invariance problem [66].

Shortly after this discovery, Hasofer and Lind [67] presented a solution to this problem. They suggested to transform the limit state function into the so-called standard space by converting the basic variables X_i into standardized variables Z_i , characterized by a zero mean value and a unity standard deviation:

$$Z_i = \frac{X_i - \mu_{X_i}}{\sigma_{X_i}} \quad (2.32)$$

It should be noted that the transformation (2.32) can be performed without complication if the random variables X_i are all uncorrelated (i.e. linearly independent). On the contrary, an intermediate step is required to find a random variable set $\mathbf{X}'$ from the correlated set $\mathbf{X}$ which corresponds essentially to the

finding of the eigen-values and vectors [43]. Weak correlation ($\rho < 0.2$) usually can be ignored whereas strong correlations ($\rho > 0.8$) can be treated as fully dependent, i.e. with one of the two correlated variables expressed by the other. More detailed information can be found in [43].

Figure 2.12 illustrates the benefits from the transformation into the normalized z space, for the simplified case of two variables. The great rise of the joint probability density hump $f_Z(z)$ now coincides with the origin of the coordinates z_1 and z_2 . Due to the axial symmetry of the hump, all contours (points of equal probability) are concentric circles around the origin. Consequently, the coordinates of the design point (z_1^*, z_2^*) , and thus reliability index β , are defined by the shortest distance between the LSF and the origin. The determination of β according to Hasofer and Lind [67] corresponds thus to a geometrical problem, expressed by (2.33), wherefore it is also being referred to as *geometrical reliability index* [68].

$$\beta = \min(\sum_{i=1}^n z_i^2)^{0.5} \quad i = 1, 2, \dots, n \quad \text{subject to } g(z) = 0 \quad (2.33)$$

In (2.33), z_i represents the coordinates of any point on the limit state surface. The solution of (2.33) can be obtained by means of different iteration-based algorithms, described, e.g. in [43]. It should be noted that β and the design point coordinates z_i^* are related by the direction cosines α_i (Figure 2.12):

$$z_i^* = -\alpha_i \cdot \beta \quad (2.34)$$

Where:

$$\sum \alpha_i^2 = 1 \quad \text{and} \quad -1 \leq \alpha_i \leq 1 \quad (2.35)$$

The direction cosines α_i represent the sensitivity of the standardized limit state function $g(z)$ at z_i^* to changes in z_i [69] what has an important practical implication: it indicates the relative importance of the individual random variables x_i for the reliability index β and the failure probability p_f .

Figure 2.12 Contours of probability density function ($f_Z(z)$) contours and original (non-linear) and linearized limit state surfaces in the standardized normal space.

The described approach is also known as the First Order Second Moment (FOSM) method, since it is based on a first order approximation (linearization) of the (non-linear) LSF and considers only the first two moments of each random variable in the calculation of β and p_f . The presented developments can

be extended to the frequent case of non-normal basic variables. Basically, this can be achieved by transforming the non-normal distributions into equivalent normal distributions, for which purpose different algorithms have been developed over the years, such as the Rosenblatt- or the Nataf transformation, described e.g. in [43]. Once this transformation has been accomplished, the resulting normally equivalent variables can be used directly in the above-described FOSM procedure. This way of proceeding is called the “advanced” or “extended” FOSM method, likewise known as the First Order Reliability Method (FORM).

Finally, it should be emphasized that due to the introduced simplifications in the analysis (linearization of the LSF, approximate representation of non-normal variables, etc.), the probability of failure p_f loses its mathematical exactitude. Keeping in mind too that p_f depends on the uncertainties associated with the definition of the limit state function, on the probabilistic models used for the stochastic characterization of the basic variables involved, and the fact that p_f does not account for human errors (section 2.1.2), it is evident that the probabilities of failure can only be interpreted as notional values, which, instead of representing real failure rates, rather reflect the lack of knowledge about the performance of the structure [1].

2.3 Treatment of risks and reliability in design codes

2.3.1 Introduction

Standards and codes provide the basis of good engineering practice and a framework for implementing performance requirements in design rules [70]. While until the 1960's the safety criteria in standards were mainly based on allowable stress principles, the developments of computational- and structural reliability methods, as well as the advances in the field of material technology during the last decades, contributed to the implementation of a new philosophy for structural design. The limit state principle was introduced and safety verification was based on the LFD (Load Factor Design) and later LRFD (Load and Resistance Factor Design) in the USA and Canada, or on the similar partial safety factor method in Europe. Simultaneously, robustness requirements were developed to address system performance, especially in relation with the design for accidental actions [17]. The significant improvements were manifested in a number of important documents, standards and guidelines, among them the European standards [11]. Diamantidis and Vrouwenvelder [70] highlight the following advanced concepts for the design of civil engineering works, according to the current Eurocodes:

- Reliability and robustness are major issues in the Eurocodes and are treated with specific provisions, which are based on modern risk analysis and risk appraisal criteria.
- A reliability class differentiation scheme is proposed in the Eurocodes with three different reliability classes. A measure of reliability is the probability of failure or the associated reliability index, which depends on the reliability class (type of structure, consequences of failure).
- In the limit state design, reliability is reflected through the design values of the design parameters and consequently through the partial safety factors and characteristic values for the load and resistance parameters.
- Limit state design is based on the consideration of local and not global failure, since design equations are usually defined and applied on a local level only. The global reliability i.e. the reliability against collapse of the entire system is treated in the robustness requirements.

- Robustness requirements and associated design rules are given in the Eurocodes. Risk analyses are recommended for important structures associated to high consequences of failure.

In the following, the principles of reliability verification by means of the partial factor method, implemented in the Eurocodes and other international standards, will be analysed in more detail (section 2.3.2). Thereby the link between this semi-probabilistic approach and the probabilistic FORM (section 2.2.3) will be highlighted. Likewise, an analysis of the nominal target reliability levels prescribed in the codes will be offered (section 2.3.3), followed by some particular considerations regarding the accidental design situations in section 2.3.4.

2.3.2 Reliability verification based on design values

Reliability verification in the current structural codes and standards is based on the limit state principle. Two fundamentally different types of limit states are generally recognised:

- Ultimate limit states (ULS)
- Serviceability limit states (SLS)

The nature of ULS is essentially different from that of SLS. ULS are associated with collapse or other similar forms of structural failure, whereas SLS refer to structural performance indicators in service conditions (deflections, vibration, cracks, etc.). While the infringement of ULS might entail loss of structural integrity, and hence considerable risks to persons, the infringement of SLS is mainly restricted to economical consequences, such as repair costs. Moreover, in contrary to the ULS, the criteria associated with SLS might depend on the requirements of the client and users (sometimes very subjective), and on the characteristics of the installed equipment or non-structural elements [64]. These differences between ULS and SLS result generally in a separate formulation of reliability requirements for both types of limit states. In advance, the focus will be placed on the ULS.

Based on the general formulation of the reliability requirement expressed by (2.21), the design requirement may be written as [53],

$$g(x_{1d}, x_{2d}, \dots, x_{nd}) > 0 \quad (2.36)$$

where $x_{1d}, x_{2d}, \dots, x_{nd}$ are design values corresponding to a set of n basic variables X_i . Accordingly, the corresponding limit state function is expressed as:

$$g(x_{1d}, x_{2d}, \dots, x_{nd}) = 0 \quad (2.37)$$

The design values x_{id} can be obtained directly from the First Order Reliability Method (FORM). As explained in section 2.2.3, these values correspond to the individual components x_i^* of the design point vector $\mathbf{x}^*$, defined as the point on the limit state surface with maximum likelihood of failure (Figure 2.12). Based on the FORM, the design value x_{id} of a variable X_i following an arbitrary statistical distribution $F(x_i)$, can be established by (2.38), depending on:

- The assumed type of distribution of the variable X_i .
- The parameters or moments of the variable X_i (e.g. mean value and variance).
- The target reliability index, β_t .
- The sensitivity factors α_i .

$$F(x_{id}) = \Phi(-\alpha_i \cdot \beta_t) \quad (2.38)$$

Several codes and standards, like [11] or [36], offer tentative values for target reliability indices β_t , what will be addressed in section 2.3.3. The values of the sensitivity factors α_i should in principle be found from a number of representative FORM calculations. For purpose of convenience, a set of standardized α_i values has been developed based on experience, which is presented in Table 2.1.

Table 2.1 Standardized α_i values according to [11, 53].

	Variable X_i	Sensitivity factor α_i
Resistance (R)	Dominating variable	0.8
	Other variables	$0.4 \cdot 0.8 = 0.32$
Action or action effect (E)	Dominating variable	- 0.7
	Other variables	$- 0.4 \cdot 0.7 = -0.28$

According to the implicit approach adopted in daily practice (section 1.1.1), structural design codes formulate the design requirement expressed by (2.36) in terms of design equations, from which the reliability verification may be performed by a comparison of design values for resistance and action effects. Thereby, the design values are not directly introduced in the design equations. The basic variables are considered by means of their representative values (characteristic- or nominal value) in conjunction with a set of partial safety factors. This procedure is known as the partial factor method. In terms of design resistance R_d and design action effects E_d , requirement (2.36) is reformulated as:

$$g(x_d) = R_d - E_d \geq 0 \quad (2.39)$$

According to [53], the design values R_d and E_d can be expressed in the following way:

$$R_d = \frac{1}{\gamma_{Rd}} \cdot R\left(\frac{f_k}{\gamma_m}; a_{nom} \mp \Delta a; \dots\right) \quad (2.40)$$

$$E_d = \gamma_{Ed} \cdot E(\gamma_f \cdot F_k; \gamma_f \cdot \psi_0 \cdot F_k; a_{nom} \mp \Delta a; \dots) \quad (2.41)$$

Where:

$R(\dots), E(\dots)$: Resistance (R) and action effects (E) as a function of...

γ_{Rd}, γ_{Ed} : Partial safety factor covering uncertainty in the resistance model and the action and/or action effect model, respectively

f_k, F_k : Characteristic value of a material property (f_k) and of an action (F_k), respectively.

γ_m, γ_f : Partial safety factor for a material property (γ_m) and for actions or action effects (γ_f), respectively.

$a_{nom}, \Delta a$: Nominal value (a_{nom}) and deviation (Δa) from this value of a geometrical property

ψ_0 : Factor for combination value of a variable action

It should be noted that the Eurocodes [11] offer an alternative safety format, in which the uncertainties in the representative values of actions and those associated with the action- and/or action effect models, might be jointly represented by merging partial factors γ_f and γ_{Ed} into a partial factor γ_F . In the same way,

the uncertainties in the material properties and those associated with the resistance model might be represented by merging γ_m and γ_{Rd} into a partial factor γ_M . This simplified approach is adopted by many standards, among them the Spanish building codes for basis of design [71] and structural concrete [72].

Finally, it is important to stress that the link, established by the design point $\mathbf{x}_i^*$ (Figure 2.12), between the explicit probability methods, such as FOSM and FORM (section 2.2.3), and the semi-probabilistic partial safety factor method provides significant benefit. By means of probabilistic methods the safety format of the design codes used in daily practice, i.e. the design equations, characteristic values and partial safety factors, may be chosen such that the reliability level of structures and its members designed according to these codes is as close as possible to a certain target reliability level (section 2.3.3). This process is commonly understood as reliability-based “code calibration”. Code calibration studies have been carried out over the last decades by several researchers, e.g. Ellingwood and Galambos [73], Nowak [74], Sorensen et al. [75], Faber and Sorensen [76] or Markova and Holický [77].

2.3.3 Inherent and target reliability levels

The structural engineering profession has an exceptionally long tradition going several thousand years back. During these years, experience and expertise have been collected to some extent by trial and error. The design of new types of structures, with, for instance, new materials or subject to new loading conditions had to be performed in an adaptive manner based on suitable extrapolations of existing knowledge and experience [1]. In other words, new developments in structural engineering are traditionally based on well-tested structural design performance. This applies not least to the level of safety inherent to the codes according to which present structures are being designed. Indeed, it is a common approach to establish the fundamentals for making decisions regarding the acceptance of risks related to new structures on the basis of the risks experienced and apparently accepted in the past, associated with *best practice* [1]. As stated at the end of the previous section, this can be achieved by means of code calibration procedures, where the reliability level for the structures designed according to the new code rules it is often sought to be as close as possible to the reliability level associated with structures designed according to existing methods [75].

The Eurocodes, for instance, recognize that the design rules provided in the code are derived on the basis of “calibration to a long experience of building tradition” [11], along with the appreciation that “risks associated with structural failure, related to the present time are reasonable” [44]. Besides, several studies show that the reliability- and risk level inherent to the rules of the current Eurocodes is subject to a large scatter [78, 79]. While part of this scatter can be attributed to the required approximations and simplifications introduced in the codified design rules to facilitate their straightforward application in daily practice [44], yet significant scope for a more consistent calibration of these rules has been identified [9, 78, 79]. For instance, it has been suggested that future calibration exercises for code rules should account for reliability differentiation for the consequences of structural failure [9].

While the current, implicit Eurocode design rules may be subject to improvement in a sense of a more consistent calibration, yet the code offers the designer the possibility to perform explicit safety verifications or to deduce partial safety factors for implicit verifications in case the provided design rules may not be sufficient (e.g. for existing- or innovative structures). Therefore, a set of target reliability indices β_t is suggested, derived through long studies combining criteria for risks to persons and economical optimization [47]. The values, shown in Table 2.2, refer to component failure and take account of possible failure consequences by differentiation of reliability classes, associated with the consequences of structural failure. The values refer to periods of $T_{ref} = 1$ and 50 years. No specific rules are provided for the adjustment of the target reliability indices to reference periods different from these, what might be

necessary for bridges, for instance, where a design working life of 100 years is usually envisaged. According to the JCSS [80] a simplified approach for determining the life-time reliability of a structure is to multiply the annual failure probability by the design working life and a factor c which accounts for the dependency between different failure events within one year ($c = 1.0$ for independent failure events).

Table 2.2 Reliability differentiation according to [11].

Reliability class	Consequences for loss of human life, economical, social and environmental consequences	Target reliability index β_t		Examples of buildings and civil engineering works
		$T_{ref} = 1 \text{ year}$	$T_{ref} = 50 \text{ years}$	
RC3	High	5.2	4.3	Bridges; public buildings.
RC2	Medium	4.7	3.8	Residential and office buildings.
RC1	Low	4.2	3.3	Agricultural buildings.

Another set of target reliability indices has been proposed by the JCSS Probabilistic Model Code [80] and was later included in ISO 2394 [36]. As reported in [81], these target reliabilities are based on economic optimization principles. The values, which refer to a period of 1 year, are given in Table 2.3 as a function of the costs of the risk reduction measure and the consequences (failure costs), both defined relative to the initial construction costs. In [36], it is being specified that the target reliabilities may be used for failure events ranging from member failures, over partial- to total structural collapse by adjustment of the failure consequences. For instance, in case a member failure would trigger a system failure with large failure costs, the target reliability for that particular member should be higher than if the structural system would be redundant, since in that case the costs associated with the member failure might be lower [81]. In a similar way, it is suggested to distinguish the type of failure (ductile with reserve capacity, ductile without reserve strength and brittle failure) when choosing the target reliability level. A structural element which would be likely to collapse suddenly without pre-warning should be designed for a higher consequence class than one for which a collapse is preceded by some kind of warning and hence enables measures to be taken to avoid severe consequences [81].

Finally, it should be mentioned that standard [36] alerts to the fact that the target reliabilities given in Table 2.3 should be seen as indicative for the support of economic optimization, and might not be acceptable for what concerns life safety risks.

Table 2.3 Target annual reliability index β_t for ultimate limit states according to [80] and [36].

Relative Cost of Safety Measure	Minor	Moderate	Large
Large	3.1	3.3	3.7
Normal	3.7	4.2	4.4
Small	4.2	4.4	4.7

2.3.4 Accidental design situations

Accidental design situations refer to exceptional conditions applicable to the structure or its exposure [11]. In section 1.1.2 it has already been explained that the design philosophy for these situations gen-

erally differs from persistent situations referring to conditions of normal use. The general design objective for the latter, aimed at providing structural members with a sufficient resistance “to sustain, with appropriate degrees of reliability and in an economical way—all actions and influences than “*can reasonably be foreseen to occur*” or that are “*likely to occur*” during execution and use [11], is, by definition, somewhat conflicting when transferred to the context of accidental situations, precisely because accidental actions *cannot be easily foreseen*, and if so, they are *unlikely to occur* within the design working life period of the structure (section 1.1.2).

Recognizing these particularities, the Eurocode for accidental actions [17] suggests a variety of design strategies (section 1.1.2) based on a distinction between so-called *unidentified* and *identified* actions. The first refer to actions from an unspecified cause that might not be foreseen, as for instance, some kind of unforeseeable human error. The latter allude to actions which might be foreseen to occur (although their occurrence is unlikely within the design working life period of the structure), as for instance, a vehicle impact on an unprotected bridge pier or a gas explosion within a building supplied with gas.

The design strategies directed towards *unidentified accidental actions* are proposed to ensure a sufficient robustness of the structure [21]. For building design, the following strategies are specified:

- *design of key elements*: A uniformly distributed nominal load of $A_d = 34 \text{ kN/m}^2$ is recommended.
- *specifying the amount of acceptable damage induced by local failure*: The indicative limit is an building area of 100 m^2 or 15% of the floor area, whichever is less, on two adjacent floors caused by the removal of an load carrying column or wall.
- *Applying prescriptive design/detailing rules*: By three-dimensional tying in view of providing structural integrity, by ensuring structural redundancy or ductile member behavior, etc.

For *identified accidental actions*, some of these strategies, such as redundancy- or ductility-providing measures, might also apply. Moreover, in some cases, an identified action might be prevented from occurring, or protection measures can be applied in order to reduce or eliminate its effects on the structure (such as barriers to avoid a vehicle impact). Finally, the structure can be designed to sustain the identified action. In view of the objectives of the present thesis (section 1.3), focus will be directed in advance to the latter option.

For purpose of member design in the context of accidental actions, the Eurocode [11] offers the following specific expression of the general formulation (2.41) for design load combinations:

$$E_d = E(\sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i \geq 1} \Psi_{2,i} \cdot Q_{k,i}) \quad (2.42)$$

Where:

G_k :	Characteristic value of a permanent action
P :	Representative value of a prestressing action
A_d :	Design value of an accidental action
Q_k :	Characteristic value of a variable action
ψ_1 :	Factor for frequent value of a variable action
ψ_2 :	Factor for quasi-permanent value of a variable action

It should be noted that in comparison to the corresponding expression for persistent- or transient design situations no partial safety factors are to be applied to load effects within the accidental load combination. The design value of the accidental action is directly defined by means of a value A_d , which in practice often corresponds to a nominal value [18], such as in the case of gas explosions, where A_d is represented by a nominal, static equivalent pressure [17]. The reason for the use of nominal values is that a reliable statistical characterization of both occurrence and magnitude of accidental actions can only seldom be carried out [18] for the available data is generally poor. In any case, the Eurocode states that for individual projects, the suggested nominal values may be altered in accordance with the client [11, 17].

The design values of the variable actions Q_i are introduced by their representative values $\psi_1 \cdot Q_k$ or $\psi_2 \cdot Q_k$ which take account of the fact that it is highly unlikely that the different Q_i will attain their maximum at the time the accidental action occurs. The former ($\psi_1 \cdot Q_k$) corresponds to a frequent value, used as well for verification of reversible serviceability limit states, which is chosen so that the time it is exceeded is 1% of the reference period (for buildings) [11]. The latter ($\psi_2 \cdot Q_k$) is the so-called quasi-permanent value, likewise employed for verification of reversible serviceability limit states and for the calculation of long-term effects. For loads on building floors, the quasi-permanent value is usually chosen so that the proportion of the time it is exceeded is 50% of the reference period [11]. Moreover, it is worth mentioning that the choice of $\psi_{1,1} \cdot Q_{k,1}$ or $\psi_{2,1} \cdot Q_{k,1}$ may be made upon national specifications. In Spain, standard [71] which, regarding the safety concept may be considered equivalent to the Eurocode [11], employs the frequent value $\psi_{1,1} \cdot Q_{k,1}$. The recommended values ψ_1 and ψ_2 (identical in [11] and [71]) are given in Table 2.4 for the case of imposed loads (use category A [82]: domestic, residential areas) and snow loads in buildings.

Table 2.4 Ψ_1 and Ψ_2 values for imposed and snow loads in buildings according to [11], [71].

Action	Ψ_1	Ψ_2
Imposed loads (Cat. A [82])	0.5	0.3
Snow ($h > 1000$ m)	0.5	0.2
Snow ($h \leq 1000$ m)	0.2	0

Regarding the formulation of structural resistance in accidental design situations, the codes contain a specific set of partial safety factors for the materials. For concrete, a value of $\gamma_c = 1.3$ has been adopted in the national standard [72, 83], while Eurocode EN1992-1-1 [84] recommends a value of $\gamma_c = 1.2$. The partial safety factor for reinforcing steel is $\gamma_s = 1.0$ according to both codes, [72, 83] and [84].

2.4 Reliability of structural systems

2.4.1 Introduction

The structural failure probability, as described in section 2.2.3, refers to a specific failure mode of a particular element under exposure to a certain load arrangement, characterized by a particular limit-state function. Moreover, given the before-mentioned link between the probabilistic methods, and the partial factor method implemented in the design codes, the reliability verifications based on the design rules contained in these codes are likewise related to a specific failure mechanism of a structural member (or one of its cross-sections) in association with a certain limit state (section 2.3.2).

However, each structural system comprises a certain number of elements whose failure, either isolated or combined with the failure of other elements, can trigger a system failure. Also, different mechanisms

can lead to failure of a particular element (even a simply supported girder can fail due to bending, shear, etc.), and most systems can be subjected to many different load arrangements [85]. Due to these reasons, and despite the fact that the explicit evaluation of the conditional system failure probability $P(S|D)$, given an initial structural damage (section 1.2), is out of scope of the present thesis, it is considered necessary to include some basic considerations on reliability of structural systems.

Within the context of the present section, a structural system failure is understood as the formation of a collapse mechanism. For statically indeterminate systems, this implies that a combination of several failing *system components* is required for the entire system to fail [85]. Component failure refers here to a particular failure mechanism of a specific system-constituent member, or one of its cross-sections. Two idealized limit cases can be distinguished [3, 43]: series systems and parallel systems (section 2.4.2). However, real structural systems very often are mixed systems with series branches containing parallel elements [85], what will be addressed in section 2.4.3.

2.4.2 Series and parallel systems

As stated before, a particular structural failure mode can be represented by the corresponding limit state function, $g(\mathbf{X})$, as expressed by (2.22). Hence, for defining the condition of system failure, it is supposed that the limit state function is composed by several functions $g_1(\mathbf{X})$, $g_2(\mathbf{X})$, ..., $g_m(\mathbf{X})$, where m corresponds to the total number of system components C_i .

Figure 2.13 Series system (adapted from [3])

In a *series system* the individual components C_i are connected in series (Figure 2.13), and consequently the failure of a single component implies the failure of the whole system, wherefore it is also referred to as “weakest-link system” [53, 86]. The failure domain (undesirable state) for such a system can be expressed by the following condition:

$$g_1(\mathbf{X}) < 0 \quad \text{or} \quad g_2(\mathbf{X}) < 0 \quad \text{or} \quad \dots \quad \text{or} \quad g_m(\mathbf{X}) < 0 \quad (2.43)$$

This condition is illustrated in Figure 2.14 (left) for the simplified case of $m=2$ system components and a limit state function characterized by $n = 2$ basic variables (X_1 and X_2). The functions represented by $g_1 = 0$ and $g_2 = 0$ represent the limit state functions corresponding to the $m=2$ two system components. If statistical independency of these components is considered, the probability of a series system failure, P_f , corresponds to the sum of the probabilities of failure of each of the m system components, $p_{f,i}$:

$$P_f \approx \sum_{i=1}^m p_{f,i} \quad (2.44)$$

According to this definition, the probability of a series system failure, P_f , increases with the number of system components m . In case of perfectly correlated system components, the system failure probability corresponds to the failure probability of the most unreliable element:

$$P_f \approx \max(p_{f,i}) \quad (2.45)$$

Figure 2.14 Failure domains (shaded) for series systems (left) and parallel systems (right) [53]

In a *parallel system*, the individual components C_i are connected in parallel (Figure 2.15), and consequently the failure of a single component does not imply the failure of the whole system.

Figure 2.15 Parallel system (adapted from [3]).

Failure of the parallel system will not arise before all the m system components fail. For this case, the failure domain can be expressed by (2.46). A graphical illustration is given in the right part of Figure 2.14 for the simplified case of $m=2$ system components and a LSF characterized by $n = 2$ basic variables (X_1 and X_2).

$$g_1(\mathbf{X}) < 0 \quad \text{and} \quad g_2(\mathbf{X}) < 0 \quad \text{and} \quad \dots \quad \text{and} \quad g_m(\mathbf{X}) < 0 \quad (2.46)$$

Based on this failure requirement, and assuming statistical independency of the system components, the probability of a parallel system failure can be obtained by the product of the probabilities of failure of each of these components, $p_{f,i}$:

$$P_f \approx \prod_{i=1}^m p_{f,i} \quad (2.47)$$

If all elements are perfectly correlated, the parallel system failure probability will correspond to the failure probability of the most reliable component. Consequently, in this case:

$$P_f \approx \min(p_{f,i}) \quad (2.48)$$

2.4.3 Structural systems

Structural systems can seldom be represented by one of the previous system idealizations alone. Normally, they correspond to mixed systems which exhibit series connections with parallel system components [3]. Thereby, the capacity of the system and its components to redistribute loads and/or load effects, by means of ductile- and/or redundant structural behaviour, plays a major role. Moreover, it must

be taken into account that the system components may present more or less statistical dependency as they have common variables. For all these reasons, the determination of failure probabilities of structural systems is anything but easy and is usually restricted to estimation of approximate values only [85].

Figure 2.16 Schematic representation of principal failure mechanisms of span A-B-C of a two-span girder [85].

In order to illustrate the influence of ductile structural behavior on the reliability of a realistic structural system, the symmetrical, continuous two-span girder under an arbitrary load configuration (Figure 2.16), described in [85] will be considered. System failure arises if either of the spans A-B-C or A-B'-C' will collapse. Regarding span A-B-C, for instance, any of the following individual- or combined failure modes (represented in Figure 2.16) will induce a collapse mechanism and hence a system failure:

- Bending failure of section A, A_M . Given failure of section A, bending failure of section B, B_M .
- Bending failure of section B, B_M . Given failure of section B, bending failure of section A, A_M .
- Shear failure of section A, A_V .
- Shear failure of section C, C_V . Given failure of section C, bending failure of section A, A_M .
- Failure of section A due to interaction of bending and shear, A_{MV} .

Based on the consideration of statistical independency between these principal collapse mechanisms connected in series (Figure 2.16), the system failure probability of span A-B-C can be formulated by the sum of the occurrence probabilities for each of these mechanisms (2.44):

$$P_{f,A-B-C} \approx P_{f,A_M-B_M} + P_{f,B_M-A_M} + P_{f,A_V} + P_{f,C_V-A_M} + P_{f,A_{MV}} \quad (2.49)$$

For those collapse mechanisms characterized by a combined failure mode, the series elements are comprised by sub-systems consisting of two parallel components (Figure 2.16). The corresponding sub-system failure probability can then be evaluated by (2.47). For instance, probability P_{f,A_M-B_M} for the combined failure mode characterized by bending failure of cross-section A and subsequent bending failure of cross-section B, given failure of section A, is defined by,

$$P_{f,A_M-B_M} = P_{f,A_M} \cdot P_{f,B_M|A_M} \quad (2.50)$$

where p_{f,A_M} is the probability of bending failure of cross-section A, and $p_{f,B_M|A_M}$ the conditional probability of bending failure of cross-section B given previous failure of cross-section A. Assuming a brittle behavior of support cross-section A-A, it can be considered that once the ultimate bending moment is attained, the cross-section becomes inactive since the resisting moment M will drastically drop with increasing curvature χ (Figure 2.16). This brittle cross-section behavior will entail a change of the static system with a subsequent, and almost instantaneous, redistribution of the bending moments towards the span. Usually, cross-section B-B will not have enough strength to resist the resulting bending moment and will fail immediately causing span A-B-C to collapse. This kind of failure chain, where the failure of one cross-section causes the entire system to fail is referred to as *progressive collapse*, which is characteristic of brittle systems [85].

The situation changes, however, if a ductile behavior of cross-section A-A is assumed. In that case, after attaining its ultimate strength, the cross-section is characterized by a specific rotation capacity allowing it to stay active (Figure 2.16). Consequently, the redistribution of bending moments towards the spans will be progressive and collapse will not occur until the ultimate strength is attained in cross-section B-B.

From the foregoing considerations, it appears to be evident -at least intuitively- that the reliability of a system with a brittle behavior is considerably less than the reliability of a similar system characterized by ductile component behavior. Indeed, in [85] it is being demonstrated that the system failure probability related to bending-induced failure mechanisms of span A-B-C is notably smaller in case the cross-section is provided with sufficient rotation capacity and hence enables failure in a ductile manner. While the corresponding demonstration is being obviated here, it should be emphasized that the brittle failure mode described before is characteristic for statically determinate elements. For such elements, it can be considered that failure of one cross-section, in spite of the fact that this cross-section might behave in a ductile manner or not, entails necessarily the entire system to collapse, what, as mentioned before, is characteristic for series systems. From the perspective of structural reliability, statically determinate elements may thus be defined as extreme cases of statically indeterminate elements with brittle behavior. Consequently, one may arrive at the conclusion that the failure probability of a statically indeterminate system with ductile behavior is generally lower than compared to similar systems consisting of statically determinate members.

2.4.4 Final observations

The foregoing descriptions for the case of a relatively simple structural system give an idea on the numerous aspects that must be taken into account when analyzing reliability of systems. For more complex systems, like entire building structures, the here presented principles might still apply, but the analysis gets very involved. For instance, it requires the consideration of properties characterizing system redundancy, such as the ability of a structure to develop alternate load paths in the often-considered case of an accidental column removal in a building [87-90]. Such an analysis must address load-carrying mechanisms not normally considered in ordinary building design, as e.g., membrane action of floors, large deformations, catenary action or the ductility characteristics of both members and member connections. For analysis of system reliability, this calls for practical and simplifying mechanical and probabilistic models and methods. Recent studies focusing on reliability-based assessment [34] or optimization [86] of structural systems show that such an analysis is practically feasible, yet there is significant scope for further research and development.

2.5 Reliability of RC elements under moment-axial force interaction

2.5.1 Introduction

Any number of studies have been carried out in the past to quantify the reliability of reinforced concrete (RC) members, the vast majority being dedicated to one dimensional problems related to individual failure mechanisms, like bending, shear or compression failure, for instance [78]. However, many additional difficulties arise when the analysis involves a multidimensional problem [91]. A typical example of utmost importance is the reliability of RC members under the joint influence of axial forces (N) and bending moments (M). In this case the reliability assessment has to be performed in the M-N space, linked to a deterministic structural analysis procedure which accounts for material and, if required, geometrical nonlinearities. Thereby important issues such as load-path dependency, possible correlations between load effects and the eccentricity-dependant statistical characteristics of structural resistance have to be accommodated. For the reliability analysis of RC columns is being addressed in the present thesis, the following sections provide a summarizing review of the available literature on this subject.

2.5.2 Interaction diagram and load path dependency

The resistance of a RC member depends on both the material (e.g. the concrete compressive strength f_c , the reinforcing steel yield strength f_y , etc.) and the geometrical properties (e.g. the cross-section dimensions, etc.), all of which are random variables. Following the definitions in [92], let $\mathbf{X}_R$ denote the (time-invariant) vector of these random quantities, where R represents resistance. For a given value of $\mathbf{X}_R$, $\mathbf{x}_R$, the resistance of a RC column submitted to the combined effects of axial forces (N) and bending moments (M) can be represented by an *interaction diagram* (Figure 2.17). It is formed by the pairs of $M_{R,i}(\mathbf{x}_R)$ and $N_{R,i}(\mathbf{x}_R)$ that, according to the underlying definition of the ultimate limit state, will cause cross-section failure. The pairs $(M_{R,i}, N_{R,i})$ can be obtained based on strain compatibility assumption, stress-strain relationships of the constitutive materials and equilibrium formulations. The interaction diagram is defined implicitly, as the joint behavior of N and M is not expressed in closed form [93].

Two regions can be distinguished in the interaction diagram [93] as a function of eccentricity $e_{R,i}$ corresponding to the failure point (2.51): At small $e_{R,i}$, failure occurs when the concrete crushes before the steel has yielded in tension, termed a *compression failure*. At large $e_{R,i}$, the reinforcement yields first, followed by a compression failure in the concrete (in case of large axial tensile forces, by tensile failure of the reinforcement), what is termed a *tensile failure*. Both regions are separated by the so-called balance point $(M_{R,b}, N_{R,b})$, corresponding to simultaneous yielding of the tensile reinforcement and crushing of concrete in compression (Figure 2.17). The corresponding eccentricity is called balanced eccentricity, $e_{R,b}$.

$$e_{R,i} = \frac{M_{R,i}}{N_{R,i}} \quad (2.51)$$

Figure 2.17 Bending moment (M) – axial force (N) interaction diagram

The interaction diagram separates the failure region from the safe region in the M-N space (Figure 2.17), i.e. any arbitrary combination of load effects $M_{E,i}$ and $N_{E,i}$ which falls inside the area enclosed by the interaction diagram represents a safe state. Let such a point be given by $(M_{E,0}, N_{E,0})$ as represented in Figure 2.18. Assuming, for sake of simplicity, a linear relationship between M and N, this point defines, together with the origin of the coordinate system, the initial action effect vector $\mathbf{E}_0$. Any further (linear) increase in load effects can then be described by vectors $\mathbf{E}_i$ beginning from point $(M_{E,0}, N_{E,0})$ [94], as shown in Figure 2.18. Failure of the member is hence represented by any of these vectors $\mathbf{E}_i$ intercepting the interaction diagram at point $(M_{R,i}, N_{R,i})$, what will be the case if the length of vector $\mathbf{E}_i$, $|\mathbf{E}_i^*|$, exceeds a value $|\mathbf{R}_i|$ given by:

$$R_i = |\mathbf{R}_i| = \sqrt{\left| \frac{M_{R,i} - M_{E,0}}{h} \right|^2 + |N_{R,i} - N_{E,0}|^2} \quad (2.52)$$

The safety margin M (section 2.2.3) is given by (2.53), where $M = 0$ represents the limit state:

$$M = R - E = |\mathbf{R}_i| - |\mathbf{E}_i| \quad (2.53)$$

From a theoretical point of view, there are infinite possibilities for the directions of vectors $\mathbf{E}_i$ and hence for the definition of safety margin M. A conservative approach is the so-called *shortest distance* criterion [91, 92, 95], which defines the direction of a vector $\mathbf{E}_1$ such that $|\mathbf{R}_i|$ is minimized (Figure 2.18). The main advantage of this approach, which resides in its invariant character [91], faces the inconvenience for need of a cumbersome minimization algorithm [92] and the lack of practical relevance in many cases. It was early appreciated that the nature of the applied loads should determine the definition of the safety margin [93]. For instance, in many normal design situations, the analysis of RC columns can be restricted to gravitational loads only, such as dead- or life loads. For this case, the safety margin can be reasonably based on proportional growth of M and N (assuming that second order effects can be neglected), from the origin of coordinates to the failure point on the interaction diagram. This is referred to as *fixed eccentricity criterion* [93]. In Figure 2.18, this is represented by vector $\mathbf{E}_2$, who follows the direction of vector $\mathbf{E}_0$ up to the failure point $(M_{R,2}, N_{R,2})$.

However, there are many situations where the assumption of a constant eccentricity is not suitable, such as in the case of relevant horizontal load effects. Consider, for instance, a column initially submitted to gravity loads $(M_{E,0}, N_{E,0})$ and subsequently exposed to a horizontal force stemming from a wind action, as analyzed in [96]. In this case, it might be more appropriate to define safety margin and limit state

function based on the so-called *fixed axial load criterion*, reflected by vector $\mathbf{E}_3$ (Figure 2.18). While the axial force level in the column is kept constant, the safety margin is expressed in terms of bending moments only. One might consider as well an extraordinary loading condition, such as associated with an explosion in the interior of a building. In this case it might be reasonable to assume that the explosion pressure acting on floors and walls would entail an increase of the bending moment in the columns while the compression axial load level decreases [37], as represented by vector $\mathbf{E}_4$ and the corresponding failure point $(M_{R,4}, N_{R,4})$.

Figure 2.18 Different load paths in M-N interaction problems

Recognizing that the safety margins (2.53) corresponding to the four represented load vectors $\mathbf{E}_i$ are different, it is obvious that the consideration of the loading history (or the loading sequence) is crucial for an adequate reliability estimation of members exposed to M-N interaction. In this respect, it is highlighted in some contributions [91, 95] that columns (in buildings) are often designed for gravity loads only, based on the before mentioned *fixed eccentricity criterion* under the assumption of proportional growth of M and N (Figure 2.18, load path $\mathbf{E}_0$ - $\mathbf{E}_2$), while the actual loading history to which the member might be subjected during its design working life might deviate from the design assumptions.

Identifying the potential load paths to which a member might be subjected would correspond to the detection of hazards and hazard situations within the context of a systematic qualitative risk analysis (section 2.1.2). In the subsequent quantitative analysis, the member reliability related to the identified load-paths can be determined taking account of the corresponding uncertainties associated with the basic random variables characterizing both the loading and the member resistance (sections 2.2.1 and 2.2.2). In the following, a brief overview will be provided on how these variables and the associated uncertainties affect the reliability level of RC columns, followed by a brief description of the most relevant methods applied in the past to determine the reliability of such members.

2.5.3 Basic variables and their influence on reliability

Loads and load effect eccentricity

Each load path or loading sequence of a column exposed to M-N interaction can be stochastically characterized by means of probabilistic models taking account of the uncertainties associated with both the loads and the models used to determine the corresponding load effects. Of specific interest for M-N interaction problems is the uncertainty in the eccentricity. For instance, in the previously mentioned, *fixed eccentricity* approach, usually applied to the analysis of columns under the effect of gravity loads, it is

implicitly assumed that the applied axial force N_E and the corresponding bending moment M_E are perfectly correlated by eccentricity e_E [92, 97]:

$$e_E = \frac{M_E}{N_E} \quad (2.54)$$

The influence of varying correlations between N_E and M_E on the column reliability has been addressed in [92, 95, 98] considering correlation coefficients different from unity. It was generally observed that perfect (positive) correlation proves to be a conservative assumption when the failure mode is characterized by compression failure (Figure 2.17), i.e. for relatively small eccentricities $e_{R,i}$ (2.51). On the other hand, it was shown that perfect (positive) correlation between N_E and M_E might not be conservative in case the failure point is being situated in the tensile failure zone (Figure 2.17).

Another issue of interest in relation to RC column reliability corresponds to sustained load effects. As commonly known, such load effects may affect the concrete compressive strength and hence the resistance of the column. Moreover, sustained loads induce concrete creep, which in turn increases the total strain and hence the member deflections, what is particularly important in case of slender columns. To account for such effects, a time-dependent reliability analysis may be performed [96]. In many cases, however, simplifying assumptions can be adopted to convert the problem into a time-invariant analysis. In [97], long-term reliability of both short and slender concrete columns was found to be smaller than short-term reliability, being the difference of minor importance however.

Resistance variables

As stated before, the resistance of a RC member depends on both material and geometrical properties. Since these properties are all random, the resistance of the member is a random variable. The statistical analysis of the cross-section resistance has been addressed in a previous study on the reliability of RC columns under gravity loads in persistent design situations [99]. In this study, resistance R_i was defined by (2.52) with $M_{E0} = N_{E0} = 0$, i.e. assuming a fixed eccentricity $e_{R,i}$. Figure 2.19 shows the design interaction diagram (R_d) corresponding to a symmetrical RC cross-section, based on code specifications for the resistance of RC elements [72] and obtained by linear connection of the R_{di} values obtained at selected eccentricities $e_{R,i}$ (including $e_R = 0$ and $e_R = \infty$). The Monte Carlo technique was then applied to simulate 1000 samples of resistance R_i at each selected eccentricity taking account of the uncertainties in the basic variables influencing the resistance (f_c , f_y , b , h , d , A_s). The corresponding results were plotted in histograms, as shown in Figure 2.19 for the normalized, balanced eccentricity ($e_{R,b}/h = 0.6$). A subsequent statistical evaluation of the results delivered for each analyzed $e_{R,i}$ the mean value $\mu_{R,i}$ and the coefficient of variation, CoV_R . The interaction diagram corresponding to the mean μ_R is compared in Figure 2.19 to the design diagram R_d . As could be expected, μ_R exceeds R_d at all eccentricities. However, ratio μ_R/R_d is found to vary considerably. Highest ratios μ_R/R_d are obtained for small e_R and vice versa, a finding confirmed in similar studies [93, 100-102] dedicated to the reliability analysis of RC columns designed according to ACI or ASCE code specifications.

Figure 2.19 Interaction diagram for a RC cross-section based on design values (R_d) and mean values (μ_R)

In addition, the coefficient of variation, CoV_R , which represents the uncertainty in the cross-section resistance R_i , varies significantly with eccentricity e_R . In [93] it was shown that the concrete compressive strength f_c appears to be an essential influence parameter on CoV_R in case of relatively small e_R . Higher uncertainties in f_c were found to significantly increase CoV_R in the compression controlled failure zone. In case of larger e_R , however, the influence of f_c on CoV_R was observed to vanish at the same time the uncertainty in bar placement, represented by effective cross-section depth d , gained importance. The uncertainty in the yield strength of the reinforcing steel was likewise observed to significantly affect CoV_R in case of larger normalized eccentricities. Due to these disproportional influences, the coefficient of variation CoV_R was found to be higher in the compression failure region than in the tensile controlled failure zone, with a pronounced transition around the balance point. These findings were later confirmed in similar studies [100-102].

When performing reliability analysis of structural elements, model uncertainties (section 2.2.1) have to be accounted for in relation with both action effects and resistance. The latter seem to depend on eccentricity e_R . In [101], a coefficient of variation for the resistance model uncertainty (taking account of M-N interaction) of $CoV_{\xi_{R,MN}} = 0.065$ is suggested for the entire compression failure zone ($e_R < e_{R,b}$), reducing linearly to $CoV_{\xi_{R,MN}} = 0.03$ when approaching pure bending failure ($e_R = \infty$). In [97], where slender columns are analysed, a higher $CoV_{\xi_{R,MN}}$ for compression controlled failure zone is adopted: $CoV_{\xi_{R,MN}} = 0.11$. In both studies a normal distribution is recommended for the statistical representation of model uncertainty $\xi_{R,MN}$ with a bias (ratio of mean to nominal value) of 1.0.

The previous observations regarding the dependency of the statistical properties of column resistance (including model uncertainties) on the failure mode, points to the fact the column reliability is affected by the eccentricity $e_{R,i}$, ranging from higher reliabilities at small e_R to smaller reliability levels at larger e_R , as can be drawn from several studies [92, 97, 102, 103]. The transition zone around the balanced eccentricity $e_{R,b}$ is sometimes observed to be affected by fluctuations of the reliability index, probably due to the inaccuracy of approximate reliability methods (section 2.2.3) [92]. Tendentiously, the concrete compression strength f_c and the longitudinal reinforcement ratio ρ_l seem to have a substantial influence on the reliability level of both short and slender RC columns, especially in the compression controlled failure zone ($e_R < e_{R,b}$). In any case, the absolute reliability levels obtained in the different analysed studies depend on the numerous deterministic and probabilistic modeling assumptions and

simplifications adopted. A detailed comparison between the results of these studies is therefore difficult, if not at all impossible, and will be obviated herein.

Slenderness

Lateral deflection in so-called slender columns result in non-negligible secondary moments due to the axial load acting through an additional eccentricity [101]. Since second order problems are characterized by interaction between action effects and resistance, no closed-form solution exists to determine the equilibrium configuration of slender columns and iterative procedures must be applied instead. Simplified approaches for performing structural analysis of buckling critical members can be roughly classified into those where the first order bending moment is being complemented by a second order term (for instance, by means of a moment magnification factor or based on a nominal curvature), and those based on an equivalent reduction of the cross-section resistance. The herein analysed studies on reliability of slender RC columns are mainly based on the latter approach [97, 100, 101, 103]. The main advantage thereby is that, in case only gravitational loads are under consideration, the perfect correlation assumption of M_E and N_E underlying the fixed eccentricity approach (section 2.5.2) may be maintained [97]. The studies show that, tendentially, the reliability level of RC columns (of normal strength concrete) decreases with increasing slenderness ratio λ , which is defined by the ratio between the effective column length and the radius of gyration of the uncracked concrete section. The results presented in [95] show that for slenderness ratios $\lambda \leq 30$ no significant changes in reliability are to be expected. Diniz and Frangopol [97] who compared the reliability index β of short ($\lambda=0$) and slender columns ($\lambda=50$) made of normal and high-strength concrete, found that the latter experience a relatively small decrease in β . The results, which must be interpreted with caution however, for they depend on the particular modeling assumptions and simplifications adopted, show to be sensitive to the eccentricity e_R . Generally, the reliability of slender columns is more sensitive for small- and intermediate e_R , whereas when pure bending conditions are approached the influence of slenderness vanishes due to the decreasing influence of the axial force.

2.5.4 Reliability methods

Several approaches for computing the reliability level of members under M-N interaction have been suggested in the past, among them Monte Carlo (MC) simulations [91, 98], numerical integration techniques [95], approximate methods like FOSM, FORM or SORM [92, 95, 99, 104] or hybrid methods combining MC simulation and such approximate methods [97, 99, 100, 102, 103].

The perhaps most general approach, in a sense that it requires fewer simplifications than other methods, is the MC simulation. It is therefore probably the most suitable for the treatment of reliability problems related to M-N interaction [105]. Basically, MC techniques might be used for simulation of both the resistance, i.e. the interaction diagram, and the action effects. The failure probability can then be obtained by simply counting the outcrossings of the safe domain (Figure 2.17). Floris and Mazzucchelli [91] came up with a more elegant procedure where the shortest distance criterion (section 2.5.2) was established as safety margin and sampled by MC. This was followed by determination of the cumulative distribution function $F(\Phi)$ (Figure 2.7) of the safety margin. Since the limit state is reached when the safety margin is zero, the probability of failure simply corresponds to $F(0)$ [91].

The large numerical expense associated with direct MC simulation methods prompted the use of more efficient, hybrid approaches. These are based on a previous MC simulation of the resistance R_i corresponding to a certain (fixed) eccentricity $e_{R,i}$ as exposed before (Figure 2.19). Subsequently, the derived statistical parameters of R_i (μ , CoV) are used, along with probabilistic models for action effects (E), for

purpose of a reliability analysis by means of the efficient FOSM- or FORM algorithm. In case, R and E are normally distributed, the reliability index based on Cornell's approach [65] (2.30) can be deduced, as shown in [99, 102]. Otherwise, non-normal variables must be previously transformed into equivalent normal variables (section 2.2.3). The studies [97, 100] have been performed in this way.

The described hybrid approaches might not be suitable for problems where the uncertainty in the load eccentricity should be accounted for. Moreover, they still require cumbersome simulation- and evaluation techniques for the statistical analysis of the RC column resistance. Therefore, direct applications of the more efficient FORM and SORM algorithms have been proposed [92, 95, 99, 104]. The computational efficiency of these algorithms lies in the fact that, compared to the simulation techniques like MC, they call the limit state function a considerably fewer number of times [92]. In a previous study [99], a FORM-based procedure has been successfully applied to RC columns exposed to gravitational design loads under the assumption of a constant eccentricity. A generic description of this procedure and its application to the herein studied, explosion-exposed members will be given in section 6.1.

2.6 Summary

Chapter 2 provides a summary of relevant issues arising around risk and reliability in structural engineering.

The principles of risk-informed decision-making are introduced in section 2.1. A generally accepted and frequently applied procedure is introduced (section 2.1.2, Figure 2.1), divided into two principal stages: qualitative and quantitative risk analysis. The former primarily intends to identify the hazards to which a system may be exposed and to combine them logically in order to establish possible hazard scenarios. The latter delivers a quantitative evaluation of those scenarios in terms of their occurrence probabilities and their potential consequences. Based on both probabilities and consequences, the resulting risks may be assessed on the grounds of corresponding acceptance criteria.

The difficulties and challenges related to the establishment of such criteria are subsequently discussed (section 2.1.3). In keeping with the scope of the thesis, the most relevant criteria for risks to persons are outlined, from the perspective of both the individual and the societal risk acceptability. Individual risk criteria aim at limiting the risk exposure for individual persons. Taking the average fatality rate from risk-neutral activities, such as car driving, as a reference, acceptable individual risk levels are suggested in the literature as a function of both voluntariness of the activity and the benefit obtained (Figure 2.3). For admissible individual risks associated with structural collapse, which are invariably involuntary and entail comparatively small benefit, different studies report values of the order of 10^{-6} to 10^{-5} y^{-1} .

On the contrary, societal risk criteria pursue the goal to limit the occurrence frequencies of events with particularly severe consequences. It seems generally accepted that frequency-consequence curves ($F-n$) provide an accurate description of such criteria (Figure 2.4). Extremely important is to assure consistency with the size of the reference system the $F-n$ criterion is being applied to. Moreover, the $F-n$ acceptance criterion should be consistent with the expected number of fatalities $E(N)$ associated with the system and the reference period considered. Otherwise, irrational decision-making might be the result.

Section 2.2 addresses the principles of structural reliability. The different types of uncertainties involved in structure-related decision problems are summarized (section 2.2.1). In most cases, it is sufficient to model the uncertain quantities by means of random variables with given cumulative distribution func-

tions and associated distribution parameters (section 2.2.2). By means of these variables, structural performance criteria can be brought into the form of a limit state function (LSF) (Figure 2.9), which serves as a base for computation of structural failure probabilities. An efficient method for this purpose is the enhanced First Order Second Moment concept (FORM), which, based on a first order approximation (linearization) of the non-linear LSF, considers only the first two moments of each random variable (section 2.2.3). This method delivers the coordinates of most likely failure point on the limit state surface, termed FORM design point, for which a geometrical relationship to the reliability index β exists (Figure 2.12).

In section 2.3, it is shown how risk and reliability are treated in current structural design codes. The concept of limit state design is introduced (section 2.3.2). This concept is based on a semi-probabilistic approach, where the involved load and resistance parameters are established in terms of design values, for which a direct relationship to the coordinates of the FORM design point exists. This link provides the basis for a risk- or reliability based calibration of code design rules. Different sets of target reliabilities are suggested in codes and guidelines for such a purpose (section 2.3.3, Table 2.2, Table 2.3).

Limit state design is based on the consideration of local (member or component) failure, whereas the global reliability i.e. the reliability against system collapse is treated in robustness requirements. The general performance requirement stated in the Eurocodes is that the structure shall be designed and executed in a way that it will not be damaged by events such as fire, explosion, impact and the consequences of human errors, to an extent disproportionate to the original cause. As a result of this principle, local failure might be acceptable in accidental design situations provided that neither the whole structure nor an important part thereof will collapse (section 2.3.4). Associated design strategies for accidental actions involve, among others, the design of key elements, on which the stability of the structure, or a large part of it, depends, the provision of ductile member behaviour and design of a sufficiently redundant structural system that disposes of alternate load paths in case of a local member failure event.

Despite the fact that the explicit evaluation of the conditional system failure probability given a local structural damage is out of scope of the present thesis, some basic considerations on reliability of structural systems are nevertheless made (section 2.4). Two idealized limit cases are distinguished; series systems and parallel systems (section 2.4.2). An example (Figure 2.16) illustrates that one of these system idealizations alone can seldom represent real structural systems (section 2.4.3). Moreover, the example shows that the system failure probability is strongly dependant on the deformation capacity of the individual system components (cross-sections). Based on this appreciation, it is revealed that the failure probability of a statically indeterminate system with ductile behavior is generally lower than compared to similar systems consisting of statically determinate members. Significant scope for further research into the issue of system reliability is identified (section 2.4.4).

Finally, section 2.5 provides an overview on the reliability analysis of RC members under the joint influence of axial forces (N) and bending moments (M), which is an in many aspects challenging subject. The reliability assessment has to be performed in a multi-dimensional space, linked to a structural analysis procedure that accounts for material- and, if required, geometrical nonlinearities (section 2.5.2). Thereby important issues such as load-path dependency (Figure 2.18), correlations between load effects and the eccentricity-dependent statistical characteristics of structural resistance (Figure 2.19) have to be accommodated (section 2.5.3). Among the different approaches in place for computing the reliability level of members under M-N interaction, FORM-based methods have proven to be computationally efficient (section 2.5.4).

Chapter 3 Explosions and their effects on structures

3.1 Gas explosions

3.1.1 Introduction

The energy supply of buildings with gas for heating, cooking, and electricity generation implies the potential for an accidental explosion. A gas explosion can be defined as a process where combustion of a premixed gas-air cloud causes a rapid increase of pressure. The succession of the following circumstances is required for a gas explosion to occur [106].

- A *gas leakage or release* occurs as a result of technical defects in gas installations (pipes, boilers, etc.), human errors during installing, repair or maintenance of such installations, or in consequence of intentional manipulation.
- The released gas forms, with the oxygen present in the air, an *inflammable gas-air cloud* in compliance with certain physical requirements.
- A *delayed ignition* of the gas-air mixture through an ignition source (spark, hot surface, etc) occurs. If the ignition takes place immediately after the gas release, i.e. before an inflammable gas-oxygen cloud is being formed, a fire might occur, but the mixture will not explode.

The literature provides statistical data on the occurrence of gas explosions in buildings in different countries. An overview of this data is offered in section 3.1.2. Subsequently, section 3.1.3 provides insight to the principal physical backgrounds of the pressure build-up in a gas explosion in the interior of a building. Finally, section 3.1.4 summarizes available approaches for modeling the explosion pressure.

3.1.2 Occurrence rate

Over the last decades, several studies have been conducted with the aim to quantify the occurrence rate of gas explosions in buildings. One of the earliest surveys was performed by Burnett in 1975 [107]. Based on data collected by the American Gas Association and the Office of Pipeline Safety, the frequency of occurrence of piped-gas explosions in the United States between 1960 and 1970 was established. A distinction was drawn for gas explosions in general, and incidents involving damage in excess of \$1000. For the latter, an occurrence rate of about $\lambda = 2 \cdot 10^{-6}$ incidents per year and dwelling was observed (Table 3.1). The occurrence rate of gas explosions in general, i.e. irrespective of the damage involved was found to be about one order of magnitude higher. Almost the same conclusions were obtained by Leyendecker and Ellingwood [108], whose study covered the occurrence of gas explosions across the US in 1970.

Ellis and Currie [109] published data on the occurrence of explosions in the United Kingdom (UK), recorded between 1971 and 1994 by the Building Research Establishment (BRE). Mean frequencies of occurrence were deduced for a reference period between 1984 and 1994. Thereby, different damage levels were distinguished. The results are summarized in Table 3.1. For gas explosions in general, an annual occurrence rate of about $\lambda = 8 \cdot 10^{-6}$ per dwelling supplied with gas was determined, what according to the authors could be an underestimate since the survey probably does not include all minor incidents.

More reliable seems the deduced frequency for “significant explosions”, involving “some kind of structural damage”, of approximately $\lambda = 2 \cdot 10^{-6}$ per year. For more “severe explosions” entailing damage to heavier claddings, infills or partitions, as well as “weaker” loadbearing elements (e.g. gable walls, etc.), λ reduces to $5 \cdot 10^{-7}$ per year. Finally, “very severe incidents”, referring to explosions inducing failure of “stronger” loadbearing elements, like RC members, are characterized by annual occurrence probabilities of about $\lambda = 2 \cdot 10^{-8}$ per dwelling supplied with gas.

In [110], relevant data regarding the frequency of occurrence of natural (piped) gas explosions in Germany are given, based on data records by the organization Gas-und Wasserfach e.V. A mean value of about 42 gas explosions per year is reported for a 20-year reference period between 1981 and 2001. This corresponds to a mean frequency of occurrence of about $\lambda = 2 \cdot 10^{-5}$ explosions per year and dwelling supplied with piped gas. A smaller average value is indicated in [7, 24] for the Netherlands, namely $\lambda = 5 \cdot 10^{-6}$ explosions per year, along with the indication that higher values would be expected if small explosions (giving little damage only) would also be taken into account.

Data on the occurrence of gas explosions in Spain is provided in press releases. According to [111], 32 explosions *with at least one fatality* were recorded between 1998 and 2007. Considering representative for this 10-year reference period an average of about 6.5 million housing units with gas supply [112], the annual occurrence probability λ for an explosion with at least one fatality is about $\lambda = 5 \cdot 10^{-6}$. A similar calculation based on information in [28] delivers a value of $\lambda = 2 \cdot 10^{-6}$ explosions *with 3 or more fatalities* (Table 3.1).

Table 3.1 Annual frequencies of occurrence of gas explosions reported in different studies.

Study	Country	Period	Annual frequency of occurrence λ (per dwelling or per dwelling supplied with gas ^a)						
			Any gas explosion	Damage > \$1000	Any structural damage	Damage to heavier claddings and “weaker” loadbearing	Damage to “stronger” loadbearing elements (e.g. RC)	≥ 1 fatality	≥ 3 fatalities
Burnett [107, 108]	US	1960/70	$2 \cdot 10^{-5}$	$2 \cdot 10^{-6}$	-	-	-	-	-
Ellis and Currie [109]	GB	1984-94	$8 \cdot 10^{-6}$ a,b	-	$2 \cdot 10^{-6}$ a	$5 \cdot 10^{-7}$ a	$2 \cdot 10^{-8}$ a	-	-
Schmidt [110]	DE	1981-01	$2 \cdot 10^{-5}$ a	-	-	-	-	-	-
Vrouwenvelde and Leira [7, 24]	NL	n.a.	$5 \cdot 10^{-6}$ b	-	-	-	-	-	-
Press release [111]	ES	1998-07	-	-	-	-	-	$5 \cdot 10^{-6}$ a	-
Press release [28]	ES	2001-10	-	-	-	-	-	-	$2 \cdot 10^{-6}$ a

^b Excluding incidents with “little damage”

A comparison of the data summarized in Table 3.1 shows a good correlation among the findings reported. Interestingly, λ seems to be almost independent of the considered time period, what in light of the continuous modernization of gas installations and appliances could not have been expected. The data compiled in Table 3.1 suggests that $\lambda = 10^{-5}$ provides a good estimate for the annual occurrence of

a gas explosion in a gas-supplied dwelling. Moreover, it is shown that λ decreases with the severity of the explosion. Very severe incidents with the potential for failure of RC elements occur with frequencies of about two to three orders of magnitude lower than compared to gas explosions in general.

In several studies it is suggested that, assuming a uniform distribution in time, the occurrence of gas explosions may be modelled as a Poisson process [24, 108, 110]. The probability of occurrence $p(\text{EX})$ of at least one explosion within a given reference period T_{ref} can then be deduced from (3.1), depending on the probability of an explosion event per unit of time, λ .

$$p(\text{EX}) = P(x > 0 | T_{\text{ref}}, \lambda) = 1 - e^{-\lambda \cdot T_{\text{ref}}} \approx \lambda \cdot T_{\text{ref}} \quad (3.1)$$

3.1.3 Pressure generation

The pressure build-up during a gas explosion in the inside of a building is a consequence of combustion in a confined environment. The combustion process results in increased temperatures due to the transformation of chemically bound energy into heat. Thereby, the expansion of the combustion products, such as CO_2 and vapour, will be limited due to confinement by the building closings such as walls and floors, what will cause the pressure to increase [106].

In a completely confined compartment, such as pipes or closed vessels in industrial installations, the maximum pressure generated during a gas explosion will depend primary on the burning velocity (velocity of the flame front relative to the unburned gas immediately ahead of the flame [106]). This velocity depends on the composition of the inflammable gas cloud, i.e. the gas type and the proportions of gas and air. The highest pressures will arise if these proportions are such that there is no excess of fuel nor oxygen after the chemical reaction has been completed, what is referred to as the stoichiometric composition [106]. For methane gas, for instance, which is the principle constituent of natural gas, the highest explosion pressures will arise for gas concentrations of about 10%. Below a methane gas concentration of 4%, and above 17%, no explosion will occur. These so-called lower- and upper flammability limits depend on the type of gas involved, the initial pressure (p_0) and temperature (t_0). The mentioned values for methane gas correspond to ambient conditions ($t_0 = 20^\circ$, $p_0 = 1$ bar).

The ignition of a hydrocarbon gas-air cloud in a fully confined compartment might entail explosion pressures up to approximately 8 bars (800 kN/m^2) [106]. Gas explosion in buildings, however, do not cause pressures of this magnitude. The main reasons are imperfect mixing of the gas-air cloud and the fact that these explosions are only partly confined. Windows, doors, light partition walls or unrestrained brick walls act as venting elements, which, in case of failure provide explosion pressure relief. Table 3.2 [113] gives typical failure pressures of these elements. Experimental observations on failure of venting panels can be found in [114, 115].

Table 3.2 Typical failure pressures of different building elements [113].

Element	Typical failure pressure (kN/m^2)
Glas windows	2 - 7
Room doors	2 - 3
Light partition walls	2 - 5
Breeze block walls (50 mm)	4 - 5
Unrestrained brick walls	7 - 15

Figure 3.1, adopted from [110], shows an explosion pressure-time curve due to an explosion of a gas-air cloud in a cubic compartment equipped with a one-sided venting wall. The curve was deduced with

numerical simulations under use computational fluid dynamics (CFD). The pressure development in the compartment can be described as follows:

During the initial phase (characterized by point n° 1), which commences with the ignition of the gas-air cloud, the explosion venting panel is still undamaged and consequently the explosion pressure rises following the so-called “closed-vessel curve” [24], i.e. the curve which describes the pressure development in a fully confined compartment. As stated before, this development is principally influenced by the gas-type and the proportions of the gas-air mixture.

At the end of the initial stage, the venting panel bursts. As soon as the flame front reaches the opening (point n° 2), the combustion products as well as the unburnt gas can escape and, consequently, the pressure decreases (point n° 3). Generally, the higher are the failure pressures of the explosion venting panels, and the larger their size, the faster will be the pressure relief.

As shown in Figure 3.1, during the pressure relief through the venting panel, the flame surface in the compartment increases. Moreover, the venting process might entail the generation of turbulent flow fields. Both effects contribute to higher burning velocities, what in turn will induce a pressure rise. If the rate of pressure rise is higher than the rate that describes the simultaneous pressure relief due to venting, the pressure in the compartment will restart to increase (point n° 4) until the flame surface in the compartment, and consequently the explosion pressure, has reached its maximum (point n° 5). In other words, after breakage of the venting panel, the pressure build-up is governed by the balance between pressure generation by increased burning velocities, and the relief of the pressure through venting [106]. The corresponding physical backgrounds are complex since they depend on the interaction of numerous parameters including:

- The composition of the gas cloud (fuel-air mixture) and filling ratio of the room with this cloud.
- The geometry of the room where the explosion occurs (room proportions and volume).
- The type, size and geometrical distribution of the venting panels.
- The location of the point of ignition (with respect to the gas cloud and to the venting panels).
- The generation of turbulences due to obstructed flow around obstacles.

A detailed description of the physical relations between these parameters and the explosion pressure generation can be found in [106] and will be omitted here.

Figure 3.1 Simulated explosion pressure generation in a cubical, vented compartment [110].

For purpose of comparison to the numerically obtained pressure-time curve described before, Figure 3.2 shows a pressure-time record of one of the tests conducted by Dragosavic [116] to deduce a method for predicting the explosion pressure that may occur in a standard dwelling. Some similarities can be observed between the curves in Figure 3.1 and Figure 3.2, such as the qualitatively similar pressure-rise according to the “closed vessel curve” before venting occurs. However, some significant differences are also manifested. For instance, the test record shows that the pressure drop after the burst of the vents can reach negative values, which were observed to be of insignificant magnitude however. Another interesting observation in the test record (Figure 3.2) is the relatively long low-pressure period (about 0.5 seconds) after the initial pressure drop due to the burst of the venting panels. At the end of this period, a second, significant pressure-rise takes place, which is characterized by rapid fluctuations, probably in consequence of turbulent post-combustion of residual gas [116].

Figure 3.2 Explosion pressure generation recorded in full-scale test [116].

3.1.4 Load models

Generalities

The prediction of the pressure generated in a gas explosion is a difficult task, since, as explained in the previous section, pressure build-up is subject to very complex physical relations. Moreover, many of the

above-mentioned influence parameters on the explosion pressure, such as the composition of the gas cloud, the filling-ratio of the compartment or the location of the point of ignition are of random nature in a sense that they are not predictable at all. The most appropriate way to account for such random variables when modeling the gas explosion pressure is by means of probabilistic approaches. However, the lack of data hinders a reliable stochastic description of these variables and hence requires a certain degree of conservatism in the model formulations.

Several empirical models for the prediction of gas explosion pressure are available in the literature [116-119]. A summary and comparison of these models, which mainly depend on the geometry characteristics of the enclosure and the properties of the venting elements, is provided in [24]. The comparison reveals important differences between the model results, indicative of the above-mentioned difficulties associated with the pressure prediction. Moreover, it is evidenced that the empirical models should not be applied beyond the scope of the experimental studies they are based on.

Origin of the Eurocode model

Of special interest for the present study is the empirical model deduced in [116], shortly after the collapse of the Ronan-Point building (Figure 1.2). Dragosavic [116] conducted a systematic survey to study the effect of room geometry and different venting components on the pressure build-up inside a rectangular enclosure due to natural gas explosions. A total of 34 tests were carried out (out of which 7 were initially designated for purpose of calibration of the test instrumentation) under varying room dimensions and static failure pressures of the venting panels. In most of the tests, a stoichiometric composition of the gas-air cloud was provided. Before its ignition in the centre of the enclosure, the cloud was homogeneously distributed by means of ventilators.

The pressure was measured at several points on the enclosure's walls and ceilings. The qualitative pressure build-up in time, shown before (Figure 3.2), was similar in all the tests conducted. In line with the explanations in the previous section, two distinct pressure pulses were identified (Figure 3.2): a first pressure pulse with maximum p_1 , occurring immediately after bursting of the venting panels, and a second pressure pulse during the venting phase, with maximum p_2 , in many cases significantly larger than p_1 . It should be noted that p_2 does not represent the maximum of the represented high frequency pressures since these are of limited structural significance [24]. Instead, p_2 was derived by calculating a time-average of the fluctuating pressures, as indicated by the dotted line in Figure 3.2. Subsequently, peak pressures p_1 and p_2 were reduced to 80 % of their measured values in order to account for enhanced material resistance when exposed to high loading rates. Finally, the envelope was plotted to deduce a model that estimates the static equivalent explosion pressure in terms of the relationship between the static failure pressure of the venting components (Table 3.2), p_{stat} , and the so-called venting coefficient, i.e. the ratio of the area of the venting panels, A_v , to the volume of the enclosure, V . The nominal, static equivalent explosion pressure (p_d) proposed by Dragosavic was adopted by a number of structural design codes, Eurocode EN 1991-1-7 [17] among them. Figure 3.3 shows p_d versus A_v/V assuming $p_{stat} = 7$ kN/m² (violet curve). The model is valid within the limits $0.05 \text{ m}^{-1} \leq A_v/V \leq 0.15 \text{ m}^{-1}$ and $V \leq 1000 \text{ m}^3$.

Figure 3.3 Gas explosion pressure versus venting coefficient A_v/V : Design value p_d , nominal value $p_{key} = 34$ kN/m² for design of key elements against unidentified accidental actions [17] and peak pressure mean value $\mu_{pEX,peak}$ according to [108] and [110].

Models for estimation of the peak pressure mean value

Some years later, Leyendecker y Ellingwood [108] analyzed Dragosavic's test data from a statistical point of view. An expression for the peak pressure mean value $\mu_{pEX,peak}$ of a gas explosion in typical housing units was found with regression analysis, and is illustrated in Figure 3.3 (blue curve). Moreover, Figure 3.3 contains a model for predicting the mean value of the explosion peak pressure proposed in [110], based on CFD simulations of natural gas explosions in dwellings (green curve). The Figure shows that this model predicts far higher peak pressures than the EN 1991-1-7 design criteria [17] and the mean value according to [108], both based on test results [116]. Considering that these tests were conducted under the conservative assumption of a homogenously distributed, stoichiometric mix of gas and oxygen, which seems highly unlikely to occur under real circumstances, this finding may seem surprising. The numerical study [110], however, took other random parameters into consideration, whose effects would favour pressure generation. These parameters, which included the adverse location of the point of ignition with respect to the location of the venting panels and especially the generation of obstacle-induced turbulence, were not addressed in detail in the tests. This could explain the wide gap between the mean value based on a numerical approach and the empirically deduced models. However, for the time being no tests that might confirm the numerical results have been conducted. As highlighted by Vrouwenvelder and Leira [24], even for the most sophisticated CFD calculations a comparison with results from tests is crucial in order to verify that the modeling assumptions have been adopted in an appropriate manner.

It should also be stressed that the described peak pressure models were derived for explosions in single-room compartments. However, Ellis and Currie [109] point out that in case of large gas leakage rates, significant concentrations of gas might be built up in a number of interconnected rooms. In that case, an initial explosion in one room could propagate into others causing a progressively more turbulent situation with the potential for a much more violent explosion causing very high pressures. Such a type of "multi-room explosion", also termed a "cascade explosion", was probably in the origin of the Ronan-Point collapse (Figure 1.2) [109]. In [27] it is stated that in this incident a pressure of about 34 kN/m² caused the failure of the loadbearing wall that triggered the progressive collapse of the building, what seems to be in contradiction to [25], where it is reported that a pressure of about 21 kN/m² was required to displace this wall. In any case, the value of 34 kN/m² has since been adopted in the Eurocode as a notional pressure for the design of key elements against unidentified accidental actions [17] (section

2.3.4). Moreover, in the UK, it is recommended as a design criterion in large panel system buildings where a piped-gas supply is present [38]. As Figure 3.3 shows, the 34 kN/m² criterion (p_{key}) provides an upper bound to the overpressure models presented here. Specific models for the description of the pressure development in cascade explosions are not available in the literature, nor is this explosion type covered by current design codes [17].

Statistical distribution and scatter associated with peak pressure models

The fitting of a statistical distribution to the explosion (peak) pressure is a difficult exercise due to the short number of available test results. Further to the central limit theorem (section 2.2.2), Schmidt assumed a normal distribution [110], as did Leyendecker and Ellingwood [108]. Vrouwenvelder et al. [7] adopted a lognormal distribution in the context of a numerical example addressed to the analysis of robustness measures in a residential building.

From a maximum of seven tests under nominally identical conditions to study the scatter inherent in the generation of explosion pressure [116], a coefficient of variation of $CoV=0.3$ and 0.36 for the peak pressure loads p_1 and p_2 , respectively, was deduced in [108]. A even lower CoV of 0.22 is recommended in [110] based on numerical results. These relatively low values are in contradiction to the indications in [24] where, based on a comparison of the Eurocode model to test results published in [120], a CoV of around 0.7 is considered reasonable for the description of the scatter associated with the gas explosion peak pressure.

Pressure-time relationships

Gas explosions in buildings are regarded as a relatively slow type of explosion event [121]. In most cases they can be described as deflagrations, characterized by a flame front which travels at subsonic speed relative to the unburnt gas [106]. In comparison to detonations (often associated with explosives), where the flame front travels at supersonic speed [106], deflagrations are characterized by a relatively slow pressure build-up.

Figure 3.4 Typical idealized pressure-time relationships for deflagrations (left) and detonations (right)

Figure 3.4 shows typical idealizations of pressure-time relationships for the deflagrative- (left) and detonative (right) explosion types. For the latter, often a zero rise time is considered ($t_r = 0$) followed by a linear (or exponential) decay. For the former, of primary interest in the present study, Schmidt [110] suggested a symmetrical triangular pressure pulse ($t_r=t_p/2$). Figure 3.2 seems to confirm that this assumption is reasonable, especially for the second pressure pulse (p_2). Beshara [122] reports on recommendations from Bela [123] according to which the pressure pulse rise time t_r of an idealized triangular

pressure pulse of total duration t_p can be assumed to depend on the relation between the venting coefficient A_v/V and the volume V of the enclosure:

$$t_r = \frac{t_p}{2} \quad \text{for} \quad \frac{A_v}{V} \geq V^{-\frac{2}{3}} \quad (3.2)$$

$$t_r = \frac{t_p}{3} \quad \text{for} \quad \frac{A_v}{V} < V^{-\frac{2}{3}} \quad (3.3)$$

The total pulse duration t_p is indicated to rise with the volume V [122, 123]:

$$t_p = 0.2 - 0.3 \text{ s} \quad \text{for} \quad V \leq 100 \text{ m}^3 \quad (3.4)$$

$$t_p = 0.4 - 0.5 \text{ s} \quad \text{for} \quad 100 < V \leq 1000 \text{ m}^3 \quad (3.5)$$

$$t_p = 0.6 - 0.8 \text{ s} \quad \text{for} \quad V > 1000 \text{ m}^3 \quad (3.6)$$

The pressure pulses shown in Figure 3.2 acted for approximately 0.2-0.3 s on the closures of rather small room volumes of up to 36 m³, what seems to confirm the validity of (3.4).

3.2 Behaviour of RC elements under blast loads

3.2.1 Introduction

Explosion-induced loading on structures is generally associated with high energies and short durations. When exposed to such conditions, the material properties might be significantly altered. Moreover, structural elements experience accelerations and hence inertia forces will influence their behaviour. Non-linear structural response gains importance for it will contribute to dissipation of kinetic energy associated with the dynamic loads. In addition, such loads may induce higher structural vibration modes and thus action effects different from those expected under quasi-static loading situations. In summary, structural response and failure modes under explosion load exposure can be significantly different from those under static loads [124].

Section 3.2.2 provides a brief description of the mechanical behaviour of concrete and steel under short load exposure. Subsequently, section 3.2.3 explores structural response and failure modes of RC elements exposed to such loads.

3.2.2 Material behaviour

It is widely known that the mechanical properties of both concrete and reinforcing steel are sensitive to strain rate, what is generally referred to as “strain rate effect”. The action of an explosion pressure wave on a structural member is characterized by high loading rates, and consequently by enhanced material strain rates. As stated before (section 3.1.4) gas explosions are regarded as a relatively slow type of impact loading [121] and consequently the influence of strain rate is fairly small compared to high velocity impact scenarios (Figure 3.5). However, even under moderate strain rates of the order of about 10^{-3} to 10^{-2} s^{-1} , which a gas explosion pressure wave is expected to induce in concrete or reinforcing steel, some material parameters are sufficiently enhanced to be accounted for in the present study.

Figure 3.5 Typical strain rates for different dynamic loads [125]

A review of the literature reveals that a large number of experimental studies has been carried out in the past with the intention to analyze the dynamic properties of concrete and reinforcing steel. The vast majority of these studies are focused on finding the stress-strain relationships at different constant strain rates $\dot{\epsilon}$ and on deducing the corresponding dynamic increase factors (DIF) for the principal material characteristics describing these relationships, i.e. the ratio between the dynamic- and the quasi-static material property. In the following, a brief overview of these studies will be provided. Current simplified modeling approaches will be addressed in section 3.3.2.

Concrete

Figure 3.6 shows DIF's for the *concrete compressive strength* (f_c) compiled by Bischoff and Perry [126] from different experimental studies. The DIF are plotted versus the strain rate $\dot{\epsilon}$ (on a log-scale). It is clearly demonstrated that f_c generally increases with increasing strain rates. A qualitatively very similar diagram can be found in [127] for the DIF corresponding to the concrete tensile strength, f_{ct} , which is shown to be far more rate-sensitive than the compressive strength.

At first glance, the wide variation in the test results attracts attention in Figure 3.6, which seems to gain importance as the strain rate increases. This might be indicative of a rate-dependant scatter of the concrete compressive strength. However, the study by Mihashi and Wittmann [128] showed that the coefficient of variation for the strength of concrete and mortar specimen tested under high loading rates (of up to 10^{-3} s^{-1}) was not significantly influenced. This suggests that the large variation observed between the test results may be mainly attributed to different testing and measurement techniques, different methods to analyse or interpret the results and different specimen characteristics, such as size, shape and aspect ratios. Bischoff and Perry concluded that comparison of results may only be possible within a particular test programme [126].

Figure 3.6 DIF for the concrete compressive strength as a function of strain rate: Test results compiled from different studies (Figure adapted from [126]) and models according to [129, 130].

In Figure 3.6 it can be observed that, for very high strain rates, f_c experiences a disproportionately high increase. According to [131], the strain rate effects in the ranges below and above a so-called transition strain rate are dominated by different physical mechanisms. Following [132], it can be assumed that these mechanisms are essentially the same for concrete in compression and tension:

- For small to medium strain rates (below the transition strain rate), the influence of *crack propagation through aggregates* and *moisture effects* are of crucial importance.
- For very high strain rates (above the transition strain rate), *inertia effects on the formation and propagation of micro-cracks* will mainly control the increase of strength.

It is well known that under an increasing quasi-static load, the fracture behaviour of concrete will be governed by the formation and propagation of microcracks. The cracks mostly propagate through the weakest link in the composite system, i.e. the interface between aggregate and surrounding cement paste matrix. When exposed to high strain rates, however, the crack velocity increases [133] and, consequently, the time available for formation and propagation of microcracks will be reduced [126]. Instead of following the weakest path, the cracks are forced to propagate through regions of higher strength. This has been shown, for instance, in [134], where it was observed that the proportion of aggregates fractured under dynamic loading conditions is higher when compared to tests under quasi-static loading. Since fracture of the more resistant aggregates demands higher energies [135], the strength of concrete will increase when subjected to high loading rates. In [126] it is reported that the dynamic increase for f_c is more significant for lower concrete grades (lower compressive strengths). In the light of the previous explanations and taking account of the fact that, under quasi-static conditions, concretes of higher strength usually fail under a greater number of aggregate fractures than lower grade concretes, the finding of higher DIF's for the latter is indeed reasonable.

The influence of moisture content on the dynamic resistance of concrete has been shown to have an important effect on strain rate sensitivity in several studies. Rossi [136], for instance, compared the concrete tensile strength (f_{ct}) of saturated and dry specimen under different strain rates and observed that only the former exhibited a significant strength increase. It was concluded that this increase was mainly due to the contribution of the free water content in the concrete pores. In [137] it was shown that the concrete strength (f_c and f_{ct}) of dry specimen is relatively strain rate insensitive below the transition strain rates (for dry concrete) which are indicated to vary between 60 and 80 s^{-1} for concrete in

compression and 1 to 10 s^{-1} for concrete in tension. On the contrary, wet concrete was found to exhibit significant increases in strength at strain rates below as well as above the transition strain rates. More experimental evidence of the moisture effect on the concrete strength is given in [138].

The disproportionate strength increase above the transition strain rate can be traced back to a delay of micro crack propagation due to material inertia effects [131]. Since gas explosions do not induce such high strain rates, a more detailed description of the corresponding physical phenomena is out of scope of the present thesis.

It should also be mentioned that other concrete properties than strength are affected by strain rate effects. For instance, it is generally accepted that the *elastic (secant) modulus of concrete loaded in compression* enhances with increasing strain rates. On the contrary, the rate sensitivity of the *tangent modulus* is subject to more controversy [126]. According to [131] the initial concrete stiffness is less sensitive to strain rate than the stiffness at higher load levels, confirming thereby that strain rate effects are strongly influenced by damage formation. Also regarding the *concrete compressive strain at maximum stress* ϵ_{c1} there seems to be no clear consensus on whether higher strain rates induce an increase of this parameter. The experimental results compiled in [126] from several studies denote a significant scatter, including DIF's below unity. However, it is believed that the huge differences in reported test results are again indicative of the difficulties associated with test techniques, procedures and data evaluation [126]. Finally, it is worth to mention that the modulus (E_c) and the fracture energy of concrete in tension (G_f) are also strain rate sensitive [121]. As far as G_f is referred to, the dynamic increase might be very significant. However, the study of Weerheim and Van Doormaal [139] showed that such a significant increase only occurs if concrete is stressed under very high tensile loading rates ($> 15 \text{ GPa/s}$). In a recent study [140], it was observed that the dynamic increase of G_f measured in plain concrete specimens was much higher than the one obtained in steel fiber-reinforced concrete specimens, a result which the authors attributed to the smaller rate-sensitivity of fiber-matrix interaction than that of the unreinforced concrete matrix.

Reinforcing steel

Since the early study conducted by Manjoine [141], the behaviour of steel under dynamic loading conditions has been subject to research. Brandes et al. [142], for instance, conducted an extensive experimental survey where both hot rolled and cold worked reinforcing steel bars were submitted to tensile tests under different constant strain rates, ranging from $5 \cdot 10^{-5}$ to 8.5 s^{-1} . It was observed that the stress-strain relationships of steels exposed to high strain rates are qualitatively similar to those obtained for quasi-static loading. The strength characteristics, like the yield stress or the tensile strength were nonetheless shown to be substantially enhanced under high loading rates, where the former was found to be far more rate-sensitive than the latter. In addition, the strain at maximum load was significantly increased when dynamically loaded, especially in case of the cold worked steel. On the contrary, the properties in the elastic range, such as the Young's modulus, were found to remain unaffected from strain rate effects.

Similar conclusions were obtained in other studies. The perhaps most quoted work stems from Malvar and Crawford [143] who compiled data from several experimental surveys. Figure 3.7 shows the DIF's for the upper yield strength of reinforcing steels characterized by different grades, i.e. different static yield strengths, ranging from $f_{y,stat} = 378$ to 601 N/mm^2 . It is clearly shown that lower grade steels entail higher DIF's. Similar observations were made for the tensile strength, although, in line with the findings in [142], the corresponding DIF's were found to be substantially lower than those for the yield strength.

Moreover, it is interesting to mention that Malvar and Crawford [143] do not report any significant increase of the strain at maximum load what seems to be in contradiction to [142]. A possible explanation for this might be the differences in the type of steel analyzed, especially with regard to its deformation characteristics. Several authors, for instance [132], point out that the type of steel may significantly affect the strain rate sensitivity of reinforcing bars.

Figure 3.7 DIF for the upper yield stress as a function of time: Test results compiled from different studies and model according to Malvar and Crawford [143] (Figure adapted from [143])

3.2.3 Structural response and failure modes

For standard RC elements, two essential response modes can be distinguished: *flexural*- and *shear* response.

It is well documented that *flexural behaviour* of RC members is described by the moment-curvature relationship. The corresponding failure mode is characterized by initial cracking of the concrete and subsequent formation of plastic hinges at locations where the yield strength of the tensile reinforcing steel is being attained. Bending failure occurs due to crushing of concrete, when the ultimate compressive strain in the critical cross-section is being reached, or, in consequence of tensile failure of the reinforcement. Whenever the structure is able to undergo large deformations in the nonlinear domain, what implies a sufficient rotation capacity, the flexural failure mode will be ductile and energy absorbing. Providing RC elements with a significant energy absorption capacity by insuring a ductile structural behaviour is one of the key issues when designing for blast loads [121].

The desired ductile, flexural behaviour can only be developed when the shear capacity exceeds the flexural resistance of a member. However, several experimental studies reported in the literature show that structures designed to exhibit a ductile, flexural behaviour under quasi-static loading conditions might fail in *shear* when exposed to higher loading rates [144-147]. It has been found that this may be mainly associated with the influence of higher vibration modes. Whereas a quasi-static load cannot affect a member any more than in its fundamental mode, a load characterized by short rise times and corresponding high frequency content will more likely excite higher vibration modes giving rise to higher shear forces [148]. The following shear failure mechanisms can be distinguished [124, 149]: diagonal (compression- or tension controlled), direct and punching shear failure:

Diagonal shear failure is associated with inclined cracks occurring perpendicular to the principal tensile stress fields along the member (Figure 3.8, left). The studies performed by Magnusson et al. [145, 150]

showed that RC beams failing in diagonal shear under explosive loadings exhibited the same crack formation- and propagation characteristics as under quasi-static loads. Normally, a certain amount of flexure occurs prior to the development of diagonal shear cracks, and consequently, in many cases these cracks evolve from initially formed bending cracks. Therefore, diagonal shear is likewise referred to as flexural shear [148]. The principle mechanisms allowing for diagonal shear stress transfer across a crack are friction, aggregate interlock, dowel action and the contribution of transverse reinforcement. The provision of an appropriate shear reinforcement as well as proper detailing of the joints (for instance, beam-column connections) is especially important to enable significant deformation capacity and thereby to avoid brittle shear failure of RC elements subjected to blast loads [124]. The RC column shown in Figure 3.8 was tested under blast load and failed in diagonal shear [151]. However, it can be observed that it was able to undergo significant displacements in the plastic regime what contributed to dissipation of energy and thereby probably to avoidance of collapse.

Figure 3.8 Left: Schematic representation of diagonal (upper) and direct (lower) shear failure modes [148]; Middle: RC columns failed in diagonal shear after exposition to blast load [151]. Right: Failure curve of a RC element with fixed support conditions [148, 152].

As stated before, diagonal shear requires structural members to exhibit some form of flexural behaviour. In the case of *direct shear response*, however, no such requirement is needed [124]. Direct shear failure is characterized by a rapid propagation of vertical cracks (Figure 3.8, left) through the depth of the element [144] and usually occurs before significant flexural deformations have been developed [149, 153]. Since this implies a very reduced energy absorption capacity, direct shear is normally characterized by an extremely brittle failure mode [144]. Krauthammer [124], who studied the direct shear phenomena in detail, states that this type of response occurs frequently at areas of geometrical- or load discontinuities. Experimental evidence for such a brittle failure type is given in [154], where the slabs of RC box-type structures were subjected to extreme loads due to detonations of explosives. Failure curves for direct shear failure of RC elements, such as shown in the right part of Figure 3.8, were developed in [152] and used for purpose of a parametric study. The curve shows that the probability of direct shear failure increases with lower pressure rise times t_r and higher magnitudes. Although the represented values are strictly valid for a specific structural RC element only, the order of magnitude of the indicated rise times suggests that the direct shear failure mode is not relevant for gas explosions, which are characterized by rise times of the order of 0.1 s (section 3.1.4).

Another type of shear failure in RC elements is the *punching shear failure* mode. The most common problem under quasi-static loading situations is the punching of RC slabs supported by columns. In relation with blast loads, punching shear has been observed to occur in case of closed-in detonations of explosives (referring to a close distance between the source of the explosion and the target) and high velocity

impacts of rigid objects, with inclined shear cracks and the formation of conical shear plugs through the thickness of the member being the consequence [149]. In relation with explosions of deflagrative nature, however, neither punching shear failures nor other types of localized failures, in the form of spalling or cabbings of concrete covers [124, 149] have been reported.

It is well known that both flexural and shear behaviour of RC elements is being influenced when *axial forces* are present (section 2.5). RC columns in building structures are submitted to axial compression forces due to gravitational loads, which interact with the bending moments or the shear forces induced by lateral influences, such as the pressure stemming from an explosion. In case of gas explosions in the interior of buildings, it must be considered that the pressure acting on the floors and roof of the storey in which the explosion takes place, might induce a tensile force component in the column which counteracts the axial compression forces [30]. The behaviour of RC columns under this specific loading scenario is not addressed in detail in the literature. Available studies mostly focus on the interaction of the blast-induced bending moments with constant, compression axial forces, as described below.

Bao and Li [155] studied the effect of axial compression forces on the flexural- and shear capacities of RC columns by means of a previously validated Finite Element (FE) model. For relatively small, laterally applied explosion loads, they observed an enhanced flexural strength of the members, manifested by a decrease in their mid-height displacement, when axial forces were applied. However, when the blast-load intensity was increased, so that plastic hinges formed at the supports and at the mid-span cross-section of the both-sided clamped columns, the axial loads considerably amplified the lateral deflection and the internal moment due to second order effects. In some cases, the consequence was a rapid loss of strength due to buckling of the longitudinal reinforcement. Of crucial importance thereby was the influence of the transverse reinforcement. In addition to its contribution to the shear capacity of the columns, transverse reinforcement was found to provide confinement to the core concrete and lateral restraint against buckling of the longitudinal reinforcement. It was concluded that such restraint is vital for RC columns bearing compressive forces to ensure a ductile failure mode when exposed to lateral blast load effects. The same conclusion was obtained by Shi et al. [156] who analysed the residual load-carrying capacity predicted by a FE model of blast-loaded RC columns in buildings. In addition, in [157], where the response of blast-exposed RC columns was studied by means of a nonlinear, multi-degree of freedom model, it is stressed that the ductility capacity can be strongly influenced by the member stiffness and the axial load level. It was found that the higher the axial load and/or the stiffness, the lower was the column's end-rotation at failure and hence the corresponding energy absorption capacity.

3.3 Simplified modeling approaches for explosion-induced structural response

3.3.1 Introduction

A number of approaches are in place for determining internal forces and stresses in structural elements exposed to explosions, ranging from simplified models based on static equivalent loads to complex, numerical simulations of the blast wave and its dynamic effects on the structure. The election of an appropriate modeling strategy primary depends on the available information concerning the explosion loading characteristics. The *fib* Model Code [130] distinguishes three levels of approximations:

- *Level 1:* If the explosion load can be represented by a quasi-static equivalent load, a linear- or nonlinear static calculation can be done.

- *Level 2*: If the explosion load can be represented by an appropriate pressure-time curve, a linear- or nonlinear dynamic calculation using a time-step method can be performed.
- *Level 3*: For detailed modeling of the explosion load, hydrocode- or finite element (FE) analysis can be carried out and the internal forces and stresses can be obtained from these calculations. As an alternative, pressure-time curves can be derived from the hydrocode calculations and subsequently be employed in a level 2 type structural analysis (or in FE calculations [158]).

The sophisticated level 3 methods are supposed to entail the highest level of precision, especially when high non-linearities are involved, as usual in structural problems associated with explosion loading (section 3.2.3). Strain rate effects (section 3.2.2) can also be introduced in an appropriate manner in the numerical level 3 approaches by defining strain-rate sensitive constitutive material models [131, 132, 159]. Moreover, such approaches might be advantageous if structures of complex geometrical configurations or complex support- or boundary conditions are in the focus of the analysis.

On the other hand, it is obvious that the complexity, and hence the required effort and costs to perform the analysis, increases from level 1 to 3. Hydrocode- or FE models require a large amount of input data (for instance with regard to the definition of material models [160]), experience and knowledge to obtain reliable results. This in addition to the costs associated with the building of the model [157]. In many applications, the large effort associated with level 3 approaches might not be justified, especially if the uncertainties associated with the loads are high, as in the case of gas explosions (section 3.1.3 and 3.1.4). Indeed, if the magnitude of the loading is highly uncertain, the demand for a high precision in the structural response is somehow fallacious. Instead, level 2 calculations might be more suitable on the grounds of the reduced required effort. Such calculations are often based on simplified dynamic models, such as single degree of freedom (SDOF) models, which are usually efficient and simple to use [124].

In the present study, the dynamic analysis (Chapter 5) is restricted to beams and columns, i.e. elements of simple geometries and support conditions. The gas explosion pressure acting on these elements will be represented by an idealized pressure-time function, based on the considerations in section 3.1.4. Consequently, a dynamic analysis of level 2 type can be performed (Chapter 5) making use of simplified and computationally efficient dynamic models. On the grounds of these considerations, sections 3.3.3 (Flexural response), 3.3.4 (Flexural response under the influence of axial forces) and 3.3.5 (Dynamic reaction- and shear forces) describe the backgrounds of these simplified dynamic modeling approaches. Previously, in section 3.3.2, models to estimate the strain rate effects on the material properties are presented, which can be applied in conjunction with the structural models described in sections 3.3.3 to 3.3.5. Finally, section 3.3.6 shows some practical applications of the simplified modeling approaches for both flexural- and shear behavior, where the focus is placed on a comparison to experimental results. This is particularly important for an estimation of the model uncertainties associated with the dynamic load effects, such as displacements, bending moments or shear forces.

3.3.2 Strain rate effects

The present section addresses simplified models for quantifying the strain rate effects on the principal material properties of concrete and reinforcing steel (section 3.2.2).

The simplest approach to account for strain rate effects is to consider approximate dynamic increase factors (DIF) such as those given in Table 3.3 for the concrete compressive strength f_c , and the reinforcing steel yield- and tensile strength, respectively, f_y and f_t [124, 130, 161]. It should be noted that these values are intended for design of structures exposed to the effects of detonations, where the *distant design range* represents detonations at larger scaled distances to the structure than the *close-in design*

range. Since the pressure rise-times are much shorter for close-in detonations, the loading rates and, consequently, the recommended DIF's, are higher. The DIF's are intentionally conservative where the level of conservatism is related to the type of structural behavior. For instance, since the flexural response is generally more ductile than the shear response, the shown DIF's for bending are less conservative than those indicated for shear [124].

Table 3.3 Dynamic increase factors for design of RC structures [124, 130, 161].

Type of stress	Distant design range			Close-in design range		
	Reinforcing bars		Concrete	Reinforcing bars		Concrete
	$f_{y,dyn} / f_{y,stat}$	$f_{t,dyn} / f_{t,stat}$	$f_{c,dyn} / f_{c,stat}$	$f_{y,dyn} / f_{y,stat}$	$f_{t,dyn} / f_{t,stat}$	$f_{c,dyn} / f_{c,stat}$
Bending	1.17	1.05	1.19	1.23	1.05	1.25
Diagonal tension	1.00	-	1.00	1.10	1.00	1.00
Direct shear	1.10	1.00	1.10	1.10	1.00	1.10
Bond	1.17	1.05	1.00	1.23	1.05	1.00
Compression	1.10	-	1.12	1.13	-	1.16

Due to the conservative approach followed in deriving the values presented in Table 3.3, it is believed that they could be applied as well, and without introducing large error, in the context of gas explosions. An alternative and somehow more accurate approach consists in applying average strain rates experienced by the materials to calculate DIF's from corresponding model formulations [124]. The most relevant models for the present study will be presented in advance.

Concrete

Many studies have been conducted in the past to derive analytical- or empirical models for predicting the relative increase in concrete compressive strength (f_c) under high loading rates. The former Model Code 90 [129] contains a model based on the power function suggested by Mihashi and Wittmann [128] to predict the effect of rising strain rates on concrete strength up to a transition strain rate of $\dot{\epsilon}_c = 30 \text{ s}^{-1}$. Above this value, a cube-root law is proposed based on the work in [162], valid for strain rates $\dot{\epsilon}_c < 300 \text{ s}^{-1}$. This frequently quoted model, accepted by most researchers as an accurate representation of actual behavior [127], is described by equations (3.7) to (3.9) below.

$$\text{DIF}_{f_c} = \frac{f_{c,dyn}}{f_{cm}} = \left(\frac{\dot{\epsilon}_c}{\dot{\epsilon}_{c0}} \right)^{1.026\alpha} \quad \text{for} \quad |\dot{\epsilon}_c| \leq 30 \text{ s}^{-1} \quad (3.7)$$

$$\text{DIF}_{f_c} = \frac{f_{c,dyn}}{f_{cm}} = 10^{(6.156\alpha-2)} \left(\frac{\dot{\epsilon}_c}{\dot{\epsilon}_{c0}} \right)^{1/3} \quad \text{for} \quad |\dot{\epsilon}_c| > 30 \text{ s}^{-1} \quad (3.8)$$

Where:

$$\alpha = \frac{1}{5+0.9f_{cm}} \quad (3.9)$$

$f_{c,dyn}$: Concrete compressive strength under dynamic loading conditions (N/mm²)

f_{cm} : Mean concrete compressive strength, $f_{cm} = f_{ck} + 8$ (N/mm²)

$\dot{\epsilon}_c$: Strain rate of concrete in compression (s⁻¹)

$\dot{\epsilon}_{c0}$: Quasi-static strain rate, $\dot{\epsilon}_{c0} = 30 \cdot 10^{-6} \text{ (s}^{-1}\text{)}$

The model accounts for the previously mentioned, larger dynamic increase for concretes of lower strengths (section 3.2.2). This can be observed in Figure 3.6 where equations (3.7) and (3.8) have been plotted for mean compressive strengths of $f_{cm} = 33$ and 58 N/mm^2 . It is shown that for these concrete grades, the model predicts increases in compressive strength between 5 and 20 % when the material is exposed to strain rates that can be expected in the context of gas explosions ($\dot{\epsilon}_c \approx 10^{-3} - 10^{-2} \text{ s}^{-1}$).

Moreover, Figure 3.6 contains the model proposed by the current Model Code 2010 [130], expressed through (3.10) and (3.11). With respect to expressions (3.7) to (3.9), the model formulations are simplified since the dependency of the quasi-static concrete compressive strength f_{cm} has been eliminated. Important to stress is that, although not explicitly mentioned in the Code, (3.10) and (3.11) aim at predicting a characteristic value as denoted by subscript k within the definition of the dynamic strength, $f_{c,dyn,k}$. As Figure 3.6 shows, this model leads indeed to significantly lower predictions of dynamic increase compared to its precursor and seems to provide a reasonable lower bound to the experimental data, including for high strength concretes as shown in recent tests [163].

$$DIF_{fc} = \frac{f_{c,dyn,k}}{f_{cm}} = \left(\frac{\dot{\epsilon}_c}{\dot{\epsilon}_{c0}}\right)^{0.014} \quad \text{for} \quad |\dot{\epsilon}_c| \leq 30 \text{ s}^{-1} \quad (3.10)$$

$$DIF_{fc} = \frac{f_{c,dyn,k}}{f_{cm}} = 0.012 \left(\frac{\dot{\epsilon}_c}{\dot{\epsilon}_{c0}}\right)^{1/3} \quad \text{for} \quad |\dot{\epsilon}_c| > 30 \text{ s}^{-1} \quad (3.11)$$

Similar models are contained in the current Model Code [130] for the DIF's corresponding to the concrete tensile strength (f_{ct}), the modulus of elasticity (E_c) and the strain at maximum stress (ϵ_{c1}). The models in [130] for E_c and ϵ_{c1} have been adopted from the former code [129] without any modifications, whereas the DIF formulation for f_{ct} has experienced similar changes as reported above for the concrete compressive strength. Interesting to note is that in this formulation the change in slope occurs at 10 s^{-1} , instead of 30 s^{-1} . In [127], another model is suggested for describing the dynamic increase of f_{ct} , according to which the change in slope takes place at 1 s^{-1} .

When applying the presented DIF equations, care should be taken when a certain structural resistance model considers a specific material property in implicit terms. A good is example, although not of relevance for the present study, is the shear resistance of concrete members without stirrups. The resistance model in [72, 84] considers the concrete tensile strength f_{ct} implicitly by means of the compressive strength f_c . Hence, when applying the DIF formulation corresponding to f_c , instead of f_{ct} , the dynamic increase in shear resistance is likely to be largely underestimated.

Reinforcing steel

The most quoted- and applied model to predict the increase in reinforcing steel yield- (f_y) and tensile strength (f_t) under dynamic loading is based on the research by Malvar and Crawford [143]. The model is shown in equations (3.12) to (3.14) below, where f_s might refer to both f_y and f_t , what is specified through the exponent α . The formulation is valid for bars with a yield stress between 290 and 710 N/mm^2 , subjected to strain rates $\dot{\epsilon}_s$ ranging from 10^{-4} to 225 s^{-1} .

$$DIF_{fs} = \frac{f_{s,dyn}}{f_{s,stat}} = \left(\frac{\dot{\epsilon}_s}{\dot{\epsilon}_{s0}}\right)^{\alpha} \quad (3.12)$$

Where:

$$\alpha = \alpha_{f_y} = 0.074 - 0.04 \frac{f_{y,stat}}{414} \quad \text{if} \quad f_s = f_y \quad (3.13)$$

$$\alpha = \alpha_{f_t} = 0.019 - 0.009 \frac{f_{y,stat}}{414} \quad \text{if} \quad f_s = f_t \quad (3.14)$$

$f_{s,stat}$: Strength of reinforcing steel under static loading conditions (N/mm²)

$f_{s,dyn}$: Strength of reinforcing steel under dynamic loading conditions (N/mm²)

$\dot{\epsilon}_s$: Strain rate of reinforcing steel (s⁻¹)

$\dot{\epsilon}_{s0}$: Quasi-static strain rate, $\dot{\epsilon}_{s0} = 10^{-4}$ (s⁻¹)

It should be observed that irrespective of whether f_y or f_t is considered, the model formulation is a function of the former ($f_{y,stat}$) only. In Figure 3.7, where equation (3.12) (in conjunction with (3.13)) is graphically represented, it is shown that the model predicts higher DIFs for lower grade steels, matching reasonably well the experimental results compiled in [143]. For ordinary reinforcing steel of grade B500 ($f_{y,stat} = 560$ N/mm²) subjected to strain rates of the order of $\dot{\epsilon}_s \approx 10^{-3} - 10^{-2}$ s⁻¹, reached during material exposure to a gas explosion, the model predicts a dynamic increase of approximately 5-12 % in yield strength and 2-4 % in tensile strength. It should be mentioned that the model given by (3.12) to (3.14) has been adopted by the current Model Code 2010 [130] for use in structural problems governed by explosions or impact. The code alerts that the use of a DIF for reinforcing steel may only be possible if the steel has sufficient ductility [130].

3.3.3 Flexural response

SDOF model

For purpose of a flexural response analysis under dynamic loading conditions, many structural elements may be represented by a simplified, single degree of freedom (SDOF) system, as illustrated in numerous books [124, 164], guidelines and manuals [161]. An example is given in Figure 3.9, which shows a simply supported beam submitted to a dynamic, uniformly distributed load $\tilde{p}(t)$, as well as its idealization by means of an equivalent SDOF system, consisting of a concentrated (or lumped) mass connected by a spring to a fixed bearing.

The dynamic load $\tilde{p}(t)$ is being represented in the equivalent system by a forcing function $F_{eq}(t)$ representing both the magnitude of this load and its time dependant variations. The equivalent lumped mass m_{eq} should account for the member's self-weight as well as for a possible contributions stemming from permanent loads. Special attention has to be drawn to the spring properties that represent structural behavior by means of an idealized resistance function. In order to account for energy dissipation due to inelastic deformations (section 3.2.3), a linear elastic-perfectly plastic resistance function may be associated with the spring (Figure 3.9). In the elastic range, equivalent resistance and deflection are related by the equivalent stiffness k_{eq} . At $\delta = \delta_y$, corresponding to the transition of the elastic to the plastic response, the maximum spring force $R_{y,eq}$ has been attained. As the displacement increases further, this value is kept constant until the ductility capacity of the structural element is reached at δ_u . In case the displacement attains a maximum value $\delta_{max} < \delta_u$, the rebound phase will be entered where the resistance decreases along a line parallel to the initial elastic slope.

Figure 3.9 Representation of a beam by an equivalent SDOF system.

The behavior of the SDOF system is mathematically described by a set of (differential) equations of motion. According to the assumed elastic-plastic resistance function (Figure 3.9), three branches are being distinguished in the formulation of those equations, i.e. the elastic (3.15), the plastic (3.16) and the unloading branch (3.17). The equations of motion represent the dynamic equilibrium between inertia forces and resistance, on the left side of the equals sign, and the loading of the system on the right.

$$m_{eq} \ddot{\delta} + k_{eq} \delta = F_{eq}(t) \quad \text{if} \quad 0 \leq \delta < \delta_y \quad (3.15)$$

$$m_{eq} \ddot{\delta} + R_{y,eq} = F_{eq}(t) \quad \text{if} \quad \delta_y \leq \delta < \delta_u \quad (3.16)$$

$$m_{eq} \ddot{\delta} + R_{y,eq} - k_{eq}(\delta_m - \delta) = F_{eq}(t) \quad \text{if} \quad (\delta_m - 2\delta_y) \leq \delta < \delta_m \quad (3.17)$$

It should be noted that structural damping has been neglected within the preceding equations. Though it is well known that the response of a structural element to a dynamic load involves a certain degree of damping, its effect on the maximum response of blast-loaded elements, which is generally of main interest, can be almost neglected [124, 164]. The main reason for that is that the energy dissipated by plastic structural deformations is much higher than the energy absorbed by structural damping.

The characteristics of the equivalent SDOF system are determined so that the deflection δ of the lumped-mass is the same as that for some significant point on the real structure, such as the mid-span cross-section of the beam represented in Figure 3.9. In particular, the equivalent system parameters in equations (3.15) to (3.17), i.e. the equivalent mass, stiffness, resistance and load, respectively, m_{eq} , k_{eq} , R_{eq} and F_{eq} , may be obtained from energy- and external work relationships between the equivalent and the real system. Therefore, an assumption concerning the deformed shape $\phi(x)$ of the considered structural element is required. Since the flexural response is generally well described by the fundamental vibration mode, it is normally assumed that this shape can be considered to be the same as that resulting from a static application of the dynamic load [164]. Prior to yielding, $\phi(x)$ can be deduced from elastic beam theory, whereas in the post-yield stage the formation of plastic hinges might be assumed. This is shown in Figure 3.10 for simply supported members under a uniformly distributed blast load. The corresponding $\phi(x)$ are given by (3.18) and (3.19), respectively, where l denotes the member span.

 Figure 3.10 Deformed shapes $\phi(x)$ of simply supported members under uniformly distributed loads in the elastic (left) and plastic (right) response stage [164].

$$\phi(x) = \frac{16}{5l^4} (x^4 - 2lx^3 + l^3x) \quad \text{if} \quad 0 \leq \delta < \delta_y \quad (3.18)$$

$$\phi(x) = \frac{2x}{l} \quad \text{if} \quad \delta_y \leq \delta < \delta_u \quad (3.19)$$

For convenience of calculation, the relation between the idealized system parameters and their respective quantities in the real system is established by transformation factors, K_i [124, 164]. A mass factor K_M is defined as the ratio between the mass of the equivalent system and the mass m of the real system:

$$K_M = \frac{m_{eq}}{m} \quad (3.20)$$

The equivalent mass m_{eq} is deduced from (3.21) based on the balance of kinetic energy, where $\bar{m}$ represents the mass m per unit length of the real system.

$$m_{eq} = \int \bar{m} \phi^2(x) dx \quad (3.21)$$

In a similar way, a load factor K_L may be deduced based on equilibrium of external work imparted to the system:

$$K_L = \frac{F_{eq}}{F} \quad (3.22)$$

For an uniformly distributed load $\tilde{p}$, the equivalent load F_{eq} on the SDOF system is given by (3.23), whereas the total force F acting on the real system will be simply $F = \tilde{p} \cdot l$.

$$F_{eq} = \int \tilde{p} \cdot \phi(x) dx \quad (3.23)$$

Since the structural resistance R represents a force that pushes against the applied force F , the resistance factor $K_R (= R_{eq}/R)$ is the same as the load factor K_L [124]. The resistance R and deflection δ are related in the elastic range by (3.24) and (3.25) for the real- and the equivalent system, respectively. Thereby k represents the member stiffness prior to yielding (Figure 3.9), which in case of a simply supported member under distributed load is given by (3.26). It is hence obvious that the stiffness factor $K_k (= k_{eq} / k)$ must equal the resistance factor and thus $K_k = K_L = K_R$.

$$R = k \cdot \delta \quad (3.24)$$

$$R_{eq} = k_{eq} \cdot \delta \quad (3.25)$$

$$k = \frac{384EI}{5l^3} \quad (3.26)$$

It is also convenient to introduce the so-called load-mass factor [124, 164], K_{LM} , defined as the ratio of the mass and load factors ($K_{LM} = K_M / K_L$). The natural period T of the equivalent system can then be expressed as per (3.27). It should be noted that since the time scale is not altered within the analysis, T represents as well the natural period of the real system.

$$T = \frac{2\pi}{\omega} = 2\pi \sqrt{\frac{m_{eq}}{k_{eq}}} = 2\pi \sqrt{\frac{K_M \cdot m}{K_L \cdot k}} = 2\pi \sqrt{K_{LM} \frac{m}{k}} \quad (3.27)$$

Based on the foregoing equations and the deformed shapes shown in Figure 3.10, the mass, load and load-mass factors given in Table 3.4 can be deduced. Values for other support- and loading conditions are tabulated in [124, 161, 164]. It should be mentioned that in practice the transition from the assumed shape in the elastic stage to the plastic stage will be progressive, i.e., the plastic hinge (Figure 3.10) will

not arise instantaneously. To account for that, the transformation factors K_i corresponding to the plastic stage should as well experience a progressive change. However, for sake of simplicity this is normally disregarded and, instead, the K_i corresponding to the plastic stage are instantaneously used as soon as the end of elastic stage has been reached.

Table 3.4 Mass (K_M), load (K_L) and load-mass (K_{LM}) - factors for simply supported members under uniformly distributed load [164].

Response stage	K_M	K_L	K_{LM}
Elastic	0.50	0.64	0.78
Plastic	0.33	0.50	0.66

Finally, the forcing function $F_{eq}(t)$, applied to the equivalent SDOF system is defined by (3.28) as the product of the equivalent peak force $F_{eq,peak}$, obtained from (3.23) setting $\tilde{p} = \tilde{p}_{peak}$, and an load-time relationship, $f(t)$. For a symmetrical triangular load-pulse ($t_r=0.5 \cdot t_p$), such as represented in Figure 3.4 (left), this relationship can be described by (3.29) and (3.30).

$$F_{eq}(t) = F_{eq,peak} \cdot f(t) \quad (3.28)$$

$$f(t) = 2 \cdot \frac{t}{t_p} \quad \text{if} \quad 0 \leq t < \frac{t_p}{2} \quad (3.29)$$

$$f(t) = 2(1 - \frac{t}{t_p}) \quad \text{if} \quad \frac{t_p}{2} \leq t < t_p \quad (3.30)$$

There are generally different ways to obtain a solution of the system of differential equations given by (3.15) to (3.17). In case the load-time function $f(t)$ may be idealized by some simple mathematical shape, such as given by (3.29) and (3.30), a closed solution could be obtained by direct integration. In order to take account of the non-linear resistance function (Figure 3.9), the response of the system in, respectively, the elastic, plastic and rebound stage, would be computed making use of the corresponding transformation factors K_i , and setting the initial conditions equal to the final conditions in the preceding stage. This, however, may be cumbersome and the application of numerical integration methods might be more convenient. Thereby, the differential equations of motion are being solved by a time-stepping method (level 2 approaches according to section 3.3.1) where the transformation factors K_i may be introduced without difficulty in the corresponding structural response stage. Frequently employed methods are the constant velocity or the Newmark- β method, both well-described in the literature [124, 164].

The general solution of the SDOF system equations is conveniently represented in so-called response charts [124, 164]. The graphs shown in Figure 3.11 (left: linear scale; right: log-scale), were obtained by means of numerical integration of equations (3.15) to (3.17) under use of the transformation factors K_i given in Table 3.4, valid for simply supported members subjected to uniformly distributed loads. The load was supposed to follow a symmetrical triangular pressure pulse of duration t_p and peak load $\tilde{p}_{peak}$. The assumed resistance function is elastic-perfectly plastic (Figure 3.9).

On the axis of ordinate, the ratio of the maximum (yield) resistance R_y to the peak force F_{peak} ($F_{peak} = \tilde{p}_{peak} \cdot l$) is being represented. Equation (3.31) denotes that both R and F are directly related to their respective bending moments (at mid-span) M_R and M_F , as expressed by (3.32) and (3.33). Ratio R_y/F_{peak} represents hence a dynamic load factor by which means a static equivalent moment can be inferred and applied in the context of conventional design methods in order to provide the required resistance moment M_R [165], according to the level 1 approach (section 3.3.1).

$$\frac{R}{F} = \frac{M_R}{M_F} \quad (3.31)$$

$$M_R = \frac{R \cdot l}{8} \quad (3.32)$$

$$M_F = \frac{F \cdot l}{8} = \frac{\tilde{p} \cdot l^2}{8} \quad (3.33)$$

On the axis of abscissa in Figure 3.11, the ratio of the pressure-pulse duration t_p to the element's natural period T is represented. As stressed in [144], rather than the actual value of either quantity, the non-dimensional time ratio t_p/T is a crucial parameter in structural dynamics. It should be recalled that T is given as a function of the load-mass factor K_{LM} by (3.27). For purpose of the graphical solution (Figure 3.11), a value of $K_{LM} = 0.7$ has been adopted which reflects some intermediate stage between the elastic ($K_{LM} = 0.78$) and plastic ($K_{LM} = 0.66$) response of the member (Table 3.4) [164].

The ratio R_y/F_{peak} is moreover represented as a function of the displacement ductility ratio μ_δ , defined as the ratio of the maximum (mid-span) displacement, δ_{max} , to the yield displacement, δ_y .

$$\mu_\delta = \frac{\delta_{max}}{\delta_y} \quad (3.34)$$

Figure 3.11 Ratio between resistance R_y and force F_{peak} for simply supported beam under uniformly distributed load with triangular forcing function ($t_r = 0.5t_p$), as a function of μ_δ and t_p/T (left: linear scale; right: log-scale).

Figure 3.11 manifests the importance of ductility in blast-load induced structural response. As the ductility ratio μ_δ increases, the required resistance R_y decreases in relation to the applied peak force F_{peak} , what is indicative of the energy dissipation due to plastic structural deformations. From an alternative point of view, it might be stated that the ductility demand increases upon reduction of the available resistance R_y . It may also be inferred from Figure 3.11 that the importance of ductility on the structural response depends significantly on ratio t_p/T . It might be noted for instance, that in the elastic solution ($\mu_\delta = 1$) the required resistance R_y may be more than 1.5 times higher than the applied peak force F_{peak} , in case the load-pulse duration approaches the natural period of the beam ($t_p \approx T$). On the contrary, this value falls significantly below unity as the ductility ratio is being increased. However, as ratio t_p/T increases, the benefit provided by ductility vanishes. Figure 3.11 (right) manifests that ratio R_y/F_{peak} approaches unity as t_p/T adopts very large values. This is indicative of the decreasing importance of inertia forces and energy absorption during the transition from the *dynamic regime* to the *quasi-static regime*. On the contrary, the so-called *impulsive regime* [124, 164] is characterized by load durations t_p significantly shorter than the natural period T . In this case, little spring resistance (R) will develop during time $t \leq t_p$, which will be negligible compared to the applied force (F). The acceleration can then be considered

as constant giving the mass an initial velocity and then leaving the system to decelerate in the free vibration phase [164]. Significant structural response will not develop before this phase is being entered. The required resistance will hence be independent of the load-pulse shape and can be simply deduced from the balance between the initially imparted kinetic energy to the system and the strain energy absorbed by it. Biggs [164] suggested that a dynamic load can be treated as impulsive if its duration t_p is less than about 10 % of the natural period T .

Approach based on energy balance and rigid plastic behaviour

In [30], a simplified modeling approach is included to estimate the required flexural resistance of RC members subjected to the effects of a gas explosion. This approach is based on energy relationships under the assumption of an impulsive loading situation, described above, where the shape of the load-pulse is of no importance for determination of structural behavior. Moreover, for sake of simplicity, structural response is considered as rigid plastic, i.e. displacements δ will not arise before a yield force R_y has been attained (Figure 3.12, left).

Figure 3.12 Idealization of resistance (left) and action (right) according to approach suggested in [30].

The approach implies the assumption of an impulsive force F_{imp} ($> R_y$) which acts on the member during time t_p (Figure 3.12, right) inducing a member displacement δ_y (Figure 3.12, left). The velocity $\dot{\delta}$ initially imparted to the system is given by (3.35), where I denotes the impulse:

$$\dot{\delta} = \frac{I}{m} = (F_{imp} - R_y) \cdot \frac{t_p}{m} \quad (3.35)$$

The kinetic energy E_k associated with the initial velocity can then be formulated as:

$$E_k = 0.5 \cdot m \cdot \dot{\delta}^2 = 0.5 \cdot (F_{imp} - R_y)^2 \cdot \frac{t_p^2}{m} \quad (3.36)$$

On the other hand, the maximum strain energy absorbed by the structural system is defined as:

$$W_s = \int_0^{\delta_y} R(\delta) d\delta = R_y \cdot \delta_y \quad (3.37)$$

At maximum response, the entire kinetic energy has been converted to strain energy and hence $W_s = E_k$. Thus, equating (3.36) and (3.37) and solving for F_{imp} delivers the following expression:

$$F_{imp} = R_y \cdot \left(1 + \sqrt{\frac{2 \cdot m \cdot \delta_y}{R_y \cdot t_p^2}}\right) \quad (3.38)$$

In [30] it is suggested to factor the term included in parenthesis ($= F_{imp}/R_y$) to the available resistance R_y of a given structural element in order to account for the energy dissipation associated with the short

load associated with gas explosions. Alternatively, it might also be reasonable to factor the inverse value of this term ($= R_y/F_{imp}$) to the applied force F_{imp} in order to deduce a static equivalent load effect which may be compared to R_y .

3.3.4 Flexural response under the influence of axial forces

As stated in section 3.2.3, the flexural behavior of RC elements might be significantly influenced by axial forces. In a SDOF analysis, this can be accounted for in a simplified way by means of an equivalent lateral load concept [166] what will be briefly exposed in the following.

Figure 3.13 shows a simply supported member subjected to the combined influence of a uniformly distributed dynamic load $\tilde{p}(t)$ and an eccentric dynamic axial force $N(t)$.

Figure 3.13 Simply supported member under combined influence of blast load $\tilde{p}(t)$ and eccentric axial force $N(t)$.

Due to its eccentric application relative to the centroid of the cross-section, the axial force induces a first order bending moment $N(t) \cdot e$ in the mid-span cross-section of the member. Moreover, a second order term $N(t) \cdot \delta(t)$ will evolve from the member deflection. Hence, the total bending moment $M_N(t)$ attributable to the influence of the axial force $N(t)$ will be:

$$M_N(t) = N(t) \cdot (e + \delta(t)) \quad (3.39)$$

By means of equation (3.40), an equivalent lateral load $\tilde{p}_N(t)$ can then be derived to cause a moment equal to $M_N(t)$ where C is a constant ($C = 8$ for a simply supported member under uniformly distributed load $\tilde{p}(t)$). The equivalent load $\tilde{p}_N(t)$ can then be added to load $\tilde{p}(t)$ at each time step of the SDOF solution procedure.

$$\tilde{p}_N(t) = \frac{M_N(t) \cdot C}{l^2} \quad (3.40)$$

Finally, it should be stressed, that this simplified approach does not consider secondary moments from frame sway, or any other mechanisms that would allow the top of the columns to deflect relative to the bottom. Such kind of behavior must be considered as a part of an analysis of the horizontal load-resisting system of the whole building, rather than within an analysis of individual members [166].

3.3.5 Dynamic reaction forces

In the previous section, it has been shown how the flexural behavior of simple structural members under explosion-induced loads can be analyzed by means of simplified dynamic models, such as SDOF models. However, SDOF models do not directly provide the member's dynamic reaction forces since those forces do not have a direct counterpart in the equivalent SDOF system. In other words, the spring force in the equivalent system (Figure 3.9) is not the same as the real support reaction of the member since this

system was selected so as to have the same deflection as the real element, rather than the same force characteristics [164].

Instead, the dynamic reactions can be derived based on force- or moment equilibrium considerations for the structural members [124, 164]. For that purpose, the distribution of acceleration $\ddot{\delta}(x, t)$, and hence inertia forces $i(x, t)$, is regarded as identical to the deflected shape assumed for the beam, $\phi(x, t)$ [164]. Following this assumption, at any point the intensity of the inertia force is proportional to the ordinate of the deflected shape.

Figure 3.14 Dynamic equilibrium in a simply supported beam [124, 164].

As an example, again a simply supported member under a uniformly distributed blast load $\tilde{p}(t)$ is considered. Taking account of symmetry conditions, moment equilibrium can be formulated for the half-beam model shown in the right part of Figure 3.14. Taking moments about the resultant inertia force $I(t)$, the dynamic reaction forces $V(t)$ may be obtained as a function of the resultant blast load $F(t)$ and beam resistance $R(t)$, which is related to the resisting bending moment $M_R(t)$ by (3.32). For the herein considered example, the dynamic reaction forces are given by [164]:

$$V(t) = 0.39 \cdot R(t) + 0.11 \cdot F(t) \quad \text{if } \delta \leq \delta_y \quad (3.41)$$

$$V(t) = 0.38 \cdot R(t) + 0.12 \cdot F(t) \quad \text{if } \delta > \delta_y \quad (3.42)$$

It should be emphasized that in the static case R equals F and hence, as expected, $V = 0.5 \cdot F$. It might also be stressed that the distinction for, respectively, the elastic (3.41) and the plastic (3.42) range, is due to the different assumed deflected shapes (Figure 3.10). However, from the preceding equations it becomes clear that the resulting difference in $V(t)$ is very small and of few practical interest [164].

Alternatively, the dynamic reactions may be obtained from force equilibrium:

$$V(t) = 0.5 \cdot (F(t) - I(t)) \quad (3.43)$$

This approach, however, requires a previous determination of the resulting inertia force $I(t)$ what can be accomplished by equation (3.44) as a function of the uniformly distributed mass per unit length of the member, $\bar{m}$, and acceleration $\ddot{\delta}(x, t)$. Since, as stated before, the acceleration across the length of the member is supposed to follow the distribution of the deflected shape $\phi(X)$, $\ddot{\delta}(x, t)$ can be expressed as the product of the mid-span acceleration $\ddot{\delta}_{\text{mid-span}}(t)$ and $\phi(X)$, as expressed by (3.45). Substituting (3.45) into (3.44) and performing the integration will finally lead to expression (3.46), where m is the

total mass of the member and K_{IL} is defined as the inertia-load factor [124]. For the herein considered example, K_{IL} is equal to the load factor K_L (Table 3.4).

$$I(t) = \int_0^1 \bar{m} \cdot \ddot{\delta}(x, t) dx \quad (3.44)$$

$$\ddot{\delta}(x, t) = \ddot{\delta}_{\text{mid-span}}(t) \cdot \phi(x) \quad (3.45)$$

$$I(t) = K_{IL} \cdot m \cdot \ddot{\delta}_{\text{mid-span}}(t) \quad (3.46)$$

3.3.6 Applications

The simplified dynamic analysis methods presented in the previous sections have been proven to deliver results which, in general terms, compare quite well to available blast test results on a variety of structural components. This conclusion can be drawn from a number of studies. Some selective studies will be summarized in advance.

In [124], the results from impact tests on a simply supported RC beam [167] are compared to theoretical results from several modeling approaches, among them a SDOF model, a finite element model (realized with the commercial code ABAQUS) and an intermediate modeling approach based on Timoshenko beam equations [153, 168]. The beam was loaded by a localized impact at mid-span, described by the force-time function included in the left part of Figure 3.15 (small figure), characterized by a load duration t_p of about 0.06 s.

The comparison of experimental and theoretical results for the beam's mid-span deflection is shown in the left part of Figure 3.15 as a function of time. The graphs show good agreement between the reported test data and the results from the different modeling approaches. This is true for both the amount of peak deflection and the time to reach peak. Somewhat higher discrepancies can be observed for the peak support reaction and the corresponding time, as shown in the right part of Figure 3.15. The simplified model based on the dynamic equilibrium equations presented in section 3.3.5 (in Figure 3.15 termed SDOF) underestimates the maximum support reaction by around 15%. Moreover, the time to peak is much shorter than predicted by the models, although the overall behaviour is well captured. It is also worth to mention that the test beam was not affected by the brittle direct shear mechanism what could be attributed to the relatively slow rise of the load ($t_r \approx 15$ ms). As reported in section 3.2.3, under relatively long rise times a RC member is unlikely to fail in direct shear.

Figure 3.15 Comparison between test [167] and modeling results for a RC beam under impact force (see small figure on the left): Left: Mid-span deflection; Right: Support reactions; adopted from [124].

Another study designated to the analysis of RC beams under explosion loads was conducted by Magnusson [144]. The tests were performed in a shock tube with a detonating charge (plastic explosive) generating a blast wave. For purpose of the SDOF calculations, the registered load-pulse was idealized by a triangular load-pulse of zero rise time and linear decay (Figure 3.4, right). A bilinear elastic-perfectly plastic resistance function (Figure 3.9) was defined where strain rate effects were factored in by considering a 20% increase of the maximum bending resistance based on the DIF formulation given by (3.12) and strain rate measurements during the tests. The member stiffness k in the elastic range was computed from (3.26) under two different assumptions for cross-sectional stiffness EI : based on the assumption of a completely cracked cross-section (EI_{II}) and based on an equivalent stiffness accounting for tension stiffening ($EI_{eq} = 1/5 \cdot EI_I + 4/5 \cdot EI_{II}$). Depending on this choice, the estimated t_p/T ratios of the RC beams varied between 0.5 and 1.2 approximately, i.e. the members were predicted to perform in the *dynamic regime* (section 3.3.3).

The study concluded that the experimental results may be predicted fairly well with SDOF analysis. Figure 3.16, adapted from [144], shows the ratio of the predicted to experimental maximum displacements δ_m of the beams, $\delta_{m,SDOF}/\delta_{m,Test}$. The results corresponding to the stiffness assumptions based on, respectively, EI_{II} and EI_{eq} , are distinguished. It may be observed that stiffness estimation might have an important influence on the SDOF model prediction, which not necessarily will be conservative, as postulated in several studies. A statistical analysis of the results reveals that ratio $\delta_{m,SDOF}/\delta_{m,Test}$ can be characterized by a mean value of about 0.9 and 1.1 for the estimations based on EI_{II} and EI_{eq} , respectively, and corresponding coefficients of variation (CoV) of about 0.15. When accounting for the reduced sample size, a CoV of approximately 0.2-0.3 would seem to be a reasonable choice for purpose of a statistical characterization of the scatter associated with ratio $\delta_{m,SDOF}/\delta_{m,Test}$.

Figure 3.16 Ratio of maximum deflection according to SDOF analysis and tests (adapted from [144]).

The study conducted by Magnusson [144] showed that, in some cases, the failure mode of the RC beams changed from a flexural failure in comparative static tests, to a flexural shear failure mode in the air-blast tests, as shown in Figure 3.17 for one of the members. Direct shear failures, however, were not observed in the tests.

Figure 3.17 Flexural shear failure after air blast-test of one of the RC beams studied in [144].

As a measure for the shear forces acting close to the member supports, Magnusson [144] determined the reaction forces based on the dynamic equilibrium formulations exposed in section 3.3.5. Figure 3.18 [144] shows a comparison of the calculated and registered reaction forces, which were filtered in order to eliminate high frequency oscillations. It can be observed that the calculated results are in good agreement with the filtered reaction forces, which represent a mean value of the unfiltered values [144]. Based on statements in [169], it was concluded that the simplified models (section 3.3.5), based on deflected shapes associated with static loading conditions, might under certain circumstances be inaccurate for the estimation of the maximum shear forces close to the supports of blast-loaded members. In a recent, theoretical study [148], where a RC beam under blast load exposure was analysed under the assumption of elastic behaviour by means of Bernoulli-Euler beam equations, the author came to a similar conclusion.

Figure 3.18 Comparison of registered and calculated support reactions for RC beam studied in [144].

The influence of high frequency oscillations on the support reactions of blast-loaded beams was analysed as well in the numerical simulations carried out in [170]. Although the study does not include a model validation through comparison with experimental results, and despite the fact that it was addressed to structural steel beams, the obtained results are found worth to mention. Schachter et al. [170] showed that the distribution of inertia forces across the span of the simply supported members was significantly affected by contributions of higher frequency modes, which are not accounted for in the simplified, dynamic equilibrium formulations (section 3.3.5). In particular, it was observed that inertia forces do not always oppose the externally applied forces and, instead, might add load to the system while the external loads are still acting. A comparison between the numerical results obtained for the maximum support reactions and the corresponding values according to dynamic equilibrium formulations (section 3.3.5) showed that the former might significantly exceed the latter. Thereby, higher displacement ductility ratios (μ_δ) led to larger discrepancies between simplified and numerical results. Likewise, smaller t_p/T and R/F ratios were found to increase these discrepancies [170].

An extensive and interesting study on the flexural behaviour of RC beams under blast loading conditions was conducted by Rong and Li [171, 172]. With the intention to quantify the differences in the response

of real elements and equivalent SDOF models, they defined the following non-dimensional indices, where δ_m and μ_δ represent the maximum displacement and the displacement ductility index, respectively, and *eq* and *rc* refer to the equivalent (SDOF) system and the real RC member properties:

$$\zeta_{\delta_m} = \frac{\delta_m^{eq} - \delta_m^{rc}}{\delta_m^{eq}} \quad (3.47)$$

$$\zeta_{\mu_\delta} = \frac{\mu_\delta^{eq} - \mu_\delta^{rc}}{\mu_\delta^{eq}} \quad (3.48)$$

The study comprised the analysis of different member support conditions. For each of these conditions, a representative set of 2000 RC beams was defined by varying member span, material properties and blast-load characteristics within reasonable limits. Strain rate effects were considered according to Table 3.3, by constant DIF factors of 1.19 for concrete in compression and 1.17 for the reinforcing steel in tension. The load was assumed to follow a triangular pressure pulse with zero rise time (Figure 3.4, right) and its characteristics varied in terms of variations of the explosive charge and its standoff distance to the target. Target values were then defined for the support rotation ($\theta_t = \theta^{eq}$), directly related to the maximum displacement ($\delta_{m,t} = \delta_m^{eq}$), and the ductility ratio ($\mu_{\delta,t} = \mu_\delta^{eq}$) of the members, ranging from $\theta_t = 1^\circ$ to 4.8° and $\mu_{\delta,t} = 3.0$ to 16.5 . Based on these target values, and the mentioned, predefined load- and member characteristics, the required reinforcement ratio ρ was then determined (for a particular cross-section geometry) on the grounds of an SDOF analysis. Subsequently, 500 out of the 2000 designed beams for each of the member support configurations were analysed by means of an experimentally validated numerical FE model. The obtained maximum displacements and ductility ratios were considered to represent the corresponding real quantities, δ_m^{rc} and μ_δ^{rc} .

Finally, the displacement index ζ_{δ_m} and displacement ductility index ζ_{μ_δ} were determined according to (3.47) and (3.48) and statistically analyzed. In Figure 3.19, the indices obtained for the 500 simply supported members analysed are plotted as a function of reinforcement ratio ρ . The fact that ζ_{δ_m} is not significantly different from zero is indicative of only small discrepancies between δ_{max}^{rc} and δ_{max}^{eq} . In general, the influence of the reinforcement ratio ρ on ζ_{δ_m} appears to be small. On the contrary, ρ is an important parameter when referring to the displacement ductility index ζ_{μ_δ} . As can be observed, the smaller ρ the higher is ζ_{μ_δ} and hence the larger is μ_δ^{eq} in comparison to μ_δ^{rc} . Since, as stated before, $\delta_{max}^{eq} \approx \delta_{max}^{rc}$, large ζ_{μ_δ} values necessarily imply large differences in the yield deflections, δ_y^{eq} and δ_y^{rc} , respectively. Precisely, the yield deflections predicted by the SDOF method are smaller than the values obtained in the numerical analysis, especially if the reinforcement ratio ρ is small. To explain this finding, Rong and Li [171, 172] relegate to the formulation used for the stiffness estimation in the SDOF analysis, which is function of ρ and which seems to overestimate the member stiffness in case the contribution of the concrete is large in relation to that of the reinforcing steel (i.e., for members with small ρ). Similar conclusions were deduced for RC members with other support conditions.

Figure 3.19 Displacement index (3.47) and ductility index (3.48) for 500 simply supported RC members as a function of reinforcement ratio ρ [171, 172].

Oswald and Bazan [173] present comparisons of maximum deflections based on SDOF analysis with test results for different type of structural components (corrugate steel panels, one-way RC walls, one-way reinforced masonry walls or two-way unreinforced masonry walls). The mean value (μ) and coefficient of variation (CoV) of the ratio between computed- and measured deflections ($\delta_{m,SDOF} / \delta_{m,Test}$) for concrete walls are summarized in Table 3.5. The indicated values are based on 76 tests conducted in the framework of eight different test programs. The mean value of 1.19 indicates that the SDOF method tends to overestimate the dynamic deflections of the RC walls. However, the corresponding CoV of 0.3 evidences that the results are subject to a significant scatter. According to [173], part of this scatter may be attributed to the incomplete information and the associated uncertainties in the SDOF input parameters for both the resistance and the loading characteristics of the test walls.

Table 3.5 Mean value (μ) and coefficient of variation (CoV) of ratio $\delta_{m,SDOF} / \delta_{m,Test}$ for RC walls in 76 tests [173].

Element type	N ^o of test programmes	N ^o of tests	μ	CoV
RC walls	8	76	1.19	0.30

The performance of one-way reinforced concrete panels under blast loads was also analyzed by Bach et al. [174]. The dynamic analysis of the panels was based on SDOF analysis or pressure-impulse diagrams, and the results were validated with tests conducted in shock tubes. The pressure-time function characterizing the blast applied to the panels could be described by a zero rise time followed by exponential decay according to the Friedlander function [165], with a duration of the positive phase varying from 21 to 42 ms. Figure 3.20 (left) shows the elevation of the four simply supported test panels (span of 1.7 m) with rectangular cross-section and reinforcement layout as shown. Moreover, a view of one of the panels after the tests is provided in the Figure, along with a comparison between experimental and test results in terms of the member mid-span deflections for three of the panels. It can be observed that the SDOF results (dotted lines) are generally in very good agreement to the experimentally obtained values (continuous lines) until the time of maximum displacement, t_{max} .

Figure 3.20 Left: Elevation, cross-section layout and view after shock-tube test of RC panels studied in [174]; Right: Comparison of registered and calculated panel mid-span displacements [174].

Figure 3.20 shows again that SDOF analysis does not compulsively leads to conservative predictions. In this context, it is interesting to mention that the deduction of the resistance function laid down in the SDOF analysis, accounted for an enhanced stiffness due to consideration of concrete tension stiffening. As pointed out before, this is indicative of the importance of an appropriate member stiffness estimation. In more general terms, it confirms that the degree of precision in the prediction of the dynamic member behaviour depends mainly on the SDOF input parameters and the associated degree of uncertainty rather than on the methodology itself.

3.4 Summary

Chapter 3 addresses the principal characteristics of gas explosions and their effects on structures.

Section 3.1 is dedicated to the description of such an explosion event and the associated loads on structures. Data on the occurrence probabilities of gas explosions, reported in studies conducted in different countries and periods, shows a good correlation among the findings (section 3.1.2, Table 3.1). Interestingly, the probability of an explosion event per time unit λ seems to be almost independent of the considered time period during which data was collected, what in light of the continuous modernization of gas installations and appliances could not have been expected. The compiled data suggests that $\lambda = 10^{-5}$ provides a fairly good estimate for the annual occurrence of a gas explosion in a gas-supplied housing-unit in western countries. Assuming a uniform distribution in time, this figure can be used to model the occurrence probability $p(\text{EX})$ in a specific time period T_{ref} based on a Poisson process (3.1).

The physical backgrounds governing the pressure generation in a gas explosion are subsequently described in section 3.1.3. It is highlighted that venting panels, like windows, doors or light-weight partition walls, play a major role in that process. Available experimental and numerical studies show that generally two pressure peaks can be distinguished (Figure 3.1, Figure 3.2). The first is mainly characterized by the resistance pressure of the venting panels. The second peak, which can attain far higher values than the first, arises in consequences of turbulent combustion processes during the relief of gas and air through the venting openings and significantly depends on the dimensions of both the venting panels and the compartment where the explosion occurs.

Different deterministic models are described in the literature, which intend to predict the maximum pressure generated in a gas explosion (section 3.1.4). A comparison reveals important differences between these models, indicative of the highly complex physical relationships and dependencies between

the many variables involved in the pressure build-up process. Moreover, some of these variables, such as the composition of the gas cloud, the filling-ratio of the compartment or the location of the point of ignition are not predictable at all by means of deterministic approaches. On the contrary, the lack of data hinders a reliable stochastic description of these variables and hence implies a certain degree of conservatism in most model expressions. Available models for a probabilistic description of the maximum explosion pressure are compared and the significant differences concerning both mean value (Figure 3.3) and scatter, evaluated. Moreover, it is shown how pressure time-relationships associated with gaseous deflagrations can be idealized for purpose of simplified dynamic analysis. A triangular load-pulse reasonably approaches such relationships (Figure 3.4). The associated pressure rise-time and total pulse duration are substantially higher than compared to detonations of explosives.

The behaviour of RC elements under the effects of explosion loads is subject of section 3.2. Such loads induce enhanced material strain rates what is commonly known as strain rate effect (section 3.2.2). Since gas explosions are regarded as a relatively slow type of impact loading, the influence of strain-rate is fairly small compared to high velocity impact or blast scenarios. Nevertheless, even under moderate strain rates, of the order of about 10^{-3} to 10^{-2} s⁻¹, which a gas explosion pressure wave is expected to induce in RC members, some material parameters are sufficiently enhanced to be accounted for in the present study. Among these properties are, for instance, the concrete tensile- and compressive strength (Figure 3.6) or the reinforcing steel yield- and tensile strength (Figure 3.7). No evidence could be found for a significant alteration of the scatter associated with these material properties when subjected to dynamic loading conditions.

Explosion-induced loading on structures is generally associated with high energies and short durations. When exposed to such conditions, structural elements experience accelerations and hence inertia forces will influence their behaviour. Moreover, non-linear structural response gains importance for it will contribute to dissipation of energy. Providing RC elements with a significant energy absorption capacity by insuring a ductile, flexural behaviour is one of the key issues when designing for blast loads (section 3.2.3). Necessarily, such behaviour can only be developed when the dynamic shear capacity exceeds the flexural strength of a member. Several experimental studies reported in the literature show that structures designed to exhibit a ductile flexural behaviour under quasi-static loading conditions might fail in shear when exposed to higher loading rates, what may be mainly associated with the influence of higher vibration modes [148]. However, due to the relatively low rise times and amplitudes of gas explosion-induced pressure waves, the extremely brittle direct- and punching shear failure modes are highly unlikely to occur in consequence of gas explosions. The investigation focusses on the compression- or tension controlled diagonal shear failure mode (Figure 3.8), likewise referred to as flexural shear for in many cases a certain amount of flexure occurs prior to the development of diagonal shear cracks.

A number of approaches are in place for determining internal forces and stresses in structural elements exposed to explosions, ranging from simplified models based on static-equivalent loads to complex, numerical simulations of the blast wave and its dynamic effects on the structure. The election of an appropriate modeling strategy primarily depends on the available information concerning the explosion loading. If the load can be represented by an appropriate pressure-time curve, simplified dynamic calculations using time-stepping schemes can be performed. The background of this simplified approach, employed in the present study (Chapter 5), is provided in section 3.3. Member flexural response can be obtained assuming a single degree of freedom system (sections 3.3.3 and 3.3.4, Figure 3.9), whereas the reaction forces, representative for the shear forces, might be determined from the dynamic equilibrium formulation applied to the members themselves (section 3.3.5, Figure 3.14). Strain rate effects can be accounted for by means of dynamic increase factors (section 3.3.2). These simplified models for dynamic

analysis have been proven to deliver results that compare reasonably well to available blast test results on a variety of structural members, especially regarding their flexural response (section 3.3.6). The degree of precision in the prediction of the dynamic member behaviour depends mainly on the model input parameters and the associated degree of uncertainty rather than on the methodology itself.

Chapter 4 Approach and basic developments

4.1 Assumptions for the inference of acceptable risks

4.1.1 Introduction

The point of departure for the definition of the study is constituted by the main objectives of the thesis formulated in section 1.3: the determination of structure-related risks to persons due to gas explosions in buildings and the subsequent establishment of rational acceptance criteria, which may be employed for the assessment of such risks. Achieving these objectives requires, at first, tackling methodological aspects such as the establishment of a mathematical framework for the quantification of structure-related risks or the definition of the procedural steps that, in a logical and clearly structured manner, should address the tasks identified in section 1.3.

Before introducing both the mathematical framework and the procedure, subject of sections 4.2 and 4.3, respectively, the present section introduces the basic ideas and assumptions regarding the inference of an acceptable risk level. The fundamental postulate is that risks associated with structures designed and built in accordance with current *best practice* principles are considered to be reasonably small and acceptable to the public (section 2.3.3). On the other hand, current *best practice* is reflected in the structural design codes in force, whose correct application entails structures that, by definition, are considered sufficiently safe and reliable. In line with these considerations, the perhaps most logical approach is to define acceptable risk to persons as the inherent risk set out in the mentioned design codes [9, 10, 13]. Acceptable risks are therefore directly linked to the *reliability level implicitly required by these codes*. As mentioned before, structural design is traditionally based on cross-section or member design where the corresponding reliability is ensured at a level which can be regarded as intrinsically acceptable. The coming section 4.1.2 discusses how and under which circumstances such a level can be defined for accidental situations associated with gas explosions. Section 4.1.3 summarizes the most relevant conclusions for the further development of the study.

4.1.2 Implicit reliability level for accidental situations

Hazards due to human errors

Latest since the occurrence of the Ronan Point building collapse (Figure 1.2), gas explosions are objectively known hazards to safety of building structures. However, although objectively known, the hazard potential associated with gas explosions is often not subjectively recognized, or sometimes simply ignored (section 1.1.3). Whether not recognized or ignored, the logical consequence of this practice is that the hazard potential associated with gas explosions in buildings is seldom counteracted by explicit measures, i.e. the structural safety concept for a building is based exclusively on the anticipated normal loading conditions of the structure (disregarding here that other accidental loading situations might be relevant). In practice, this situation, which is reflected by path c of the flow chart shown in Figure 4.1, implies the design of each constitutive member of a structure to resist the action effects that would be expected according to the persistent design situation, as defined in the codes in question. Assuming a strict design, i.e. a member performance that complies exactly with the safety requirements laid down in the code rules ($E_d = R_d$), each member would then be provided with the strictly required design resistance for the persistent situation $R_d = R_{d,req,per}$.

Given the relatively low occurrence probability of gas explosions (section 3.1.2), it is likely that no such accidental incident occurs during the envisaged design working life of a building. In this case, the structural members are expected to resist the (foreseen) action effects due to normal loads with a minimum reliability level $\beta = \beta_{\text{req,PER}}$, i.e. the reliability level implicitly required by the codes associated with the persistent load situation (path c_2 in Figure 4.1). However, in the unlikely case a gas explosion occurs during the design working life of the structure, and influences the action effects on a specific member, its performance and the corresponding reliability level would be uncertain and subject to both the available resistance (provided to resist normal loading conditions only) and the action effects associated with the (unforeseen) accidental situation (path c_1 in Figure 4.1). A possible failure event would have to be traced back to some category of human error (ignorance, negligence, etc.). Human errors, in turn, are not covered by the design rules laid down in current structural codes and standards (section 1.1.1). Consequently, in keeping with the objectives of the study, hazards due to human errors are not accounted for in the present study.

Figure 4.1 Event tree identifying design situations, load scenarios and corresponding reliability level associated with the hazard potential stemming from gas explosions in building structures.

Member design as measure to counteract hazard potential

Drawing attention again to Figure 4.1, in advance it will be considered that the hazard *gas explosion* has been subjectively recognized and taken into account by some kind of counteracting measure. As stated before, design codes such as [17], offer a variety of possible measures to counteract the effects of accidental actions in building structures, based on the general performance requirement, that “*the structure shall be designed and executed in a way that it will not be damaged to an extent disproportionate to the original cause*” (section 1.1.2). Among those measures, one might quote prescriptive design and detailing rules foreseen to enhance structural redundancy and/or robustness. Other strategies are related to the mitigation of consequences such as tolerating local member failure provided alternative load paths

might develop which ensure the overall stability of the structural system. Although it might not be an attractive solution from an economical point of view, design of structural members to withstand the effects of an accidental action constitutes another possible measure for which purpose the codes offer specific design rules (section 2.3.4). In compliance with the persistent design situations, the failure probabilities associated with cross-sections or members designed for the accidental situation, and by extension the corresponding risks, might be referred to as intrinsically acceptable by the current legislation. The inference of requirements for structural safety will be based on this appreciation.

Mandatory design situation and corresponding reliability level

Having defined member design as the principle measure to reduce structure-related risks in relation with the hazard *gas explosion* in buildings, the question arises if in a particular practical situation, the accidental design situation would be indeed mandatory or, if on the contrary, structural design would be governed by normal loading conditions (i.e. the persistent situation). Simply speaking, the latter situation, represented by path b in Figure 4.1, would be at hand if the required resistance (e.g. in terms of the required amount of tensile reinforcement for a particular member cross-section under bending stresses) for a particular persistent load combination would be higher than demanded by the leading accidental load arrangement. Assuming again strict design provisions ($E_d = R_d$), in this case the member would be provided with a design resistance $R_d = R_{d,req,per}$. Conversely, if the accidental design situation would be mandatory (path a), design would demand a design resistance $R_d = R_{d,req,acc}$.

In the latter of the two described design situations (path a), the element is supposed to perform in an adequate way according to the code provisions in case the unlikely, accidental loading scenario due to a gas explosion occurs during the design working life of the member (path a₁). The corresponding provided reliability level would represent the reliability level implicitly required by the codes associated with the explosion load scenario ($\beta_{req,EX}$). However, in the more likely case that the explosion does not occur (path a₂), the provided member resistance will probably largely exceed the action effects due to the normal loads to which the member is subjected during the design working life ($R_{prov} \gg R_{req}$). Hence, the corresponding member reliability level might be expected to largely exceed the value demanded by the codes for the abnormal loading conditions that prevailed in the member design ($\beta_{req,PER} \gg \beta_{req,EX}$). Equivalently, it can be assumed that under these circumstances the failure probability of the member, and hence the corresponding risk to persons, would be insignificantly small.

In case the mandatory design situation would have been governed by normal loading conditions ($R_d = R_{d,req,per}$), as represented by path b in Figure 4.1, structural performance of the strictly designed element is associated with a minimum reliability level $\beta_{req,PER}$ if indeed no explosion occurs (path b₂), i.e. the reliability level implicitly required by the codes associated with the persistent load scenario. Contrarily, in case of an unlikely explosion scenario, the member might be expected to perform “on the safe side” since this scenario was taken into account and judged to require a smaller resistance than the persistent loading scenario under normal building use conditions. Hence, as indicated in Figure 4.1 (path b₁), the member reliability level associated with the explosion load scenario would be expected to exceed the required value for the persistent situation ($\beta_{req,EX} > \beta_{req,PER}$). However, this hypothesis needs to be validated given that the reliability of members exposed to highly uncertain load effects stemming from gas explosions is, to date, almost unstudied and might be governed by factors that do not allow for a straightforward conclusion regarding its magnitude. In any case, it seems evident that the failure probability of such members cannot be judged (a priori) to be insignificantly small, nor can be presupposed that the corresponding risk to persons is negligible.

It should be stated that in many practical situations the distinction between persistent and accidental situation as discussed is not relevant. Examples are structural elements that are situated outside the potential influence zone of a gas explosion (e.g. sufficiently far away from the potential explosion source) or the design for a specific member failure mode which might only be triggered by one out of the two situations. For these cases, the definition of the implicitly required reliability level is straight forward.

4.1.3 Final remarks

The following conclusions should be emphasized and kept in mind for the inference of risk-based requirements for structural safety associated with gas explosions in buildings:

- The acceptable risk level is related to the member reliability level implicitly required by the design codes which is a priori unknown.
- Risks due to human errors (ignorance, negligence, etc.) are not covered by the design rules laid down in the codes and, consequently, will not be accounted for in the study.
- Depending on the particular circumstances of each case, member design might be governed by the persistent or the accidental design situation, respectively.
- If the accidental situation is mandatory for design, but the unlikely explosion event does not occur during the design working life of the structure, the member reliability, in this case characterized by normal loading conditions, might be assumed to largely exceed the level implicitly required by the design codes associated with the explosion load scenario (Figure 4.1, path a_2).
- If the persistent situation is mandatory for design, but the unlikely explosion does occur during the design working life of the structure, it may not be stated a priori that the member reliability, in this case characterized by the abnormal, highly uncertain loading conditions, would largely exceed the level implicitly required by the design codes associated with the persistent load scenario (Figure 4.1, path b_1). Further investigation into this issue is required.

4.2 Mathematical model for quantitative risk analysis

4.2.1 Introduction

In the previous section, the basic assumptions for the establishment of acceptable levels of structure-related risks were established. The present section addresses the definition of a mathematical framework for the quantification of such risks. The developments, which are in line with the main principles established in prior studies dedicated to risks in buildings under normal use conditions [9, 13], are based on the current state of knowledge regarding explicit approaches for the analysis, assessment and management of risks associated with technical systems, outlined in sections 2.1 and 2.2. The following sub-section establishes the basic equations for quantification of risks associated with such systems. Subsequently, section 4.2.3 introduces building structure-related specifications.

4.2.2 General framework

In keeping with the considerations made in section 2.1.3, the probability that a person i will lose its live in a specific hazard scenario j associated with a given technical system is defined as the individual risk (r_{ij}). Formally, r_{ij} can be expressed by (4.1) as a function of the occurrence probability of the hazard scenario p_j and the lethality rate for this person given the scenario, l_{ij} .

$$r_{ij} = p_j \cdot l_{ij} \quad (4.1)$$

Where:

- l_{ij} : Lethality rate for person i due to hazard scenario j
 p_j : Occurrence probability of hazard scenario j
 r_{ij} : Individual risk to person i associated with hazard scenario j

In general, different hazard scenarios (j) associated with a particular technical system can contribute to the life safety risk of a particular person. In case of statistical independency between the n_j hazard scenarios which might be relevant, and considering small values for the contributions r_{ij} to the overall individual risk r_i to person i , the following relationship holds:

$$r_i = \sum_{j=1}^{n_j} r_{ij} = \sum_{j=1}^{n_j} p_j \cdot l_{ij} \quad (4.2)$$

Where:

- r_i : Individual risk to person i

Considering that n_i persons might be exposed to a particular hazard scenario j associated with a given technical system, the societal risk (section 2.1.3) to these persons, R_j , is defined as the sum over the individual risks r_{ij} (4.1) corresponding to each of the n_i persons exposed:

$$R_j = \sum_{i=1}^{n_i} r_{ij} = p_j \cdot \sum_{i=1}^{n_i} l_{ij} \quad (4.3)$$

Where:

- R_j : Societal risk to persons associated with hazard scenario j

Finally, if it is again assumed that the n_j relevant hazard scenarios which might occur within a given technical system are statistically independent of each other, the societal risk to persons associated with the entire system is defined as the sum over the risks R_j (4.3) corresponding to each of the n_j hazard scenarios:

$$R = \sum_{j=1}^{n_j} R_j = \sum_{j=1}^{n_j} (p_j \cdot \sum_{i=1}^{n_i} l_{ij}) \quad (4.4)$$

Where:

- R : Societal risk to persons associated with the considered technical system.

4.2.3 Application to building structures

Probabilities

The preceding section describes a mathematical framework for quantification of risks to persons associated with specific hazard scenarios (j) within a particular technical system. In the present context, the technical system considered is a building structure and the hazard scenario is defined as a particular collapse scenario triggered by failure (f) of one of the main constitutive members with specific structural properties. Such a failure is characterized by a specific failure mode (k) to arise under a particular load arrangement (l) on the member. The corresponding occurrence probability, i.e. the probability for the member to fail in mode k under the influence of load arrangement l , $p_{f,kl}$, or simply p_f , will represent probability p_j in equations (4.1) to (4.4).

$$p_j \equiv p_{f,kl} \equiv p_f \quad (4.5)$$

Where:

$p_{f,kl}$, p_f : Probability of failure of a structural member (in failure mode k under load arrangement l)

The failure probability p_f can be formally expressed by equation (2.23). It can be determined by means of structural reliability methods (section 2.2.3) provided that the uncertainties associated with the basic variables involved in the limit state function associated with the considered failure mode (k) and load arrangement (l) of the member can be properly quantified (section 2.2.1). The relation between failure probability p_f and the reliability index β is established by (2.31).

For load arrangements associated with accidental situations, such as internal gas explosions, the occurrence rate of the accidental action itself must be factored into p_f , as expressed by equation (1.2). In line with the definition of the occurrence rate of a gas explosion, $p(\text{EX})$, in section 3.1.2, the failure probability p_f of a structural member is defined as:

$$p_f = p(\text{EX}) \cdot p_{f,kl|\text{EX}} \equiv p(\text{EX}) \cdot p_{f|\text{EX}} \quad (4.6)$$

Where:

$p(\text{EX})$: Occurrence rate of a gas explosion in a gas-supplied housing unit

$p_{f,kl|\text{EX}}$, $p_{f|\text{EX}}$: Probability of failure of a member (in mode k) provided that a gas explosion has occurred and influences the loading conditions (l) of the member.

In general, a specific structural member might fail under different circumstances. In the present context, where members in building structures are addressed, different load arrangements (l) due to normal (persistent) and/or abnormal (accidental) loading conditions must be distinguished (section 4.1.2). Moreover, it must be considered that, depending on the loading particularities a member might be subjected to, different failure modes (k) might trigger its collapse. In summary, each structural member is exposed to several potential failure (f) or hazard scenarios (j). Assuming statistical independency between these scenarios, the accumulated failure probability of a structural member, P_f , can be expressed as the sum over the n_j individual failure probabilities associated with each of the n_l load arrangements (l) and n_k failure modes (k) the member might be subjected to.

$$P_f = \sum_{j=1}^{n_j} p_j = \sum_{l=1}^{n_l} \sum_{k=1}^{n_k} p_{f,kl} \quad (4.7)$$

It should be noted that equation (4.7) is based on the assumption that a structural member can be idealized by a series system constituted by n_j system components (section 2.4.2), corresponding to the n_j potential failure scenarios (j) of the member. Since in a series system the failure of a single system component implies necessarily the failure of the whole system it is implicitly assumed here that the occurrence of each of the n_j failure scenarios will necessarily induce member collapse.

Following the exposed assumptions concerning series system, and taking account of the fact that a building structure is comprised by several constitutive members, the use of equation (4.7) can be extrapolated from the member to the system (structure) level. In that case, the series system components correspond to the n_j potential failure scenarios associated with the totality of the structure-comprising members. It hence appears that the risk associated with a specific structure increases with the number of potential failure scenarios, i.e. with the number of constitutive members, load arrangements and member failure modes, and consequently, is affected by an important *scale effect* [9].

Consequences

Following the mathematical framework defined by equations (4.1) to (4.4), the quantification of building structure-related risks would require suitable estimations of the lethality rate l_{ij} for a person i given a particular hazard scenario j . The probability for a specific person to lose its life in a particular structural collapse scenario depends on multiple influences, such as:

- The *nature of the collapse scenario* which, in turn, will depend on manifold aspects such as the structural typology, dimensions, materials, connections and details, the type and intensity of the triggering event, etc.
- The *building occupancy conditions* such as the density and the distribution of persons present on the area affected by the collapse scenario, etc.
- The *functional building characteristics*, like the availability and arrangement of emergency exits, etc.

For the time being no models are available for the estimation of lethality rate l_{ij} taking account of the mentioned influences. Simplified approaches are hence necessary for consequence modeling within the context of the present study. Instead of modeling l_{ij} , it seems more feasible to realize an estimation of the number of fatalities N_j due to a particular hazard scenario j , which formally corresponds to the sum over the (unknown) lethality rates l_{ij} , corresponding to each of the n_i exposed persons to scenario j :

$$N_j = \sum_{i=1}^{n_i} l_{ij} \quad (4.8)$$

Hence, substituting (4.8) into (4.3) and (4.4), the societal risks to persons associated with a particular collapse scenario on one hand, and the totality of the potential collapse scenarios in the entire structure, on the other, can be readily quantified by means of equations (4.9) and (4.10), respectively.

$$R_j = p_j \cdot N_j \quad (4.9)$$

$$R = \sum_{j=1}^{n_j} p_j \cdot N_j \quad (4.10)$$

For the estimation of the individual risks it will be assumed, for sake of simplicity, that the lethality rate l_{ij} is equal for each of the n_i exposed persons to a particular hazard scenario j . Consequently, equation (4.8) simplifies into (4.11):

$$N_j = n_i \cdot l_{ij} \quad (4.11)$$

It should be noted that since l_{ij} has been assumed as a constant, the individual risk r_{ij} (4.1) and r_i (4.2) to each of the n_i exposed persons appear to be constant as well. As expressed by (2.13), individual and societal risks, are then simply related by the number of exposed persons, n_i . Indeed, solving (4.11) for l_{ij} and substituting it into expressions (4.1) and (4.2), the individual risk r_{ij} and r_i can be readily deduced from expressions (4.12) and (4.13), respectively, as a function of n_i and the societal risks R_j associated with one or n_j particular hazard scenarios (j):

$$r_{ij} = p_j \cdot \frac{N_j}{n_i} = \frac{R_j}{n_i} \quad (4.12)$$

$$r_i = \sum_{j=1}^{n_j} r_{ij} = \frac{1}{n_i} \sum_{j=1}^{n_j} R_j \quad (4.13)$$

Probability-consequence plots

The mathematical framework defined in the previous sections is based on the assumption of statistical independency between the n_j hazard scenarios (j) associated with a particular building structure. In line with this assumption, these scenarios can be represented in a probability-consequence diagram, shown schematically in Figure 4.2, also termed *risk profile* [12].

In such a risk profile, the $j = 1, 2, \dots, n_j$ hazard (collapse) scenarios are numbered and arranged according to the magnitude of the consequences associated with these scenarios, in terms of the number of fatalities, N_j , represented on the axis of abscissa. The axis of ordinates represents the occurrence probabilities of the scenarios p_j , in the present context represented by the probability p_f (4.5) for a structural member to fail within a specific reference period, in a particular failure mode (k) when exposed to a specific load arrangement (l). The maximum value on the ordinate represents the accumulated failure probability P_f associated with the entire structural system (4.7), i.e. the probability that the structure will be affected by a collapse scenario of one of its constitutive members within a specific reference period.

The area under the risk profile for a given structure corresponds to the societal risks to persons, R , associated with the structure, according to (4.10). It should be noted that these risk profiles are closely related to the frequently employed $F(n)$ diagrams introduced in section 2.1.3 (Figure 2.5). This relationship constitutes a fundamental piece for the establishment of acceptance criteria for societal risks in the present study.

Figure 4.2 Schematic representation of a probability-consequence diagram (risk profile)

4.3 Procedure

4.3.1 Introduction

Having established the basic assumptions for the inference of requirements for the assessment of structure-related risks due to gas explosion in buildings (section 4.1), and once defined the mathematical framework for the quantification of such risks (section 4.2), the present section identifies the specific tasks to be addressed in the present study in order to achieve the defined objectives. Based on the generic description of a risk-informed decision analysis in section 2.1.2, a procedure comprising eight main steps has been defined. Following its description in section 4.3.2, section 4.3.3 offers a brief summary of this procedure including indications on the subsequent sections of the present thesis where the corresponding tasks will be addressed.

4.3.2 Description

The procedure initially requires the definition of the general *context, scope and objectives of the study* (*step 1*), what has been basically addressed in Chapter 1 of the present thesis. The necessary mathematical framework for the estimation of structure-related risks (*step 2*) was previously established in section 4.2.

With the principle aim to cover the vast majority of the cases encountered in practice, *step 3* involves *selecting representative sets of hypothetical but realistic building structures and their constitutive members* choosing the parameters with the greatest effect on design, concerning geometry, loads or material strength.

In *step 4*, a qualitative risk analysis (section 2.1.2) is being carried out with the principal aim to identify the *relevant (accidental) hazard scenarios (j)* associated with the previously defined structures. According to the definition in section 4.2.3, in the present context a hazard scenario describes a particular structural collapse scenario induced by failure (*f*) of a structural member according to a specific failure mode (*k*) induced by a certain load arrangement (*l*). Suitable assumptions to describe the extensions of a system collapse given the member failure are to be adopted.

According to the established mathematical framework (*step 2*), structure-related risks to persons are quantified as a function of both the occurrence probabilities (p_j) of the identified hazard scenarios (*step 3*) and their consequences in terms of loss of human life (N_j). The establishment of p_j and N_j is subject of *steps 5* and *6*, respectively.

According to equation (4.5), the occurrence probability p_j is represented by the failure probability of a particular structural member p_f . Further to the considerations in section 4.1.2, assuring member performance to comply exactly with the safety requirements laid down in a consistent set of design codes, the associated failure probability p_f can be assumed to be acceptable by definition and represents hence the *reliability level implicitly required by the codes* in question. In line with these statements, the establishment of the probabilities p_j (*step 5*) comprises the following four sub-steps:

- The mathematical formulations of the relevant member failure modes (*k*) under a specific load arrangement (*l*), identified in *step 4*, are defined in terms of limit state functions (LSF's) and their comprising basic variables.
- Based on the established LSF's, the selected structural members (*step 3*) are strictly designed ($E_d = R_d$) for the accidental situation in accordance with the specifications of a consistent set of structural codes. Subsequently, it is verified if the design resistance R_d should indeed be provided according to the accidental design assumptions, or, if on the contrary, the persistent situation under normal loading conditions prevails (section 4.1.2). The strict design in accordance with the mandatory design situation provides the members with the required resistance properties according to the code rules employed.
- Since the standardized rules for structural design are not based on any explicitly established probabilistic models for the design variables, the degree of uncertainty associated with these rules is unknown and must be quantified. This step corresponds to the stochastic characterization of the basic variables involved in the previously defined LSF's.

- Moreover, the code rules for the accidental design situation do not explicitly consider dynamic effects (structural inertia, energy dissipation and strain rate effects) which arise as a consequence of the relatively high loading rates gas explosions induce in structural members (section 3.2). To quantify such effects, a specific, deterministic dynamic analysis based on efficient, simplified models is carried out. The findings from this analysis are factored into the LSF's.
- Finally, based on the defined LSF's, a reliability analysis of the members based on the FORM algorithm (section 2.2.3) will be carried out for each of the relevant load arrangements and failure modes identified (*step 4*). Due to the involved complexity, the analysis of the columns under the combined influence of bending moments and axial forces (section 2.5), requires a specific reliability solution algorithm.

The estimation of the consequences N_j due to the considered hazard scenarios calls for suitable *consequence models* what is the subject of *step 6* of the analysis procedure. An empirical model will be deduced from compiled data on explosion-induced collapse events reported in the literature and in the press. In keeping with the general scope of the study, the model should enable a prediction of the number of fatalities as a function of relevant parameters characterizing the magnitude of the damage associated with a particular hazard scenario, in line with the assumptions met under *step 4*.

In *step 7*, the implicitly accepted risks associated with each of the defined building structures (*step 3*) are deduced according to the following sub-steps:

- Establishment of the occurrence probabilities p_j (*step 5*) of each of the identified hazard scenarios (*step 4*).
- Determination of the corresponding failure consequences N_j on the grounds of the model deduced in *step 6*.
- Construction of the implicitly accepted risk profiles (Figure 4.2) as a function of both member failure probabilities p_j and consequences N_j . This should be done under the inclusion of hazard scenarios associated with normal loading conditions (Figure 4.1), considering the results of prior studies [9, 10, 13].
- Integration of risk profiles

The final step (*step 8*) involves a detailed evaluation and interpretation of the results obtained. Based on these results, a proposal regarding the acceptance of structure-related risks to persons associated with gas explosions in RC structures should be made taking account of earlier proposals [9, 10, 13].

4.3.3 Summary

The main steps comprising the procedure described in the previous section are briefly summarized as follows (see also Figure 4.3):

1. Definition of context, scope and objectives of the study (Chapter 1).
2. Establishment of a mathematical framework (section 4.2).
3. Choice of a representative set of building structures and constitutive members (section 4.4).
 - Selection of analysis variables and range of corresponding numerical values.
4. Identification of relevant hazard scenarios (section 4.5).

- Definition of member loading characteristics and corresponding failure modes.
 - Assumptions for extensions of system collapse given a local member failure.
5. Determination of the reliability level implicitly required by the codes.
 - Establishment of LSF's (section 4.6).
 - Strict design of members according to design codes (section 4.7).
 - Characterization of basic variables (section 4.8).
 - Dynamic analysis of members (Chapter 5).
 - Reliability analysis of members (Chapter 6).
 6. Deduction of consequence models (Chapter 7).
 7. Determination of implicitly acceptable risks associated with building structures (Chapter 8.)
 - Establishment of the occurrence probabilities of the defined hazard scenarios.
 - Estimation of the consequences to persons associated with these scenarios.
 - Construction of risk profiles under inclusion of results from prior studies corresponding to buildings under normal use conditions [9, 10, 13].
 - Integration of risk profiles.
 8. Analysis, evaluation and interpretation of the obtained results (Chapter 8)
 - Deduction of acceptance criteria for life safety risks

Figure 4.3 Procedural flow-chart

4.4 Representative set of structures and members

4.4.1 Introduction

In relation with the definition of the technical systems to be analysed in the present study, the scope has been previously limited to ordinary RC building structures, intended for residential use (Figure 1.3). *Step 3* of the established procedure (section 4.3) involves the definition of representative sets of such kind of structures and their constitutive main members. For this purpose the most relevant structural design parameters are identified and varied within practically realistic ranges. The subsequent sections summarize the selection of the parameters and their corresponding nominal values along with the hypothesis on which grounds this selection has been based. The mentioned parameters are related to:

- The geometry of the building structures (section 4.4.2).
- The characteristics of the explosion venting components used in these buildings (section 4.4.3).
- The characteristics of the main constitutive structural members (section 4.4.4).

Finally, section 4.4.5 addresses the combination of the selected parameters in order to define the representative sets of structures and members.

4.4.2 Geometry of structures

The structures are supposed to be provided with a rigid concrete core (containing staircase or lift shaft) in order to guarantee sway stability and overall structural resistance to global horizontal influences, such as wind or seismic actions. Around this core, RC frames are placed which are geometrically described by the following parameters (Figure 4.4):

- Building height H , width B and length L ;
- Building proportions, $H : B : L$
- Number of storeys, n_s (excluding ground floor and roof)
- Storey height, h_s
- Distance between axes of load-bearing columns in longitudinal sense of the building, l ;
- Distance between axes of load-bearing columns in transverse sense of the building, $l/2$;

Figure 4.4 Designation of geometrical building parameters (Left: Elevation; Right: Plan view)

The RC frames are constituted by columns, spaced at a distance l in the longitudinal sense of the structure, and by beams spanning this distance. The beams give support to a secondary floor system (secondary beams, slabs) with a spanning distance $l/2$ between the principal frames in transverse sense of the buildings. It is being assumed that a potential collapse of these secondary floor elements would affect a locally confined zone with negligible consequences for building users in comparison to those attributed to the failure of a main member. For this reason, the contribution of the secondary structure to the risks will not be taken into account in the further development of the study.

The numerical values adopted for the mentioned geometrical parameters are summarized in Table 4.1. It should be noted that the indicated building proportions ($H : B : L$) are approximate only. The number of storeys (excluding ground floor and roof) are varied from $n_s = 0$ to $n_s = 30$. Considering a constant storey height of $h_s = 4$ m, the height of the structures varies between 4 and 124 m. In the plan view, main building dimensions (L and B) between 5 and 300 m are accounted for. The distance between the axes of the load-bearing columns in the longitudinal sense of the building, l , is varied from 5 to 40 m.

Table 4.1: Variation of geometrical parameters for definition of a representative set of RC buildings structures

H : B : L (aprox.)	n_s	H [m]	B [m]	L [m]	l [m]
1 : 1 : 1	1	8	10	10	5; 10
	10	44	40	40	5; 10; 20
	15	64	80	80	10; 20
	20	84	60	60	10; 15
1 : 8 : 8	0	4	20	20	10; 20
	0	4	40	40	5; 10; 20
	3	16	120	120	5; 10; 20; 40
	5	24	200	200	5; 10; 20; 40
	10	44	300	240	10; 15; 30
	10	44	300	300	10; 15; 30
8 : 1 : 1	10	44	5	5	5
	20	84	10	10	5; 10
	30	124	15	15	5; 15
1 : 2 : 2	1	8	20	20	20
	3	16	30	30	5; 15; 30
	5	24	40	40	5; 40
	10	44	60	60	5; 10; 20; 30
2 : 2 : 1	5	24	20	10	5; 10
	10	44	30	15	5; 15
	20	84	60	30	10; 15
2 : 1 : 2	5	24	10	20	5; 10; 20
	10	44	15	30	5; 15; 30
	20	84	30	60	5; 15
1 : 2 : 1	1	8	20	10	5
1 : 4 : 1	1	8	40	10	5; 10
	5	24	80	20	10; 20
	15	64	180	45	5; 15
1 : 1 : 4	1	8	10	40	5; 10
	5	24	20	80	10; 20
	15	64	45	180	5; 15
1 : 4 : 8	0	4	10	20	20
	5	24	100	200	5; 10; 20; 40
	10	44	180	300	10; 15; 30
4 : 8 : 1	10	44	60	10	5; 10
	20	84	120	15	5; 15
	30	124	200	20	10; 20
4 : 1 : 8	10	44	10	60	5; 10; 20
	20	84	15	120	10; 15
	30	124	20	200	10; 20
1 : 8 : 4	10	44	300	120	10; 15; 30

The described geometrical parameter selection accounts for a large variety of practical cases, yet it is noted that buildings of extreme dimensions, such as high-rise buildings are out of scope of the study.

4.4.3 Explosion venting components

In section 3.1.3 it has been exposed that the pressure generation of gas explosions in buildings is strongly influenced by the characteristics of the venting components. It was explained that these non-structural elements are intended to fail at a relatively early stage of the pressure development procedure and thereby to provide a pressure relief. Further, in section 3.1.4 it was summarized that several of the currently available models to predict the maximum pressure generated in internal gas explosions consider the venting effect by two parameters: the static resistance pressure of the venting panels, p_{stat} , and the venting coefficient A_v/V (Figure 3.3). Both parameters will depend on the geometrical- and mechanical properties of the building facades (including windows), floor- and ceiling systems or internal partition walls (including doors).

In order to define a representative set of venting components in residential building structures, p_{stat} will be varied between 2 and 7 kN/m² in order to account for typical failure pressures of windows, doors and light partition walls (Table 3.2), values likewise adopted in Schmidt's study [110]. The venting coefficient A_v/V will be varied from 0.05 to 0.15 m⁻¹ corresponding to the application limits of the current Eurocode load model [17]. According to this model, the here assumed best case scenario (with regard to pressure development), characterized by relatively large venting panels of relatively low failure pressure ($A_v/V = 0.15 \text{ m}^{-1}$ and $p_{\text{stat}} = 2 \text{ kN/m}^2$), will entail a nominal explosion pressure of around $p_d = 6 \text{ kN/m}^2$. In the worst case scenario, characterized by relatively small panels of relatively high failure pressure ($A_v/V = 0.05 \text{ m}^{-1}$ and $p_{\text{stat}} = 7 \text{ kN/m}^2$), the Eurocode model delivers a nominal design pressure of about $p_d = 23 \text{ kN/m}^2$.

The described *best case scenario* could be representative of enclosures with light partition walls (plasterboards, etc.) and/or facades with a relatively high percentage of ordinary glass windows. On the other hand, the assumed *worst-case scenario* would represent situations where windows of relatively high resistance provide venting in enclosures covered with cladding panels capable to sustain the expected explosion pressure (including their anchorage to the main structure), such as concrete panels. Available experimental studies [114, 175] provide evidence that concrete cladding panels may resist explosion pressures significantly higher than the nominal value associated with the here assumed worst-case scenario, $p_d = 23 \text{ kN/m}^2$.

On the other hand, there are manifold incidents in practice which show that cladding panels often do not resist the explosion pressure, in case of which they act as venting components. An example is shown in Figure 4.5, where the masonry façade panels were blown out due to the action of the blast. According to the indication in Table 3.2, such masonry panels may resist static pressures of up to 15 kN/m², approximately, i.e. far larger values than the maximum failure pressure for the venting components adopted herein, $p_{\text{stat}} = 7 \text{ kN/m}^2$. However, it should be taken into account that these situations are characterized at the same time by relatively large venting areas A_v as manifested in Figure 4.5. It can be observed that almost the entire masonry panel, corresponding to the enclosure where the deflagration took place, has collapsed, what in addition to the observable failure of internal partition walls and the floor system, provided significant pressure relief during the explosion event. For such situations it is reasonable to assume venting coefficients A_v/V higher, in many cases probably significantly higher, than 0.1 m⁻¹. According to the Eurocode load model, the described situations ($p_{\text{stat}} > 7 \text{ kN/m}^2$, $A_v/V > 0.1 \text{ m}^{-1}$) are characteristic of explosion pressures exclusively governed by the failure pressure of the venting components, p_{stat} (see peak p_1 according to Figure 3.2). Assuming the before-mentioned value of $p_{\text{stat}} =$

15 kN/m² as an indicative value for ordinary masonry panels, the Eurocode model delivers a nominal explosion pressure of $p_d = 18$ kN/m², which is covered by the herein assumed worst case scenario with an associated nominal value of $p_d = 23$ kN/m².

Figure 4.5 Explosion-induced damage to a residential, RC building with masonry facade panels [176]

Taking into account the preceding considerations, it may be assumed that the parameter selection for the relevant characteristics of the building venting elements, and thereby of the expected explosion loading, covers the majority of potential situations which might exist in practice.

4.4.4 Main member characteristics

Boundary conditions

The RC frames geometrically defined above (section 4.4.2) are constituted by beams and columns. For sake of simplicity and since it is a conservative approach with regard to the reliability level, *statically determinate members* will be considered. As previously stated (section 2.4.3), the failure probabilities of statically determinate members are generally higher than those of statically indeterminate members (with ductile behavior) under identical loading conditions. Taking account of this circumstance and disregarding the fact that the member boundary conditions might influence as well the magnitude of the consequences in case of collapse, it can be assumed that the risks to persons associated with statically determinate structures are generally higher than those related to comparable, statically indeterminate structures. Therefore, the analysis of statically indeterminate structures could lead to an important underestimation of the implicitly acceptable level for structure-related risks. To this extent, the consideration of structures with statically determinate members is justified.

Materials and loads

In order to define a representative set of beams and columns, the material characteristics and loads listed in Table 4.2 are selected and varied within the indicated ranges. A brief description of the adopted parameters and their corresponding nominal values is given below.

The study is limited to members of normal strength concrete with characteristic values for the concrete compressive strength between $f_{ck} = 25$ and 50 N/mm². The widely used reinforcing steel type B500 is being assumed for purpose of the analysis, with a characteristic value for the yield strength of $f_{yk} = 500$

N/mm². Class B steel according to [84] implies a characteristic strain at maximum load of $\varepsilon_{uk} = 5\%$ and a minimum ratio between tensile and yield strength of $k = (f_t/f_y)_k = 1.08$.

Table 4.2: Definiton of member parameters and corresponding nominal values

Parameter	Symbol	Nominal Values	Unity
Concrete compressive strength	f_{ck}	25; 35; 50	N/mm ²
Yield strength of longitudinal reinforcement	f_{yk}	500	N/mm ²
Yield strength of shear reinforcement	f_{ywk}	500	N/mm ²
Reinforcing steel strain at maximum force	ε_{uk}	0.05	-
Ratio of reinforcing steel tensile strength to yield	$k = (f_t/f_y)_k$	1.08	-
Material density (reinforced concrete)	ρ_c	25	kN/m ³
Permanent loads of floor system (excluding self-weight of the principle beams)	$g_{f,k}$	0.5; 5; 8 if $l = 5; 10$ m 4; 9; 13 if $l = 15; 20$ m 6; 10; 13 if $l = 30; 40$ m	kN/m ²
Permanent loads of roof system (excluding self-weight of the principle beams)	$g_{r,k}$	1.0; 5 if $l = 5; 10$ m 1.1; 9 if $l = 15; 20$ m 1.2; 10 if $l = 30; 40$ m	kN/m ²
Imposed loads on building floors (Cat A [82]: Domestic/ Resident. Areas)	$q_{i,k}$	2	kN/m ²
Reduction factor for imposed loads on beams	α_A	$\frac{5 \cdot \psi_0}{7} + \frac{A_0}{A_l} \leq 1.0$	-
Reduction factor for imposed loads on columns	α_n	$\frac{2 + (n - 2) \cdot \psi_0}{n}$	-
Snow loads on roof	$q_{s,k}$	1.5; 2; 2.5; 3	kN/m ²
Static activation pressure of explosion vents (section 4.4.3)	p_{stat}	2; 5; 7	kN/m ²
Venting coefficient (section 4.4.3)	A_v/V	0.05; 0.10; 0.15	m ⁻¹
Explosion pressure	p_d	$\max(3 + p_{stat}; 3 + 0.5p_{stat} + \frac{0.04}{(\frac{A_v}{V})^2})$	kN/m ²

Note: The values represented in *italic* are taken into account exclusively for determination of the *member reliability level required by the codes* (step 5 of the analysis procedure) whereas the non-italic values are considered *as well* for the establishment of the *implicitly accepted risks* (step 7) associated with the building structures geometrically defined in Table 4.1 and the subsequent deduction of *risk-based acceptance criteria* (step 8). This distinction is introduced in order to assure that the total number of building structures analyzed remains within reasonable limits.

For determination of the structural member's self-weight and dynamic mass, a material density of $\rho_c = 25$ kN/m³ is being accounted for. Permanent loads of floor- or roof system are varied as a function of the beam span l (section 4.4.2), distinguishing three categories: small spans ($l = 5, 10$ m), medium spans ($l = 15, 20$ m) and large spans ($l = 30, 40$ m). This hypothesis is being adopted since the type, and hence the self-weight of the secondary elements constituting the floor or roof system, usually depends on the distance these elements span, which, as assumed herein, is frequently defined as a function of the principle beam span (l). Three different characteristic values for permanent actions of the floor system ($g_{f,k}$) are considered for each of the three mentioned beam span categories: a minimum value representing light floor systems, a maximum value, accounting for heavy systems and an intermediate value for standard solutions. The self-weight of the non-structural components comprising the roof-system ($g_{r,k}$) is being accounted for with two different values representing, respectively, a light- and a heavy solution.

Based on the assumption of residential building use, a characteristic value of $q_{i,k} = 2$ kN/m² is considered for uniformly distributed, imposed loads in buildings [82]. For the definition of the beams, this load is

reduced by factor α_A (Table 4.2) in function of the loaded area A_l and the combination value ψ_0 for variable actions according to [11]. Complementary, a reduction factor α_n (Table 4.2) is being applied to $q_{i,k}$ if the imposed loads (from the same category) acting on a column stem from $n > 2$ storeys situated above this member and have not been reduced by ψ factors [82].

In addition to the imposed loads, snow loads on the building's roof are considered. Four different load categories are distinguished taking account of different climatic zones in Spain. In line with previous studies [9, 78], the corresponding characteristic values vary from $q_{s,k} = 1.5$ to 3 kN/m^2 .

Finally, the magnitude of the nominal explosion load p_d is defined by the Eurocode model (Figure 3.3), and parameters p_{stat} and A_v/V characterising the venting components of the buildings, according to the indications in section 4.4.3.

4.4.5 Combination of parameters

The combination of the material- and loading parameters defined in Table 4.2 leads to a representative set of 486 RC floor beams with varying span lengths l from 5 to 40 m (section 4.4.2). Moreover, the combination of the values given in Table 4.2 yields a representative set of 28512 RC columns, characterized by a constant member height ($h_c = h_s = 4 \text{ m}$) and a varying number of $n_s = 0$ to 30 storeys situated above the member in question (section 4.4.2).

The representative sets of structural members defined as described are employed to determine the reliability level implicitly required by the codes for the accidental situation *gas explosion*. In addition, out of these sets a number of members is selected (see subscript in Table 4.2), which constitute the building structures geometrically described in Table 4.1, to be employed for the establishment of implicitly accepted risk profiles (Figure 4.2) and the subsequent derivation of risk-based acceptance criteria. Combining the selected member parameters with the geometrical building parameters, a representative set of 4512 building structures is being obtained, constituted by $n_m = 6$ to 40931 RC members and varying building net room areas between $A = 200$ and 990000 m^2 .

4.5 Accidental hazard scenarios

4.5.1 Introduction

The present section addresses, in accordance with *step 4* of the established procedure (section 4.3), the identification of the relevant hazard scenarios to the defined structures. Based on the definition of an accidental scenario characterized by a building-internal gas explosion (section 4.5.2), this step involves identifying the most relevant load arrangements the constitutive structural members defined in section 4.4 might be subjected to and to establish the corresponding potential failure modes (section 4.5.3). Moreover, suitable assumptions have to be met concerning the question under which circumstances, and to which extent, a local member failure might induce a structural system failure (section 4.5.4).

4.5.2 Accidental scenario

The accidental loading scenario considered in the present study is schematically reflected in Figure 4.6. Two subsequent stages can be distinguished:

Previous to the explosion event, the building structures are subjected to gravitational loads of both permanent and variable character, corresponding to the normal use conditions of the buildings (*stage 1*). The gravitational loads are homogeneously distributed on the floors ($g_r + q_i$) and the roof ($g_r + q_s$) of the

structure, from where they are transferred to the principle load bearing elements, i.e. beams and columns. The corresponding tributary width for the beams, $l_{G+Q} = l/2$ and the tributary area for the columns, $A_{G+Q} = l \cdot l/2 = l^2/2$, are defined in accordance with the geometrical assumptions exposed in section 4.4.2 (Figure 4.4).

Figure 4.6 Loading scenario in building structures; Stage 1: Normal use conditions; Stage 2: Explosion

In *stage 2*, a gas explosion occurs in a certain zone of the building. The explosion generates a blast overpressure $p_{EX}(t)$ which will act simultaneously with the gravitational loads identified in *stage 1*. Based on the considerations in section 3.1.4, a symmetrical pressure pulse ($t_r = t_p/2$) is adopted for the pressure-time relationship characterized by a peak-value, $p_{EX,peak}$, as shown in Figure 4.7. The magnitude of $p_{EX,peak}$ will be exclusively described by the venting parameters p_{stat} and A_v/V (section 4.4.3).

Figure 4.7 Explosion pressure - time relationship assumed in the study

In line with [17], pressure $p_{EX}(t)$ is supposed to act homogeneously distributed on the nearest closures (floor, ceiling, walls, façade panels, etc.) of the room or compartment where the explosion event occurs. These closures are aligned with the axes of the load bearing columns in both directions of the building. From the closures, the pressure is transferred to the principle load bearing elements of the structure. Due to the locally confined influence zone of the explosion, it is reasonable to assume for this purpose smaller tributary widths and areas than those adopted for the gravitational loads (*stage 1*). For sake of simplicity, the tributary width for the explosion pressure transfer from the ceiling- or floor systems to the beams below or above, l_{EX} , will be defined as $l_{EX} = l_{G+Q}/2 = l/4$. Accordingly, the tributary area to

determine the explosion-induced axial force in the columns will be defined as $A_{EX} = l/2 \cdot l/4 = l^2/8$. Moreover, it is assumed that a horizontal blast load component $\tilde{p}_{EX,h}$ will be transferred from the vertical closures to the columns situated in the alignments of these closures under an tributary width $l_{EX} = l/2$.

4.5.3 Load arrangements and failure modes of members

Based on the considerations in the preceding section, in the following the most relevant *load arrangements* and *failure modes* of the principle structural members will be identified. From section 3.2.3 it can be concluded that the relevant failure modes for standard RC elements like beams and columns under the influence of gas explosions in building structures are essentially the same which have to be addressed under persistent, quasi-static loading conditions. In particular, bending- and diagonal shear failure must be considered, taking account of interactions with axial forces in case of the columns. In harmony with the assumption of static determinate members, the analysis of bending-shear interaction will be obviated in the present study.

Beams

For the beams, the following load arrangements are distinguished (see Figure 4.6 and Figure 4.8):

- *Load arrangement $p_{EX(+)}$* : The beams supporting the bottom floor system of the enclosure where the explosion occurs are subjected to a blast load component $\tilde{p}_{EX}(t)$ (stage 2) which adds to the previously present gravitational loads, $\tilde{g} + \tilde{q}_i$ (stage 1).
- *Load arrangement $p_{EX(-)}$* : The beams supporting the ceiling of the enclosure where the explosion occurs are subjected to a blast load component $\tilde{p}_{EX}(t)$ (stage 2) which counteracts the previously present gravitational loads (stage 1). Since in this case the contribution of the latter to the resulting action effects is favourable, only permanent loads $\tilde{g}$ will be considered ($\tilde{q}=0$).

Figure 4.8 Load arrangements $p_{EX(+)}$ (upper) and $p_{EX(-)}$ (lower) for beams; Stage 1: Normal use conditions, stage 2: Explosion

The first of the two described load arrangements, $p_{EX(+)}$ is considered relevant for failure due to excessive, positive bending ($M_{(+)}$) at mid-span of the beams. Moreover, situation $p_{EX(+)}$ is critical for shear failure close to the support cross-section of the beams, where failure of the compression strut (V_c) should be distinguished from failure of the tension tie (V_s). In addition to $p_{EX(+)}$, the consideration of load arrangement $p_{EX(-)}$ is only justified in case the static equivalent action effects induced by the explosion loading $\tilde{p}_{EX}$ equal at least those stemming from the gravitational loads $\tilde{g}$. Following this assumption, a potential negative bending failure ($M_{(-)}$) at the mid-span cross section of the beams will be investigated.

In turn, a shear failure of the beams due to load arrangement $p_{EX(-)}$ is considered to be comparatively unlikely and is not being accounted for. As stated before, such a scenario should be analysed under the influence of load arrangement $p_{EX(+)}$, where the explosion load is added to the gravitational loads.

Columns

The most relevant load arrangement in view of the reliability analysis of the columns is highlighted in Figure 4.6 and Figure 4.9 as $N_{EX(-)}$: The columns situated in the alignments of the vertical closures confining the compartment where the explosion occurs are subjected to a blast-induced tensile axial force component $N_{EX}(t)$ (stage 2) which counteracts the previously acting axial compression force due to gravitational loads, N_{G+Q} (stage 1). Moreover, the explosion will induce a horizontal blast load component $\tilde{p}_{EX}(t)$ on the columns.

Under the described influences, the mid-span cross section of the columns might be subject to a potential bending failure under the presence of axial forces (M-N). The bending moment (M) stems from the horizontal, explosion-induced component $\tilde{p}_{EX,h}(t)$ and from the eccentrically acting axial force in the member (including second order effects). For sake of simplicity, the eccentricity e_{min} (Figure 4.8), which accounts for geometric imperfections, will be applied to both axial force components, due to, respectively, the gravitational loads N_{G+Q} and the explosion load $N_{EX}(t)$. In order to account for the most unfavourable combination of M and N, two axial force combinations are established, leading to, respectively, a minimum value (N_{min}), which considers exclusively permanent loads, and a maximum value (N_{max}).

In addition to the described bending failure scenario, load arrangement $N_{EX(-)}$ might induce shear failure close to the support cross sections of the columns. Again the influence of axial forces on the failure mechanism should be accounted for. Thereby, a distinction should be drawn for failure of the compression strut (V_c-N) and the tension tie (V_s-N), respectively. For the former, the maximum axial compression force level (N_{max}) should be accounted for, while the assessment of the latter should be conservatively based on the minimum value (N_{min}).

Figure 4.9 Load arrangements $N_{EX(+)}$ (left) and $N_{EX(-)}$ (right) on columns; Stage 1: Normal use conditions; Stage 2: Explosion

The columns situated below and above the alignments of the vertical closures confining the compartment where the explosion occurs are subjected to an axial force component $N_{EX}(t)$ (stage 2) which enhances the compression force level in the member, initially characterized by the gravitational loads, N_{G+Q} (stage 1). Assuming again an initial eccentricity e_{min} , this load arrangement, termed $N_{EX(+)}$

(Figure 4.6 and Figure 4.9), will induce bending moment (including second order term)- axial force interaction at mid-span of the members. However, the occurrence of a corresponding failure scenario is considered to be comparatively unlikely and will not be accounted for in the present study. This appreciation is supported by a performed design check according to which load arrangement $N_{EX(+)}$ is not found to be mandatory for any of the studied columns. The mandatory design situation is governed either by the persistent load scenario (only gravitational loads), or by the before-introduced accidental load arrangement $N_{EX(-)}$ (see also section 4.7.3).

Finally, it should be stressed that the study does neither account for biaxial bending of the columns, nor will bending-shear interaction at the member support be considered.

Summary

The most relevant hazard scenarios to be considered in the present study are:

For the *beams*:

- Positive bending failure $M_{(+)}$ at the mid-span cross-section due to load arrangement $p_{EX(+)}$
- Negative bending failure $M_{(-)}$ at the mid-span cross-section due to load arrangement $p_{EX(-)}$
- Shear failure V_c or V_s close to the support cross-section due to load arrangement $p_{EX(+)}$

For the *columns*:

- Failure at the mid-span cross-section induced by bending-axial force interaction $M-N$ due to load arrangement $N_{EX(-)}$
- Failure close to the support cross-section induced by shear-axial force interaction, V_c-N or V_s-N , due to load arrangement $N_{EX(-)}$

4.5.4 Member failure-induced system collapse

The foregoing section focuses on member failure scenarios. In addition, a number of simplifying assumptions are adopted in the present study to account for the possibility of a *partial or total system collapse scenario induced by member failure*:

In line with the assumption of statically determinate structural members (section 4.4.4), a *beam failure* is supposed to induce immediate loss of equilibrium of the secondary floor system (secondary beams or slabs) to which it provides support. In addition, due to falling debris, the beam collapse will affect the area of the floor situated immediately below, without inducing its collapse, however.

The assumption of statically determinate members implies that the *loss of a specific column* would give rise to the immediate collapse of the columns corresponding to the same alignment in the floors above, and necessarily of the floor systems supported by these. On the other hand, as assumed in case of beam failure, a column failure will not entail a progressive collapse of the floors below, due to, for instance, impact of debris and/or uncontrolled redistribution of loads, as could be observed in the collapsed buildings shown in Figure 1.2.

Moreover, the lack of structural continuity associated with the assumption of statically determinate members implies that a member collapse will not entrain and drag down structural elements situated in adjacent frames, as reported in explosion-induced structural collapse scenarios of continuous structural systems [177, 178].

4.6 Limit state functions

4.6.1 Introduction

Subsequent to their qualitative description, in the present section the hazard scenarios are represented in terms of limit state functions (LSF's) according to *step 5* of the established procedure (section 4.3). With a few exceptions, these LSF's are based on the codes employed for member design, i.e. the Spanish building codes (CTE) for basis of design [179] and actions [180], as well as standard EHE [83] for the resistance of RC structures (see also section 4.7.1). To a large extent, these codes are compatible with the respective Eurocodes [11, 17, 82, 84].

The mentioned exceptions are introduced in order to consider in an explicit manner certain aspects, which in the mentioned code rules, if at all accounted for, are being addressed in an implicit way. This concerns particularly the dynamic effects (inertia forces, energy dissipation and strain rate effects) due to the relatively high loading rates gas explosions might entail (section 3.2). The defined LSF's accommodate such effects. The following section 4.6.2 describes the functions established for the beams. Subsequently, section 4.6.3 addresses the developments for the columns.

4.6.2 Beams

Bending failure at mid-span ($M_{(+)}$ or $M_{(-)}$)

The most generic formulation of the limit state corresponding to bending failure at the mid-span cross-section of the beams is given by equation (4.14), which expresses equilibrium between the applied and the resisting bending moment, respectively, M_E and M_R .

$$M_R - M_E = 0 \quad (4.14)$$

The action effect M_E (4.15) can be divided into a static ($M_{E,stat}$) and a dynamic ($M_{E,dyn}$) contribution, respectively, due to the gravitational- and the explosive loading on the members. The latter takes account of inertia forces and energy dissipation during the structural response to the dynamic load.

$$M_E = M_{E,stat} + M_{E,dyn} = \xi_{M_{E,stat}} \cdot (\mp M_G + \psi_1 \cdot M_Q) + \xi_{M_{E,dyn}} \cdot M_{EX} \quad (4.15)$$

Where:

$\xi_{M_{E,stat}}$:	Model uncertainty coefficient for determining bending moments due to static actions
M_G :	Bending moment at mid-span due to permanent actions (including member self-weight $\tilde{g}_c$ and permanent loads of the floor system, $\tilde{g}_f$, Table 4.2)
M_Q :	Bending moment at mid-span due to variable actions (imposed loads $\tilde{q}_i$ including reduction factor α_A , Table 4.2)
ψ_1 :	Factor for frequent value of a variable action (section 2.3.4)
$\xi_{M_{E,dyn}}$:	Model uncertainty coefficient for determining bending moments due to dynamic actions
M_{EX} :	Bending moment at mid-span due to explosion load $\tilde{p}_{EX}$

It should be remarked that equations (4.14) to (4.17) are valid for both positive ($M_{(+)}$) and negative ($M_{(-)}$) bending failure of the beams (section 4.5.3). In case of the former, the moments due to gravitational loads ($M_G + M_Q$) are added to the explosion-induced moment M_{EX} , whereas for the latter moment

M_G is rested and $M_Q=0$ (Figure 4.8). Coefficient ψ_1 [11] is introduced in the LSF as a simplification to account for combination of variable actions in accidental situations.

The resisting moment M_R (4.16) is function of force F_{s1} in the tensile reinforcing steel layer A_{s1} (Figure 4.10), defined under assumption of a bi-linear stress-strain law (4.17). A contribution of the compression reinforcement (A_{s2}) is not considered in case of the beams. Parameters k_c and a_c define the parabola-rectangle law adopted for concrete under compression [84]. The definition of the material parameters of both steel and concrete take account of the dynamic stress rates induced by gas explosions.

$$M_R = \xi_{M_R} \cdot (F_{s1} \cdot d - \frac{k_c}{a_c} \cdot \frac{F_{s1}^2}{b \cdot \alpha_{cc} \cdot f_{c,dyn}}) \quad (4.16)$$

$$F_{s1} = (f_{y,dyn} + \frac{(\varepsilon_{s1} - \varepsilon_{y,dyn})}{(\varepsilon_{u,dyn} - \varepsilon_{y,dyn})} \cdot (f_{t,dyn} - f_{y,dyn})) \cdot A_{s1} \quad (4.17)$$

Where:

ξ_{M_R} :	Model uncertainty coefficient for determining resisting bending moments
b :	Width of the mid-span cross-section
d :	Effective depth of the mid-span cross-section
α_{cc} :	Coefficient taking account of long-term effects on the concrete compressive strength
$f_{c,dyn}$:	Concrete compressive strength under dynamic loading conditions
a_c :	Integration coefficient for determination of the resulting concrete compressive force F_c under assumption of a parabola-rectangle diagram
k_c :	Coefficient for determination of the depth of the resulting concrete compressive force F_c under assumption of a parabola-rectangle diagram
F_{s1} :	Force resisted by the tensile reinforcement layer
A_{s1} :	Cross-sectional area of the tensile reinforcement layer
$f_{y,dyn}$:	Yield strength of reinforcing steel under dynamic loading conditions
$f_{t,dyn}$:	Tensile strength of reinforcing steel under dynamic loading conditions
$\varepsilon_{y,dyn}$:	Yield strain of reinforcing steel under dynamic loading conditions
$\varepsilon_{u,dyn}$:	Strain at maximum force of reinforcing steel under dynamic loading conditions

 Figure 4.10 Rectangular RC cross-section, strain (ε) and stress (σ) distribution and internal forces

Shear failure at support (V_c or V_s)

The most generic formulation of the limit state corresponding to shear failure of the beams is given by equation (4.18), which expresses equilibrium between the applied- and the resisting shear forces, respectively, V_E and V_R .

$$V_R - V_E = 0 \quad (4.18)$$

The action effect V_E (4.19) can be divided into a static ($V_{E,stat}$) and a dynamic ($V_{E,dyn}$) contribution, respectively, due to the gravitational and the explosive loading on the members. The latter should consider the fact that dynamic shear forces induced by blast loads might attain significantly higher values than under static loading conditions (section 3.2).

$$V_E = V_{E,stat} + V_{E,dyn} = \xi_{V_{E,stat}} \cdot (V_G + \psi_1 \cdot V_Q) + \xi_{V_{E,dyn}} \cdot V_{EX} \quad (4.19)$$

Where:

- $\xi_{V_{E,stat}}$: Model uncertainty coefficient for determining shear forces due to static actions
- V_G : Shear force at the support cross-section due to permanent actions (including member self-weight $\tilde{g}_c$ and permanent loads of the floor system, $\tilde{g}_f$, Table 4.2)
- V_Q : Shear force at the support cross-section due to variable actions (imposed loads $\tilde{q}_i$ including reduction factor α_A , Table 4.2)
- $\xi_{V_{E,dyn}}$: Model uncertainty coefficient for determining shear forces due to dynamic actions
- V_{EX} : Shear force at member support due to explosion load $\tilde{p}_{EX}$

The shear resistance limited by failure of the compression struts ($V_{R,c}$) and the tensions tie ($V_{R,s}$) is defined by means of expressions (4.20) and (4.21), respectively. Although in case of members subject to uniformly distributed loads, as supposed herein, resistance $V_{R,s}$ does not need to be verified at a distance less than the effective depth of the cross-section from the face of the member support [84], the investigations into shear failure in the present study will be generally based on the support cross-section.

It should be noted that expressions (4.20) and (4.21) are based on the assumption of a 90° -angle between the shear reinforcement and the longitudinal beam axis (α) and a 45° -angle between the concrete

compression strut and this axis (θ). Furthermore it must be stressed that the contribution of the concrete to $V_{R,S}$ (by aggregate interlock) is being neglected.

$$V_{R,c} = \xi_{V_{Rc}} \cdot (b_w \cdot d \cdot 0.5 \cdot \eta_c \cdot f_{c,dyn}) \quad (4.20)$$

$$V_{R,s} = \xi_{V_{Rs}} \cdot \left(\frac{A_{sw}}{s} \cdot f_{yw,dyn} \cdot 0.9 \cdot d \right) \quad (4.21)$$

Where:

- $\xi_{V_{Rc}}$: Model uncertainty coefficient for determining the shear resistance limited by crushing of the compression struts
- b_w : Width of support cross-section
- η_c : Reduction factor for concrete cracked in shear ($\eta_c = 0.6$ if $f_{ck} < 60$ N/mm²)
- $\xi_{V_{Rs}}$: Model uncertainty coefficient for determining the shear resistance limited by yielding of the shear reinforcement
- $\frac{A_{sw}}{s}$: Cross-sectional area of shear reinforcement per unit length along the longitudinal beam axis
- $f_{yw,dyn}$: Yield strength of shear reinforcement under dynamic loading conditions.

4.6.3 Columns

Failure due to bending-axial force interaction at mid-span (M-N)

The LSF corresponding to a failure of the intermediate cross-section of the columns is based on the M-N interaction sequence shown in Figure 4.11 (M normalized for cross-section depth h), induced by load arrangement $N_{EX(-)}$ (Figure 4.9). Two sequential M-N paths are distinguished:

- Path n°1 (blue) describes the M-N interaction due to the initially acting gravitational loads. These cause a static axial compression force, $N_{E,stat}$, which, due to its eccentric action with respect to the center of gravity of the cross-section, entails a bending moment $M_{E,stat}$.
- Path n°2 (red) represents the behavior during the explosion event, which introduces a dynamic axial force component $N_{E,dyn}$ reducing $N_{E,stat}$ to a resulting force N_E (4.22). Simultaneously, a dynamic moment $M_{E,dyn}$ evolves from the horizontal explosion load $\check{p}_{EX,h}$ (Figure 4.9), on one hand, and the eccentric action of the axial force on the other, increasing the initial moment $M_{E,stat}$ to a resulting value M_E .

$$N_E = N_{E,stat} - N_{E,dyn} \quad (4.22)$$

$$M_E = M_{E,stat} + M_{E,dyn} \quad (4.23)$$

The limit state corresponding to the described M-N sequence is reached if the point characterizing the resulting action effects (M_E, N_E) falls on the M-N interaction diagram, and hence the safety margin (SM) is reduced to zero (Figure 4.11). Formally, this limit state can be expressed, in its most general terms, by simultaneous fulfilling of the following two equations:

$$N_R - N_E = 0 \quad (4.24)$$

$$M_R - M_E = 0 \quad (4.25)$$

Taking account of the fact that the reliability of RC elements subjected to M-N interaction is load-path dependent (Figure 2.18), a more problem-specific definition of the LSF can be formulated based on direction of path n°2, which describes the structural response during the explosion. Assuming this path to be reasonably well represented by a linear function, the limit state can be expressed as follows:

$$\left[\left(\frac{M_{R,dyn}}{h} \right)^2 + N_{R,dyn}^2 \right]^{0.5} - \left[\left(\frac{M_{E,dyn}}{h} \right)^2 + N_{E,dyn}^2 \right]^{0.5} = 0 \quad (4.26)$$

Where :

- $M_{E,dyn}$: Bending moment due to dynamic actions (explosion loading)
- $N_{E,dyn}$: Axial force due to dynamic actions (explosion loading)
- $M_{R,dyn}$: Resisting bending moment available to sustain dynamic actions
- $N_{R,dyn}$: Resisting axial force available to sustain dynamic actions
- h : Depth of the mid-span cross-section

The available resisting bending moment and axial force to face the explosion-induced, dynamic action effects, respectively, $M_{R,dyn}$ and $N_{R,dyn}$, can be substituted by the difference between the total resistance, M_R and N_R , and the initially acting, static action effects, $M_{E,stat}$ and $N_{E,stat}$:

$$\left[\left(\frac{M_R - M_{E,stat}}{h} \right)^2 + (N_R - N_{E,stat})^2 \right]^{0.5} - \left[\left(\frac{M_{E,dyn}}{h} \right)^2 + N_{E,dyn}^2 \right]^{0.5} = 0 \quad (4.27)$$

Figure 4.11 Sequential exposure of column mid-span cross-section to M-N interaction (left) and corresponding deformed member shape (right); Path 1 (blue): Static loading stage; Path 2 (red): Dynamic loading stage

The initially, prior to the explosion event, present action effects, stemming from the gravitational loads are defined by equations (4.28) and (4.29). The bending moment $M_{E,stat}$ is composed by a first and a second order term, respectively, $M_{E,stat,I}$ (4.30) and $M_{E,stat,II}$ (4.31). The former arises due to the initial

eccentricity $e_{\min}$ [83], given by (4.32), whereas the latter is a consequence of the member deflection at the intermediate cross-section due to the gravitational actions, δ_{stat} (Figure 4.11). This deflection takes account of concrete creep-effects.

$$N_{E,\text{stat}} = \xi_{N_{E,\text{stat}}} \cdot (N_G + \psi_1 \cdot N_{Q1} + \psi_2 \cdot N_{Q2}) \quad (4.28)$$

$$M_{E,\text{stat}} = M_{E,\text{stat,I}} + M_{E,\text{stat,II}} \quad (4.29)$$

$$M_{E,\text{stat,I}} = N_{E,\text{stat}} \cdot e_{\min} \quad (4.30)$$

$$M_{E,\text{stat,II}} = N_{E,\text{stat}} \cdot \delta_{\text{stat}} \quad (4.31)$$

$$e_{\min} = \max\left(\frac{h}{20}; 2 \text{ cm}\right) \quad (4.32)$$

Where:

$\xi_{N_{E,\text{stat}}}$:	Model uncertainty coefficient for determining axial forces due to static actions
N_G :	Axial force due to permanent actions (including self-weight of principle beams and columns supported by the column in question as well as permanent loads of the floor and roof system, respectively, g_f and g_r , Table 4.2)
N_{Q1} :	Axial force due to the leading variable action (imposed loads q_i including reduction factor α_A or snow load q_s , Table 4.2)
N_{Q2} :	Axial force due to the accompanying variable action (imposed loads q_i including reduction factor α_A or snow load q_s , Table 4.2)
ψ_1, ψ_2 :	Load combination coefficients for accidental design situations (section 2.3.4)
$e_{\min}$:	Minimum eccentricity (Figure 4.11).
δ_{stat} :	Deflection at the intermediate cross-section due to static actions (Figure 4.11).

The action effects due to the explosion-induced, dynamic loading on the members, which take account of inertia forces and energy dissipation during the short-term load exposure of the members, are given by the expressions established below, respectively for the axial force (4.33) and the bending moment (4.34). Again, the resulting bending moment is composed by a first order (4.35) and a second order term (4.36). The first order terms M_{EX} and $M_{N_{\text{EX}}}$ stem from the horizontal load $\check{p}_{\text{EX,h}}$ and from the eccentrically applied axial force N_{EX} , respectively (Figure 4.11), whereas the second order term arises due to the combined action of the resulting axial force N_E and the resulting deflection δ at the intermediate cross-section, reduced by the initial term caused by the gravitational actions ($N_{E,\text{stat}} \cdot \delta_{\text{stat}}$).

$$N_{E,\text{dyn}} = \xi_{N_{E,\text{dyn}}} \cdot N_{\text{EX}} = \xi_{N_{E,\text{dyn}}} \cdot N_{\text{EX}} \quad (4.33)$$

$$M_{E,\text{dyn}} = M_{E,\text{dyn,I}} + M_{E,\text{dyn,II}} \quad (4.34)$$

$$M_{E,\text{dyn,I}} = \xi_{M_{E,\text{dyn}}} \cdot (M_{\text{EX}} - M_{N_{\text{EX}}}) \quad (4.35)$$

$$M_{E,\text{dyn,II}} = N_E \cdot \delta - N_{E,\text{stat}} \cdot \delta_{\text{stat}} \quad (4.36)$$

Where:

- $\xi_{N_{E,dyn}}$: Model uncertainty coefficient for determining axial forces due to dynamic actions
- N_{EX} : Axial force due to the explosion pressure
- $\xi_{M_{E,dyn}}$: Model uncertainty coefficient for determining bending moments due to dynamic actions
- M_{EX} : Bending moment at the intermediate cross-section due to horizontal explosion load $\tilde{p}_{EX,h}$
- $M_{N_{EX}}$: Bending moment at the intermediate cross-section due axial force N_{EX}
- N_E : Resulting axial force (4.22)
- δ : Resulting deflection at mid-span of the column (Figure 4.11).

The member resistance (M_R , N_R) depends on the geometrical- and material characteristics of the symmetrically reinforced concrete cross sections ($A_s/2 = A_{s1} = A_{s2}$, Figure 4.10), as expressed by (4.37) and (4.38). It should be noted that reinforcement layers A_{s1} and A_{s2} will be referred to as the tensile- and the compression reinforcement layer, respectively, although depending on the considered case, A_{s1} might as well be under compressive stress exposure and/or A_{s2} under tensile stress.

$$N_R = \xi_{N_R} \cdot (F_c - F_{s1} + F_{s2}) \quad (4.37)$$

$$M_R = \xi_{M_R} \cdot (M_{Fc} + M_{Fs1} + M_{Fs2}) \quad (4.38)$$

Where :

- ξ_{N_R} : Model uncertainty coefficient for determining resisting axial forces
- F_c : Internal force resisted by concrete in compression
- $F_{s1,2}$: Internal forces resisted by the reinforcement layers A_{s1} and A_{s2}
- ξ_{M_R} : Model uncertainty coefficient for determining resisting bending moments
- M_{Fc} : Internal moment resisted by concrete in compression
- $M_{Fs1,2}$: Internal moments resisted by the reinforcement layers A_{s1} and A_{s2}

The resisting internal forces and moments, referred to the center of gravity of the mid-span cross-section, are given by equations (4.39) to (4.44) below:

$$F_c = a_c \cdot x \cdot b \cdot \alpha_{cc} \cdot f_{c,dyn} \quad (4.39)$$

$$F_{s1,2} = \varepsilon_{s1,2} \cdot E_s \cdot A_{s1,2} \quad \text{if } \varepsilon_{s1,2} < \varepsilon_{sy} \quad (4.40)$$

$$F_{s1,2} = (f_{y,1,2,dyn} + \frac{(\varepsilon_{s1,2} - \varepsilon_{y,dyn})}{(\varepsilon_{u,dyn} - \varepsilon_{y,dyn})} \cdot (f_{t,1,2,dyn} - f_{y,1,2,dyn})) \cdot A_{s1,2} \quad \text{if } \varepsilon_{s1,2} \geq \varepsilon_{sy} \quad (4.41)$$

$$M_{Fc} = F_c \cdot \left(\frac{h}{2} - k_c \cdot x \right) \quad (4.42)$$

$$M_{Fs1,2} = F_{s1,2} \cdot \left(d - \frac{h}{2} \right) \quad (4.43)$$

$$x = \frac{(\varepsilon_{s2} - \varepsilon_{s1}) \cdot d + \varepsilon_{s1} \cdot h}{\varepsilon_{s1} + \varepsilon_{s2}} \quad (4.44)$$

Where:

x :	Neutral axis depth
$\varepsilon_{s1,2}$:	Strain in the tensile (ε_{s1}) and compression (ε_{s2}) reinforcement layers
$A_{s1,2}$:	Cross-sectional area of the tensile (A_{s1}) and compression (A_{s2}) reinforcement layers
E_s :	Modulus of elasticity of reinforcing steel
$f_{y1,2,dyn}$:	Yield strength of steel in tensile ($f_{y1,dyn}$) and compression ($f_{y2,dyn}$) reinforcement layers under dynamic loading conditions
$f_{t1,2,dyn}$:	Tensile strength of steel in tensile ($f_{t1,dyn}$) and compression ($f_{t2,dyn}$) reinforcement layers under dynamic loading conditions
$\varepsilon_{y1,2,dyn}$:	Yield strain in the tensile ($\varepsilon_{y1,dyn}$) and compression ($\varepsilon_{y2,dyn}$) reinforcement layers under dynamic loading conditions
$\varepsilon_{u1,2,dyn}$:	Strain at maximum force in the tensile ($\varepsilon_{u1,dyn}$) and compression ($\varepsilon_{u2,dyn}$) reinforcement layers under dynamic loading conditions

The resistance (M_R , N_R) may be limited by exceedance of the ultimate curvature (χ_u) of the intermediate cross-section, or by member buckling. In general terms, the former failure mode is representative for smaller axial loads and/or insignificant slenderness ratios λ whereas members characterized by larger λ and/or axial forces are generally prone to buckling. In the present study, the difficulty arises from the fact that the axial force level N_E in the columns is variable due to the dynamic contribution $N_{E,dyn}$ (4.33). Moreover, as stated before, simultaneously to the decrease in axial force, the explosion induces an increase of the bending moment in the members and thereby of the member deflection. Consequently, buckling of the columns might only occur if this increase is relatively high in relation to the decrease in axial force. On the contrary, second order effects will be insignificant and failure will not arise until χ_u will be reached in the critical (intermediate) cross section, coincident with the ultimate moment resisted by this cross-section ($M_{R,u}$). In advance, the corresponding limit states will be termed *buckling limit state* (BLS) and *ultimate moment limit state* (UMLS), respectively (Figure 4.12).

Figure 4.12 Differentiation of buckling limit state (BLS) and ultimate moment limit state (UMLS) in M-N (left) and M- χ (right) representation

It should be noticed that buckling necessarily entails a cross-section failure as well, i.e. χ_u in the critical cross-section will be necessarily exceeded when the column buckles and in that sense the distinction between the BLS and UMLS does not seem to be straightforward. However, in a buckling-critical member an unstable equilibrium situation arises *before* χ_u will be attained in the critical cross-section, at a point, where any additional curvature increase, no matter how small, triggers an uncontrolled and irreversible collapse mechanism of the member [181]. This point corresponds to the initiation of yielding of the reinforcement at the critical cross-section (Figure 4.12, right). Assuming a certain curvature distribution along the member, the corresponding yield eccentricity e_y (including first and second order terms) can be inferred from the yield curvature χ_y of the critical cross-section. The BLS will be attained when upon reaching χ_y the function representing the applied bending moment, $M_E(\chi, N_E)$, is tangent to the resisting moment $M_R(\chi, N_E = N_{R,y})$. On the contrary, buckling will not occur at χ_y . Instead, the plastic response stage will be entered and the limit state (UMLS) will not be attained before $M_E(\chi, N_E) = M_R(\chi, N_E = N_{R,u})$ at χ_u . Consequently, in the UMLS, energy dissipation due to plastic deformations can be accounted for in the analysis, whereas this cannot be done in case of the BLS.

Failure due to shear-axial force interaction at support (V_c -N or V_s -N)

Regarding a potential shear failure of the support cross-section of the columns, the general limit state equation formulated for the beams (4.18) applies ($V_R - V_E = 0$).

Action effect V_E can be exclusively attributed to the horizontal explosion load $\tilde{p}_{EX,h}$ on the columns (Figure 4.9) where account is taken of dynamic effects due to the short load duration.

$$V_E = V_{E,dyn} = \xi_{V_{E,dyn}} \cdot V_{EX} \quad (4.45)$$

Where:

$\xi_{V_{E,dyn}}$: Model uncertainty coefficient for determining the shear forces due to dynamic actions

V_{EX} : Shear force at column support due to the horizontal explosion load $\tilde{p}_{EX,h}$

The formulations of the shear resistance of the column's support cross-section (assuming $\alpha = 90^\circ$ and $\theta = 45^\circ$) under influence of the axial force level in the members are given by (4.46) and (4.48). For the

resistance of the compression struts $V_{R,c}$, this influence will be accounted for by reduction coefficient K (4.47), according to [83], which takes account of the stress state in the compression chord.

$$V_{R,c} = \xi_{V_{Rc}} \cdot (K \cdot b_w \cdot d \cdot 0.5 \cdot \eta_c \cdot f_{c,dyn}) \quad (4.46)$$

$$K = \min\left(\frac{5}{3} \cdot \left(1 - \frac{\sigma_c}{f_{c,stat}}\right); 1.0\right) \quad (4.47)$$

Where:

- K : Reduction coefficient taking account of the stress state in the compression chord
- σ_c : Axial stress state in the concrete cross-section ($\sigma_c > 0$ for compression) taking account of the force resisted by the longitudinal reinforcement.
- $f_{c,stat}$: Concrete compressive strength under static loading conditions

The shear resistance of the tension tie, $V_{R,s}$ (4.48), is composed by two terms, respectively, the contribution of the shear reinforcement, V_{su} , and the contribution of the concrete in compression, V_{cu} . It should be noted that V_{cu} might adopt negative values, i.e. act unfavorably, in case the resulting axial force N_E (4.22) is a tensile force what will be the case if the explosion-induced axial force component $N_{E,dyn}$ (4.33) exceeds the previously acting compression force in the columns, $N_{E,stat}$ (4.28), due to the gravitational actions. As assumed for the beams, the concrete contribution due to aggregate interlock is being neglected in the formulation of $V_{R,s}$.

$$V_{R,s} = \xi_{V_{Rs}} \cdot (V_{su} + V_{cu}) = \xi_{V_{Rs}} \cdot \left(\frac{A_{sw}}{s} \cdot f_{yw,dyn} \cdot 0.9 \cdot d + 0.15 \cdot \sigma_c \cdot b_w \cdot d \cdot \beta_c\right) \quad (4.48)$$

Where:

- V_{su} : Contribution of the shear reinforcement to shear resistance $V_{R,s}$
- V_{cu} : Contribution of concrete in compression to shear resistance $V_{R,s}$
- β_c : Coefficient taking account of shear crack inclination θ_E . If $\theta_E = \theta$, as assumed herein, $\beta_c = 1.0$

4.7 Strict design

4.7.1 Introduction

In section 4.4 a representative set of RC members has been defined by selecting, within reasonable ranges, the nominal values for the most relevant member design parameters (Table 4.2). Once established the accidental hazard scenarios these members might be subjected to (section 4.5), and represented these scenarios in terms of LSFs (section 4.6), the present section addresses the structural design of the relevant member cross-sections. The design is based on the semi-probabilistic safety format (section 2.3.2) where partial safety factors are employed along with the representative values specified in Table 4.2 to deduce the design values for both action effects (E_d) and resistance (R_d). Inasmuch as conservative design has a significant effect on the reliability level, it should be performed strictly ($E_d = R_d$) so the structural members comply exactly with the structural safety requirements laid down in the consistent set of codes employed (section 4.1.2). The detailed design procedure and

assumptions are presented in section 4.7.2, followed by a summary of the results obtained (section 4.7.3).

As stated before in section 4.6.1, the Spanish building codes [179], [180] and standard EHE [83] on, respectively, basis of design, actions and resistance of structures are used in the present study. It must be stated that by now the mentioned set of codes has been replaced by updated versions [71, 72, 182]. In this context it is important to highlight that gas explosions are not explicitly dealt with in the current version of the national standard for actions on structures [182]. Only a brief statement is made on the need to specify, « in the design project of buildings used as chemical plants, laboratories or for storage of explosive materials, the accidental actions to be considered along with the characteristic value and the corresponding model ». No further guidance is given, nor is any specific indication made concerning gas explosions in residential buildings. In the light of the hazard potential such incidents might entail (section 1.1.3), this is considered a deficiency in the current national legislation. In any case this observation justifies the adoption of draft [180] for purpose of the present study, which regarding actions in general, and accidental actions in particular, is equivalent to the respective Eurocodes [17, 82]. Also the herein employed CTE code-draft [179] and standard EHE-98 [83] are, to a large extend, compatible with the respective Eurocodes on, respectively, basis of design [11] and design of RC structures [84]. In section 4.7.4, the main differences between the codes used in the present study and the corresponding standards in force (national standards and Eurocodes) are described.

4.7.2 Design procedure and assumptions

A design procedure has been established consisting essentially of three steps:

- Based on the hazard scenarios identified in section 4.5 and the corresponding LSFs defined in section 4.6, the intermediate and support cross-sections of the selected members are designed for the *accidental design situation* (section 2.3.4).
- Where relevant, the next step involves a design of the same cross-sections according to the *persistent situations*, which refer to conditions of normal use. The corresponding hazard scenarios and design assumptions are adopted from prior studies [78, 99].
- Where relevant, a *design check* (Figure 4.1) is carried out to identify the *mandatory design situation*, i.e. the situation which fits the cross-section in question with the largest resistance.

Beams - Bending failure at mid-span ($M_{(+)}$ or $M_{(-)}$)

The design of the mid-span cross-section of the beams will be based on the limit state function for bending (4.14). For a given design bending moment M_{Ed} , given characteristic material strengths (f_{ck} and f_{yk}), cross-section dimensions (b and h) and concrete cover (d_1), the strict design should deliver the required amount of longitudinal (tensile) reinforcement. When positive bending ($M_{(+)}$) is concerned, this amount should result from a comparison of the accidental- (load arrangement $p_{EX(+)}$) and the persistent design situation, respectively, $A_{s1,req,acc}$ and $A_{s1,req,per}$:

$$\Pi_{d,req} = \max(A_{s1,req,acc}; A_{s1,req,per}) \quad (4.49)$$

The mid-span cross-section width will be defined as $b = \max(0.2 \text{ m}; h/2)$, with an initially fixed minimum depth h_{min} a function of the beam-span l : $h_{min} = l/15$. For the established cross-sectional dimensions the design parameter $A_{s1,req,acc}$ will then be determined, such that M_{Rd} , based on (4.16) will equal $M_{Ed,acc}$. This is being followed by a verification of the cross-sectional ductility according to (4.50). In case this requirement is not met, depth h will be successively increased (in steps of $\Delta h = 5 \%$) and $A_{s1,req,acc}$ determined again until the cross-section ductility is verified. Finally, the required reinforcement due to

the persistent design situation, $A_{s1,req,per}$, will be obtained such that $M_{Rd} = M_{Ed,per}$, under assumption of the cross-sectional dimensions laid down for determination of $A_{s1,req,acc}$.

$$\frac{x}{d} \leq 0.4 \quad (4.50)$$

The previously fixed cross-sectional dimensions are subsequently employed for design of the mid-span cross-section subjected to negative bending ($M_{(-)}$). Since under the assumptions met such a failure cannot be triggered by the persistent load scenario, the required amount of the tensile reinforcement will be directly deduced from the accidental design situation (load-arrangement $p_{EX(-)}$):

$$\Pi_{d,req} = A_{s1,req,acc} \quad (4.51)$$

It should be recalled that the design for $M_{(-)}$ makes only sense for members exposed to a design-explosion load which exceeds the opposed, gravitational design loads (Figure 4.8).

Beams - Shear failure at support (V_c or V_s)

The design of the support cross-section of the beams will be based on the limit state function for shear (4.18). For a given design shear force V_{Ed} , a given characteristic concrete compressive strength (f_{ck}), cross-section depth (h) and concrete cover (d_1), the strict design should deliver the required width of the support cross-section b_w in order to assure the resistance of the compression strut, $V_{Rd,c}$ (4.20). This width should result from a comparison of the accidental (load-arrangement $p_{EX(+)}$) and the persistent design situation, respectively, $b_{w,req,acc}$ and $b_{w,req,per}$:

$$\Pi_{d,req} = \max(b_{w,req,acc}; b_{w,req,per}) \quad (4.52)$$

Depth h of the support-cross-section will be adopted from the design of the mid-span cross-section. The required widths $b_{w,req,acc}$ and $b_{w,req,per}$ will then be determined in compliance with the design limit state conditions $V_{Rd,c} = V_{Ed,acc}$ and $V_{Rd,c} = V_{Ed,per}$, respectively.

Moreover, the support cross-section of the beams should be designed in order to guarantee the resistance of the tension tie, $V_{Rd,s}$ (4.21). Therefore, for a given design shear force V_{Ed} , a given characteristic yield strength of the shear reinforcement (f_{yk}), cross-section depth (h) and concrete cover (d_1), the strict design should deliver the required amount of transverse reinforcement per unit length (A_{sw}/s). Again, this amount should result from a comparison of the accidental- (load arrangement $p_{EX(+)}$) and the persistent situation, respectively, $(A_{sw}/s)_{req,acc}$ and $(A_{sw}/s)_{req,per}$:

$$\Pi_{d,req} = \max[(A_{sw}/s)_{req,acc}; (A_{sw}/s)_{req,per}] \quad (4.53)$$

The required amounts of shear reinforcement $(A_{sw}/s)_{req,acc}$ and $(A_{sw}/s)_{req,per}$ will be determined in compliance with the design limit state conditions $V_{Rd,s} = V_{Ed,acc}$ and $V_{Rd,s} = V_{Ed,per}$.

Columns - Failure due to bending-axial force interaction at mid-span ($M-N$)

The design of the intermediate cross-section of the columns will be based on the general limit state functions for bending-axial force interaction (4.24, 4.25). For a given pair of design axial force N_{Ed} and bending moment M_{Ed} (including second order effects), given characteristic material strengths (f_{ck} and f_{yk}), cross-section dimensions (b and h) and concrete cover (d_1), the strict design should deliver the required amount of longitudinal reinforcement, $A_{s,req}$ ($A_{s,req} = 2 \cdot A_{s1,req} = 2 \cdot A_{s2,req}$). Also in this case, $A_{s,req}$ should result from a comparison of the accidental (load arrangement $N_{EX(-)}$) and the persistent design situation, respectively, $A_{s,req,acc}$ and $A_{s,req,per}$:

$$\Pi_{d,req} = \max(A_{s,req,acc}; A_{s,req,per}) \quad (4.54)$$

The initial, quadratic cross-sectional dimensions of the columns will be defined as a function of the axial compression force level due to the gravitational loads, N_{G+Q} (Figure 4.9), where a minimum value of $b_{min} = h_{min} = 0.2$ m will be respected. Under addition of the explosion-induced action effects, which will principally cause a simultaneous decrease of N and increase of M (Figure 4.11), $A_{s,req,acc}$ should be determined such that the resulting pair $(M_{Ed}, N_{Ed})_{acc}$ falls exactly on the M - N interaction-diagram defined according to the accidental design situation. In case the corresponding failure point falls within the tensile failure domain (Figure 2.17), characterized by yielding of the tensile reinforcement A_{s1} , ductility requirement (4.50) is to be checked. If not fulfilled, the cross-section depth h is to be increased (in steps of $\Delta h = 5\%$) followed by a new determination of $A_{s,req,acc}$. This procedure is being repeated until (4.50) is fulfilled. For compression failure, the ductility requirement can be ignored. Finally, the required amount of reinforcement due to the persistent design situation, $A_{s,req,per}$, will be obtained under assumption of the cross-sectional dimensions laid down for determination of $A_{s1,acc}$.

Under the assumption that member deflection under exclusively vertical loads can be neglected, second order effects have been ignored in design for the persistent design situation. In the accidental design situation, however, such effects might be important and should consequently be considered. For this purpose, the bending moment $M_{Ed,acc}$ (4.55) is determined for a nominal eccentricity e_{tot} [83], based on developments in [181]. Eccentricity e_{tot} , represented in the left part of Figure 4.13, is defined as a function of the first order eccentricity e_I , the second order eccentricity e_{II} and coefficient Ψ . The latter is defined as the ratio between the design values for the ultimate- and the yield moment ($\Psi = M_{Rd,u} / M_{Rd,y}$) and introduced to convert the yield eccentricity to the ultimate eccentricity, enabling thereby an ordinary cross-sectional design according to the UMLS (section 4.6.3) under use of the corresponding strain limits provided by the design code [83], shown in the right part of Figure 4.13. The determination of Ψ is based on an approximate design formula given in the mentioned code.

Figure 4.13 Eccentricity (e) – curvature (χ) relationship (without creep-induced term) assumed in member design for accidental design situation (left); Strain limits for ultimate limit state design according to [83] (right)

The first order eccentricity e_I (4.56) takes account of the minimum eccentricity e_{min} and the moment M_{EX} due to the horizontal explosion load on the columns (Figure 4.9). Second order effects are considered by eccentricity e_{II} (4.57) as a function of the effective column length l_0 , constant c depending on the curvature distribution along the member and nominal curvature χ_{tot} . For constant c , standard [83] suggests

a value of $c = 10$ corresponding to a sinusoidal curvature distribution. The nominal curvature χ_{tot} is constituted by the yield curvature due to loads of short durations, χ_y , and a concrete creep-induced term, χ_{cc} . It should be observed that the determination of χ_y is not straightforward since it depends on the, a priori unknown, required amount of longitudinal reinforcement $A_{s,\text{req}}$ in the cross-section. Hence, unless simplified design formulations are employed, χ_y (and $A_{s,\text{req}}$) can only be determined in an iterative design procedure, such as proposed in the Model Code [130]. Similar to the method based on nominal curvature proposed by the Eurocodes [84], the herein used design code [83] suggests approximate design formulations for the determination of χ_y . In many cases, the use of such approximate design formulas will result in conservative design solutions. However, the design might nevertheless be termed as *strict* since it delivers the strictly required amount of reinforcement according to the code rules employed.

$$M_{\text{Ed,acc}} = N_{\text{Ed,acc}} \cdot e_{\text{tot}} = N_{\text{Ed,acc}} \cdot \Psi(e_I + e_{II}) \quad (4.55)$$

$$e_I = e_{\text{min}} + \frac{M_{\text{EX,d}}}{N_{\text{Ed,acc}}} \quad (4.56)$$

$$e_{II} = \frac{l_0^2}{c} \cdot \chi_{\text{tot}} \quad (4.57)$$

$$\chi_{\text{tot}} = \chi_y + \chi_{cc} \quad (4.58)$$

Columns - Failure due to shear-axial force interaction at support (V_c-N or V_s-N)

The design of the support cross-section of the columns will be based on the limit state functions for shear-axial force interaction. For a given pair of design axial force N_{Ed} and shear force V_{Ed} , based on (4.45), given characteristic material strengths (f_{ck} and f_{yk}), cross-section depth (h), concrete cover (d_1) and cross-sectional area of the longitudinal reinforcement (A_s), the strict design should deliver the required width of the support cross-section b_w in order to assure an adequate resistance of the compression strut, $V_{\text{Rd,c}}$ (4.46). Since, according to the assumptions met, the persistent load scenario involves exclusively gravitational actions, which, according to the assumptions met, do not contribute to the probability of column shear failure, the design of width b_w is only required for the accidental situation (load arrangement $N_{\text{EX}(-)}$):

$$\Pi_{\text{d,req}} = b_{w,\text{req,acc}} \quad (4.59)$$

The depth h of the support-cross-section, as well as the amount of longitudinal reinforcement A_s , will be adopted from the design of the mid-span cross-section. The required width $b_{w,\text{req}}$, will then be determined in compliance with the design limit state condition $V_{\text{Rd,c}} = V_{\text{Ed,acc}}$.

The strict design of the tension tie should be carried out in a way that for a given pair of design axial force N_{Ed} and shear force V_{Ed} , given characteristic material strengths (f_{ck} , f_{yk} and f_{yw}), cross-section dimensions (b_w and h), concrete cover (d_1) and cross-sectional area of the longitudinal reinforcement (A_s), the required amount of transverse reinforcement per unit length (A_{sw}/s) is being provided. As explained before, column design for shear is only required for the accidental situation (load arrangement $N_{\text{EX}(-)}$):

$$\Pi_{\text{d,req}} = A_{sw,\text{req,acc}} \quad (4.60)$$

Besides from cross-section depth h , width b_w will be adopted from the previous design of the compression strut. The required amount of shear reinforcement (A_{sw}/s)_{req,acc} will then be determined in compliance with the design limit state condition $V_{\text{Rd,s}} = V_{\text{Ed,acc}}$, where $V_{\text{Rd,s}}$ is based on equation (4.48). It should be observed that, for high axial compression forces N_{Ed} , the contribution of the concrete (V_{cu}) to the shear

resistance might suffice to resist the action effect $V_{Ed,acc}$ without any need for a shear reinforcement. In this case, a strict design of the support cross-section is still possible if the longitudinal reinforcement A_s is being increased ensuring the axial stress state in the concrete cross-section σ_c , and consequently V_{cu} , to diminish, and, at the same time, the required shear reinforcement $(A_{sw}/s)_{req,acc}$ to increase. Hence for these cases, A_s is being gradually enhanced until the first of the following two events occurs: $(A_{sw}/s)_{req,acc}$ equals the minimum shear reinforcement according to [83], given by (4.61), or the concrete contribution to the shear resistance becomes negligibly small ($V_{cu} \cong 0$).

$$\left(\frac{A_{sw}}{s}\right)_{min} = \frac{0.02 \cdot f_{cd} \cdot b_w}{f_{yd}} \quad (4.61)$$

4.7.3 Results

Mandatory design situations and influencing variables

Table 4.3 identifies the mandatory design situations for the structural members designed according to the procedure outlined in the previous section. Concerning positive bending failure ($M_{(+)}$), the strict design leads to 213, out of the initially defined 486 beams, governed by the accidental design situation, whereas in 273 cases the persistent design situation is mandatory. Unsurprisingly, the same proportion (213/273) is obtained for the design of the compression strut (V_c) and the tensile tie (V_s) at the support cross-section of the beams. A strict design for negative bending moments ($M_{(-)}$), relevant only for the accidental design situation, is possible in only 136 out of the 486 beams, since for the remaining 350 members the design moment due to gravitational loads exceeds the value due to the explosion, acting in the opposed sense (Figure 4.8).

The design for M-N interaction at the mid-span cross-section of the columns concludes with 4755 out of 28512 members governed by the accidental design situation and 23757 members where the persistent design situation is mandatory. As mentioned before, column design for shear (V_c -N, V_s -N) is only required for the accidental design situation.

Table 4.3: Identification of mandatory design situation

Element	Cross-section	Failure mode	Design-parameter Π_d	Mandatory design situation		Total
				Accidental	Persistent	
Beams	Mid-span	$M_{(+)}$	A_{s1}	213	273	486
		$M_{(-)}$	A_{s2}	136	-	136/486*
	Support	V_c	b_w	213	273	486
		V_s	A_{sw}	213	273	486
Columns	Mid-span	M -N	A_s	4755	23757	28512
	Support	V_c -N	b_w	28512	-	28512
		V_s -N	A_{sw}	28512	-	28512

* The design for negative bending ($M_{(-)}$) is possible in only 136 out of 486 initially defined beams

In the following, it will be analysed in more detail under which circumstances the design of the members is likely to be governed by the accidental design situation. In Figure 4.14 it can be observed that the variable with greatest effect on design of the *beams* is the explosion-venting coefficient A_v/V (section 4.4.3). As could be expected, the design of members situated in compartments characterized by small A_v/V ratios, and hence by large explosion loads, is more prone to be governed by the accidental situation than if A_v/V is large: 72% out of the 213 beams where this situation is mandatory are characterized by small A_v/V ratios of 0.05, whereas the share reduces to 13% if $A_v/V = 0.15$. The contrary effect is observed for the gravitational loads. In 51% of the 213 beams, the permanent loads on the floor system $g_{f,k}$

(Table 4.2) are low, whereas 21% of the members are subjected to high permanent load levels. It is further shown that the beam design is more likely to be determined by the accidental situation if span l is relatively small what can be mainly attributed to the established relation between l and the permanent load levels supported by the members (section 4.4.4).

Figure 4.14 Percentages (corresponding to the most influent design variables) out of 213 RC beams strictly designed according to the *accidental* design situation.

Summarizing the foregoing it can be concluded that the relation between the magnitude of the explosion- and the gravitational loads on the beams is of utmost importance for member design and, most probably, for the member reliability level. In order to quantify this relation, parameter v_{acc} will be introduced (4.62), defined as the ratio between the nominal explosion load $\tilde{p}_d$ and the total nominal load supported by the beams:

$$v_{acc} = \frac{\tilde{p}_d}{\tilde{g}_{c,k} + \tilde{g}_{f,k} + \psi_1 \cdot \tilde{q}_{i,k} + \tilde{p}_d} \quad (4.62)$$

Varying, within the chosen limits indicated in Table 4.2, the explosion load-controlling parameters A_v/V and p_{stat} , on one hand, and the permanent loads $g_{f,k}$, on the other, a wide range of ratios v_{acc} is covered. For the 213 beams strictly designed according to the accidental design situation (under load arrangement $p_{EX(+)}$), v_{acc} varies between 0.38 and 0.86, with a mean of 0.57 (Table 4.4). For smaller v_{acc} ratios it becomes more likely that the design will be governed by the persistent design situation. For the 273 beams designed in that way, the v_{acc} ratios vary between 0.14 and 0.45, with a mean of 0.29. Thus a transition zone between $v_{acc} = 0.38$ and $v_{acc} = 0.45$ is being identified where, a priori, no conclusion on the mandatory design situation of the beams can be drawn.

Table 4.4: Explosion load parameter v_{acc} for beams (load arrangement $p_{EX(+)}$)

v_{acc}	Mandatory design situation	
	Persistent	Accidental
Min	0.14	0.38
Mean value	0.29	0.57
Max	0.45	0.86

Figure 4.15 identifies the shares corresponding to different design variables out of the 4755 RC columns (Table 4.3) strictly designed for failure mode M-N, according to the accidental load-arrangement $N_{EX(-)}$. It can be observed that design of the columns is significantly influenced by the distance between the columns in the longitudinal sense of the building, l (Figure 4.4). It is shown that the accidental situation is likely to become mandatory for design if distance l is relatively small: In 87% of the 4755 columns, distance l equals 5 or 10 m, whereas in only 1 % of these members l corresponds to a length of 30 or 40

m. Moreover it can be drawn from Figure 4.15 that columns supporting a relatively small number of storeys (n_s) are more prone to be governed by the accidental design situation than if n_s is relatively large: 49% of the 4755 columns do support the weight of less or equal to $n_s = 1$ storey, whereas in only 3% of these members the number of storeys situated above is equal or larger than $n_s = 20$. Other parameters of significant influence on column design are the permanent loads of the floor system, $g_{f,k}$ and the venting coefficient A_v/V of the compartments where the columns are being situated. Less influent, although of more importance than in case of the beams, is the nominal concrete compressive strength f_{ck} . A higher concrete strength implies comparatively smaller cross-section dimensions and hence a higher member slenderness, what makes the member more prone to be affected by second order effects triggered through the explosion-induced member deflection.

Figure 4.15 Percentages (corresponding to most influent design variables) out of 4755 RC columns strictly designed according to *accidental* situation for M-N interaction.

The significant influence of parameters l , n_s and also $g_{f,k}$ on the design of the columns can be principally traced back to the axial force level supported by the members. Generally, the higher the number of storeys situated above the column (n_s), or the higher the permanent loads of the floor system ($g_{f,k}$), the larger is the compression force in the members due to gravitational loads, N_{G+Q} . Larger distances l do not only imply larger levels of permanent loads, $g_{f,k}$ (section 4.4.4), but also larger cross-section dimensions, and consequently, a larger self-weight of the load-bearing structure. Thus, the larger l , the higher is the compression force in the columns due to gravitational loads, N_{G+Q} .

It seems self-explanatory that higher levels of N_{G+Q} will enhance the likelihood that the column design for M-N interaction should comply with the rules of the persistent situation rather than the accidental. What should be highlighted here is that distance l significantly influences this likelihood not only due to its impact on the gravitational load level. Besides from this influence, l has a significant impact on the relation between the explosion-induced bending moment M_{EX} , due to the horizontal explosion load $\tilde{p}_{EX,h}$, and the axial force N_{EX} due to the explosion pressure acting on floor and ceilings of the compartment (Figure 4.9). In consequence of the defined tributary widths and areas in section 4.5.2 (l_{EX} and A_{EX}), the larger distance l , the higher is the axial tensile force component N_{EX} in relation to moment M_{EX} . This, in

turn, has significant consequences for member design, as is being conceptually shown in the left part of Figure 4.16, where three cases are being distinguished, characterized by different levels of design bending moment M_{Ed} and, for sake of simplicity, a constant design axial force N_{Ed} : Case n° 1 represents small values for l and hence a relatively large explosion-induced increase of the bending moment in relation to the decrease of the axial force, i.e. a comparatively high ratio M_{EX}/N_{EX} . The corresponding design point $(M_{Ed,acc,1}, N_{Ed,acc})$ is situated on the M-N diagram representing the required resistance according to the accidental situation, termed $R_{d,acc,1}$. In case n° 2, the value of distance l is supposed to be higher than in the previous case. Consequently, ratio M_{EX}/N_{EX} decreases and so does the required resistance: $R_{d,acc,2} < R_{d,acc,1}$. Necessarily, the likelihood that the accidental design situation governs the member layout diminishes as well. In case n° 3, where l is again increased, ratio $M_{EX,d}/N_{EX,d}$ is even smaller and the corresponding design point $(M_{Ed,acc,3}, N_{Ed,acc})$ calls for a required resistance $R_{d,acc,3,req}$ which is not even sufficient to face the, prior to the explosion event, present gravitational actions $(G+Q)$. Hence, in this case the persistent design situation would be mandatory by all means.

Figure 4.16 Conceptual representation of the influence of distance l on column layout according to the accidental design situation (load arrangement $N_{EX(-)}$) (left); Definiton of angle α_e (right).

As a measure for the location of the design point (M_{Ed}, N_{Ed}) on the M-N interaction diagram, the design eccentricity $e_d = M_{Ed} / N_{Ed}$ (2.51) is introduced and, in order to deal with a dimensionless parameter, normalized with the member's cross-section depth, h (Figure 4.16, right):

$$(e/h)_d = \tan(\alpha_e) = \frac{M_{Ed}}{N_{Ed}} \cdot \frac{1}{h} \quad (4.63)$$

Figure 4.17 shows the ratio between the design axial force $N_{Ed,acc}$ and the maximum design resistance under pure compression, $N_{Rd,max,acc}$ (Figure 4.16, right) as a function of the normalized design eccentricity $(e/h)_{d,acc}$ as obtained for the 28512 columns designed according to the accidental situation. Very large or very small ratios $(e/h)_{d,acc}$ represent conditions approaching pure bending ($N_{Ed,acc} \rightarrow 0$) whereas $(e/h)_{d,acc}$ values tending to zero are indicative of negligible bending moments ($M_{Ed,acc} \rightarrow 0$). Positive ratios $(N_{Ed}/N_{Rd,max})_{acc}$ represent design compression forces ($N_{Ed,acc} > 0$), prevailing in about 98.5 % of the shown results, whereas in 1.5% of the analyzed columns the explosion-induced axial force exceeded the value due to the gravitational loads resulting in a tensile axial force ($N_{Ed,acc} < 0$).

From Figure 4.17 it can be inferred that larger ratios $(e/h)_{d,acc}$ are indicative of members designed for the accidental situation, where the corresponding design point falls, in most cases, within the tensile failure domain. With a few exceptions, this situation is mandatory for member layout if $(e/h)_{d,acc} \gtrsim 1$. On

the contrary, the design point will fall most probably into the compression failure domain if member design is governed by the persistent situation what is found to be very likely if $(e/h)_{d,acc} \lesssim 0.4$. In between these limits, no conclusions can be drawn *a priori* on the mandatory design situation of the columns. If the resulting axial force is a tensile force ($N_{Ed,acc} < 0$), the design is very likely to be performed according to the requirements of the accidental situation. Only in cases where the influence of bending moments is relatively small ($(e/h)_{d,acc} \gtrsim -1$) it might be possible that the persistent design situation calls for a larger reinforcement degree.

Figure 4.17 Ratio between design axial force $N_{Ed,acc}$ and maximum design resistance $N_{Rd,acc}$ as a function of normalized design eccentricity

Figure 4.18 shows that the position of the design point on the M-N interaction diagram is likewise influenced by second order effects. These are represented by ratio $\varphi_{MII,d}$ between the second order contribution $M_{Ed,II}$ and the total design moment M_{Ed} , shown for the 4755 columns governed by the accidental situation as a function of the normalized eccentricity $(e/h)_{d,acc}$. As could be expected, larger resulting axial force levels, represented by small $(e/h)_{d,acc}$ ratios, induce higher second order effects. Moreover, it is noteworthy that due to the conservative code formulations, even very large $(e/h)_{d,acc}$ still entail second order moments of about 6% of the total design moment.

$$\varphi_{MII,d} = \frac{M_{Ed,II}}{M_{Ed,I} + M_{Ed,II}} = \frac{M_{Ed,II}}{M_{Ed}} \quad (4.64)$$

Figure 4.18 Ratio between second order- and total design bending moment as a function of normalized design eccentricity; Members strictly designed according to accidental design situation (load arrangement $N_{EX(-)}$)

Geometrical characteristics of the strictly designed members

Table 4.5 shows the aspect- and reinforcement ratios characterizing the mid-span and the support cross-section of the 486 RC beams designed according to the mandatory design situation. The aspect ratio h/b of the mid-span cross-section varies between 1.67 and 2.00. For positive bending ($M_{(+)}$), the reinforcement ratios for the longitudinal tensile reinforcement at mid-span range from $\rho_l = 0.004$ to 0.029, whereas for negative bending ($M_{(-)}$) far smaller values ratios are being obtained due to the favourable influence of the gravitational loads.

In contrast to the findings from the mid-span cross-section, the aspect ratios of the support cross-section vary largely from $h/b_w = 5.6$ to 41.6. The mean value is situated around $h/b_w = 10.5$. Such high aspect ratios indicate that in practice the requirements for constructive feasibility in many cases probably entail a conservative design of the compression strut. It should also be observed that the relatively small values obtained for b_w entail relatively large shear reinforcement ratios ρ_w , fluctuating from 0.013 to 0.026.

Table 4.5: Geometrical characteristics of mid-span and support cross-section of RC beams strictly designed according to mandatory design situation

Parameter	Ratio	Mean	Min	Max
Mid-span cross-section aspect ratio	h/b	1.96	1.67	2.00
Longitudinal reinforcement ratio ($M_{(+)}$)	$\rho_l = A_{s1} / (b \cdot h)$	0.014	0.004	0.029
Longitudinal reinforcement ratio ($M_{(-)}$)	$\rho_l = A_{s1} / (b \cdot h)$	0.003	0.000	0.010
Support cross-section aspect ratio	h/b_w	10.47	5.62	41.61
Shear reinforcement ratio	$\rho_w = A_{sw} / (s \cdot b_w)$	0.019	0.013	0.026

The aspect and reinforcement ratios characterizing the intermediate- and the support cross-section of the 28512 RC columns designed according to the mandatory design situation are shown in Table 4.6. The mean value of the h/b relationship of the intermediate cross-section is only scarcely above the minimum of 1.0, which characterizes about 66% of the analysed members. Less than 2 % of the members are characterized by aspect ratios $h/b > 2$, the maximum is $h/b = 4$. The longitudinal reinforcement ratio oscillates between $\rho_l = 0.005$ to 0.187 with a mean value of 0.028.

The aspect ratios of the support cross-section are higher than those obtained for the intermediate cross-section, although in comparison to the beams far smaller: The mean value of ratio h/b_w is situated at about 1.8. In about 77% of the 28512 members, this ratio is smaller than 2 and in 85% of the cases studied smaller than 3. The obtained shear reinforcement ratios are likewise smaller than those found for the beams.

Table 4.6: Geometrical characteristics of intermediate and support cross-section of RC columns strictly designed according to mandatory design situation

Parameter	Ratio	Mean	Min	Max
Intermediate cross-section aspect ratio	h/b	1.1	1.0	4.0
Longitudinal reinforcement ratio*	$\rho_l = A_s / (b \cdot h)$	0.028	0.005	0.187
Support cross-section aspect ratio	h/b_w	1.8	1.0	13.2
Shear reinforcement ratio	$\rho_w = A_{sw} / (s \cdot b_w)$	0.003	0.000	0.030

* Based on $A_s = A_{s1} + A_{s2}$

4.7.4 Differences to codes and standards in force

As mentioned in the introduction to section 4.7, the present study is based on a consistent set of national codes which in the meanwhile were substituted by updated versions. The relevant differences between the herein applied codes rules and those contained in the standards in force (national codes or Eurocodes) are described as follows:

- The perhaps most significant difference concerns the partial factors for the concrete compressive strength in the accidental design situation. While a value of $\gamma_c = 1.3$ has been adopted in the national standards [72, 83], Eurocode EN1992-1-1 [84] recommends a value of $\gamma_c = 1.2$.
- In the employed code for design of structural concrete members [83], the coefficient α_{cc} taking account of long term effects on the compressive strength was fixed to 0.85. The current version EHE-08 [72] recommends, in accordance with the current Eurocode EN1992-1-1 [84], a value of $\alpha_{cc} = 1.0$ although values less than 1.0, but not less than 0.85 [72] or 0.8 [84], might be adopted as a function of the relation between the permanent- and the total loads.
- The nominal curvature-based formulation of the design eccentricity e_{tot} (4.55) to take into account second order effects in problems characterized by M-N interaction was withdrawn from the current version of the design code for structural concrete, EHE-08 [72].
- The formulation of coefficient K (4.47), to be factored into the shear resistance $V_{R,c}$, taking account of the stress state in the compression chord, has experienced a change from EHE-98 [83] to the standard in force, EHE-08 [72]. This is shown in Figure 4.19. According to the formulation in [72], adopted from EN1992-1-1 [84], $V_{R,c}$ may increase up to 1.25 times the value suggested by the former EHE-98 [83] if the compressive stresses are moderate ($0.25f_{cd} < \sigma_c < 0.5f_{cd}$). Beyond this limit, the larger the stresses in the compression chord, the smaller is the difference between both formulations.

Figure 4.19 Comparison of coefficient K for modification of shear resistance $V_{R,c}$ according to EHE-98 [83] and EHE-08/EN1992-1-1 [72, 84].

The impact of the mentioned differences on the overall results and conclusions of the present study is considered to be generally small, if not at all negligible, as probably in the case of the changes concerning coefficient α_{cc} . Section 6.2.4 addresses the influence of partial factor γ_c for concrete, coefficient K for modification of shear resistance $V_{R,c}$, and the eccentricity e_{tot} for design under influence of second order effects on the reliability level of the studied RC members.

4.8 Characterization of basic variables

4.8.1 Introduction

As a part of *step 5* of the established analysis procedure (section 4.3), the present section addresses the representation of the uncertainties associated with the reliability problems treated in the present study. As stated in section 2.2.2, this can be performed on the basis of a statistical characterization of the random variables involved in the defined LSFs (section 4.6), such as the variables quantifying loads, geometry or material properties. Therefore, probabilistic models in terms of appropriate statistical distributions and their corresponding moments have to be established.

In a prior study [78], probabilistic models for the most relevant structural design variables were deduced which represent the uncertainty associated with the rules laid down in the herein employed consistent set of design codes (section 4.7.1). These models, presented in Table 4.7, are generally consistent with the models laid down in the Probabilistic Model Code [80] and are suitable for practical applications. Although they were derived in the context of persistent design situations, some may also be adopted for the herein envisaged reliability analysis in connection with gas explosions. Models designed for static load variables or referring to the geometric dimensions of structural members, for instance, do not require modification for a use in the context of accidental situations. Other variables, however, are to be adapted to take into consideration the effects of high loading rates induced by gas explosions. This concerns particularly the variables characterizing the dynamic material behaviour, which might be subject to considerable modifications in comparison to a static load exposure (section 3.2.2). This is being addressed in section 4.8.3, preceded by the stochastic characterization of the variables representing the dynamic loads and load effects (section 4.8.2).

Table 4.7 Probabilistic models for static actions, action effects and resistance of RC members [78]

Variable	Notation	Type*	Bias (μ/v_k)**	CoV (σ/μ)***
<i>Static actions</i>				
Principle RC member self-weight	g_c	N	1.00	0.04
Other permanent loads	g_f, g_r	N	1.00	0.10
Imposed loads on building floors	q_i	G	0.68	0.26
Snow load	q_s	G	0.33	0.81
<i>Uncertainties associated with modeling of action effects</i>				
Bending moments	$\xi_{M_{E,stat}}$	LN	1.00	0.10
Shear	$\xi_{V_{E,stat}}$	LN	1.00	0.10
Axial compression forces	$\xi_{N_{E,stat}}$	LN	1.00	0.05
<i>Geometry of RC members</i>				
Cross-sectional dimensions	b, h, b_w	N	1.00	0.03
Effective cross-section depth	d	N	1.00	0.04
Reinforcing steel cross-section area	A_s, A_{sw}	N	1.00	0.02
<i>Material strength</i>				
Concrete compressive strength	$f_{c,stat}$	LN	1.24	0.18
Reinforcing steel yield strength	$f_{y,stat}, f_{yw,stat}$	LN	1.12	0.05
Reinforcing steel tensile strength ****	$f_{t,stat}$	LN	1.24	0.06
<i>Uncertainties associated with modeling of resistance of RC members</i>				
Bending moments	ξ_{M_R}	LN	1.00	0.05
Shear (tensile tie)	$\xi_{V_{RS}}$	LN	1.00	0.05
Shear (compression strut)	$\xi_{V_{RC}}$	LN	1.40	0.25
Axial compression forces	ξ_{N_R}	LN	1.00	0.05

* Distribution type: N = Normal, LN = Lognormal, G = Gumbel

** Bias = ratio between mean value μ and nominal value v_k

*** CoV (coefficient of variation) = ratio between standard deviation σ and mean value μ

**** Model derived in present study (see section 4.8.3)

4.8.2 Dynamic loads and load effects

Approach

The present section addresses the probabilistic characterization of the explosion-induced action effects affecting the defined RC members (bending moments M_{EX} , $M_{N,EX}$, shear forces V_{EX} and axial forces N_{EX}). The approach followed in the present study is to deduce these action effects, referred to as E_{EX} in the general notation, from the corresponding quantities $E_{EX,peak}$ which would result from a static application of the peak value $p_{EX,peak}$ of the time-dependant explosion pressure $p_{EX}(t)$ (Figure 4.7). For this purpose, dynamic load factors DLF_M , DLF_V , DLF_N , in the general notation, DLF_E , are introduced (Figure 4.20) which take account of dynamic effects, such as inertia forces and energy-dissipation, which arise in consequence of the short load durations. The DLF_E relate the action effects E_{EX} and $E_{EX,peak}$ as follows:

$$E_{EX} = DLF_E \cdot E_{EX,peak} \quad (4.65)$$

Figure 4.20 Schematic illustration of dynamic load factor DLF_E for computation of dynamic, explosion induced action effect E_{EX} from corresponding value $E_{EX,peak}$ due to statically applied peak-load $p_{EX,peak}$

Predicting the dynamic load effects as described is obviously subject to uncertainties. Factoring in the model uncertainties by means of coefficients $\xi_{E,dyn}$, the general expression for the dynamic action effects E_{dyn} ($M_{E,dyn}$, $V_{E,dyn}$ and $N_{E,dyn}$), compatible with the definitions of the LSFs (section 4.6), is given as:

$$E_{dyn} = \xi_{E,dyn} \cdot E_{EX} = \xi_{E,dyn} \cdot DLF_E \cdot E_{EX,peak} \quad (4.66)$$

The determination of the magnitudes $E_{EX,peak}$ requires a probabilistic model for the explosion peak-load $p_{EX,peak}$, what will be addressed below. Subsequently, reference will be made to the probabilistic representation of the DLF_E including the associated model uncertainty.

Explosion peak pressure

In section 3.1.4, different models for a probabilistic characterization of $p_{EX,peak}$ were presented. It was shown that considerable differences exist between these models regarding both the definition of a mean value and the corresponding scatter. The huge differences concerning the variability of the peak pressure generated in a gas explosion are especially noteworthy and, based on the available information, cannot be explained in a reasonable way. Since, on the other hand, the gas explosion pressure plays a fundamental role in the evaluation of the reliability level of the herein studied beams and columns it is considered essential to analyse this variable in more detail. For this purpose, the experimental results in [116], which, as exposed in section 3.1.4, constitute the basis of the current Eurocode load model (Figure 3.3), are subjected to a statistical analysis.

A total of 15 test results within ranges of $1.2 \text{ kN/m}^2 < p_{stat} < 21.4 \text{ kN/m}^2$ and $0.05 \text{ m}^{-1} < A_v/V < 0.15 \text{ m}^{-1}$ are taken into account in the derivation of a model for the mean value of the explosion peak pressure $\mu_{p_{EX,peak}}$, all of which were obtained under the conservative assumption of a homogeneously distributed, stoichiometric mix of gas and oxygen [116]. Four additional tests, where the gas concentration was varied, are not included in the regression analysis for these tests were conducted only for a specific p_{stat} value ($p_{stat} = 21.4 \text{ kN/m}^2$) what would entail certain distortion in the statistical evaluation. The 15 test results are plotted in Figure 4.21, which distinguishes the peak pressures corresponding to the first (upper figure) and the second (lower figure) pressure pulse observed in the tests carried out [116], respectively p_1 and p_2 (see also Figure 3.2). Linear regression analysis is conducted to find the functions best representing mean $\mu_{p_{EX,peak}}$, shown in Figure 4.21. In line with the suggestion in [116], the exponential model obtained for p_1 depends alone on p_{stat} , whereas the power-law model which characterizes p_2 is function of both p_{stat} and A_v/V .

Figure 4.21 Derivation of mean value (lines) for explosion peak pressure from test results (dots) [109]; upper: first peak pressure pulse (p_1); lower: second pressure pulse (p_2)

Following the approach in [116], the final expression for the mean peak pressure model $\mu_{pEX,peak}$, published in [183], is given by the maximum of the functions found for p_1 and p_2 . Under the assumption of $p_{stat} = 7 \text{ kN/m}^2$, chosen for purpose of example, $\mu_{pEX,peak}$ is compared in Figure 4.22 to the mean value of the empirical model determined in [108] (blue curve), which was likewise deduced from Dragosavics test series [116]. The fact that this model predicts different mean values of the peak pressure than the herein obtained model (lower pressures for ratios A_v/V up to approximately 0.11 and higher values beyond that limit) can be mainly attributed to the circumstance that a total of 6 tests (encircled in Figure 4.21), conducted to quantify the scatter associated with the pressure generation under nominally equally circumstances, were not taken into account in the statistical evaluation in [108]. Interestingly, the herein obtained model is qualitatively similar to the mean value of Schmidt's model (green curve) based on numerical simulations [110]. However, this model predicts far higher explosion pressures across the entire range of A_v/V , what, as already mentioned in section 3.1.4, can be probably traced back to the influence of random parameters, like the point of ignition or obstacle-induced turbulences, which Schmidt studied in detail. On the contrary, it should be kept in mind that the herein derived model $\mu_{pEX,peak}$ is conservative (i.e. predicts comparatively high pressures) with regard to the gas concentration of the gas-oxygen mixture before ignition, as mentioned before. All in all, the derived model is judged to provide a fairly good estimate of the expected peak pressure in a gas explosion event.

Figure 4.22 Gas explosion peak pressure $p_{EX,peak}$ versus venting coefficient A_v/V ($p_{stat} = 7 \text{ kN/m}^2$): Mean value obtained in the present study versus mean values according to [108] and [110].

In order to analyse the scatter corresponding to the obtained pressure model, only the test results corresponding to a venting panel failure pressure $p_{stat} = 21.4 \text{ kN/m}^2$ are taken into account for in this case the sample size is sufficiently high to perform a statistical evaluation. As shown in Table 4.8, the sample consists of $s_n = 8$ test results if pressure pulse p_1 is under concern, whereas for p_2 this number reduces to 7. Sample coefficients of variation ($CoV_{p_{EX,peak,s}}$) of 0.33 and 0.36 are found for p_1 and p_2 , respectively, confirming the findings in [108].

Table 4.8 Test results [116] considered for quantification of scatter associated with peak pressure $p_{EX,peak}$

Test n°	$A_v / V \text{ [m}^{-1}\text{]}$	$p_{EX,peak,p1} \text{ [kN/m}^2\text{]}$	$p_{EX,peak,p2} \text{ [kN/m}^2\text{]}$
17	0.05	27.0	*
19	0.15	13.0	9.0
25	0.15	18.0	22.5
26	0.15	12.0	10.4
27	0.15	18.5	12.3
28	0.15	18.5	11.0
29	0.15	10.5	10.0
30	0.15	24.6	15.0
$\mu_{p_{EX,peak,s}}$		17.8	12.9
$\sigma_{p_{EX,peak,s}}$		5.9	4.7
$CoV_{p_{EX,peak,s}}$		0.33	0.36

*Not considered due to different venting coefficient A_v/V

In the following, the statistical uncertainties (section 2.2.1) due to the reduced sample size will be factored into the analysis. Therefore, the following two assumptions will be made:

- the population mean $\mu_{p_{EX,peak}}$ (2.16) is known and equals the sample mean $\mu_{p_{EX,peak,s}}$
- the population variance $\sigma_{p_{EX,peak}}^2$ (2.17) is unknown

If, for example, a normal distribution is assumed to characterize variable $p_{EX,peak}$, a p-fractile estimate $x_{p_{EX,peak,p,est}}$ associated with a specific probability p can be made as a function of coefficient $k_{s,p}$, taking account of the sample size [184]:

$$x_{p_{EX,peak},p,est} = \mu_{p_{EX,peak},s} + k_{s,p} \cdot \sigma_{p_{EX,peak},s} \quad (4.67)$$

Subsequently, the population standard deviation $\sigma_{p_{EX,peak}}$ can be estimated from (4.68) as a function of $x_{p_{EX,peak},p,est}$ and coefficient $k_{s,p,\infty}$ which accounts for an infinite sample size:

$$\sigma_{p_{EX,peak}} = \frac{x_{p_{EX,peak},p,est} - \mu_{p_{EX,peak},s}}{k_{s,p,\infty}} \quad (4.68)$$

The population coefficient of variation, $CoV_{p_{EX,peak}}$, can then be readily obtained by formulating the ratio between $\sigma_{p_{EX,peak}}$ and $\mu_{p_{EX,peak}}$. It should be mentioned that coefficients $k_{s,p}$ and $k_{s,p,\infty}$, which can be derived from the noncentral t-distribution, depend on both the probability p corresponding to the desired fractile and on the confidence level γ that the p -fractile estimate $x_{p_{EX,peak},p,est}$ will lie on the safe side of the real value $x_{p_{EX,peak},p}$. Confidence levels γ greater than 0.75 are recommended in order to take into account statistical uncertainty [184].

Tables 4.9 and 4.10 show the obtained results depending on different assumptions for the statistical distribution of variable $p_{EX,peak}$, probability p and confidence level γ . The latter turns out to be the most influent parameter. While the population coefficients of variations $CoV_{p_{EX,peak}}$ oscillate between 0.43 to 0.49 if $\gamma = 0.75$ is assumed, $CoV_{p_{EX,peak}}$ raises to values between 0.62 and 0.75 if a confidence level of $\gamma = 0.95$ is demanded. The choice between a normal and a lognormal distribution (for the latter, equations (4.67) and (4.68) change), however, has a minor impact on $CoV_{p_{EX,peak}}$. Nevertheless, it should be observed that the assumption of a normal distribution might entail negative p -fractile estimates, even if a probability of $p = 0.05$ is considered. Based on the experimental observations in [116] it is considered highly unlikely that peak pressures of the pressure pulses p_1 or p_2 might adopt negative values. It should also be recalled in that context that the negative pressure phase following pulse p_1 is characterized by values of insignificant magnitude (Figure 3.2). Consequently, the assumption of a lognormal distribution seems to be a more appropriate choice, following recommendations in [7] (section 3.1.4). Defining a confidence level of $\gamma = 0.95$ as a requirement, the adoption of $CoV_{p_{EX,peak}} = 0.7$ turns out to be a defensible value for both p_1 and p_2 . For reasons of simplification and for want of more detailed data, this value is assumed as a constant across the entire range of possible values of A_v/V and p_{stat} . The finding $CoV_{p_{EX,peak}} = 0.7$ is in line with the indications in [24] (section 3.1.4).

Table 4.9 Determination of coefficient of variation $CoV_{p_{EX,peak}}$ for pulse p_1 , taking account of sample size $s_n = 8$

Distribution	Lognormal (LN)				Normal (N)			
γ	0.75		0.95		0.75		0.95	
p	5	1	5	1	5	1	5	1
$x_{p_{EX,peak},p,est}$	8.07	6.06	5.76	3.89	4.86	-0.14	-1.02	-7.85
$\sigma_{p_{EX,peak}}$	0.45	0.44	0.65	0.63	7.84	7.69	11.42	11.01
$CoV_{p_{EX,peak}}$	0.47	0.46	0.73	0.70	0.44	0.43	0.64	0.62

Table 4.10 Determination of coefficient of variation $\text{CoV}_{\text{pEX,peak}}$ for pulse p2, taking account of sample size $s_n = 7$

Distribution	Lognormal (LN)				Normal (N)			
γ	0.75		0.95		0.75		0.95	
p	5	1	5	1	5	1	5	1
$x_{\text{pEX,peak},\text{p,est}}$	6.09	4.63	4.26	2.89	2.40	-1.71	-2.97	-8.75
$\sigma_{\text{pEX,peak}}$	0.43	0.42	0.65	0.62	6.38	6.27	9.64	9.30
$\text{CoV}_{\text{pEX,peak}}$	0.45	0.44	0.72	0.69	0.49	0.49	0.75	0.72

In the notation of (2.18), the derived probabilistic model for variable pEX,peak can be summarized by (4.69), where $\mu_{\text{pEX,peak}}$ is specified in Figure 4.22. In any case, it should be stressed that the foregoing developments are generally indicative of the high degree of uncertainty associated with the probabilistic model derived. An analysis of the explosion pressure influence on the reliability level of the RC members is recommended (Chapter 6).

$$\text{pEX,peak} \sim \text{LN}(\mu_{\text{pEX,peak}}, 0.7 \cdot \mu_{\text{pEX,peak}}) \quad (4.69)$$

Dynamic load factors

As stated before, within the defined LSF, the estimation of dynamic action effects E_{EX} (4.65) will be based on dynamic load factors DLF_E . These will be deduced from a deterministic, structural analysis based on simplified dynamic models, such as described in section 3.3, what will be subject of Chapter 5 of the present thesis. The DLF_E are subject to the uncertainties associated with the simplified models employed for their determination, accounted for by coefficients $\xi_{E,\text{dyn}}$ (4.66). These coefficients will be represented by a lognormal distribution, with mean $\mu_{\xi_{E,\text{dyn}}} = 1.0$ and standard deviation $\sigma_{\xi_{E,\text{dyn}}}$. In the notation of (2.18), it follows:

$$\xi_{E,\text{dyn}} \sim \text{LN}(1.0, \sigma_{\xi_{E,\text{dyn}}}) \quad (4.70)$$

The combined, lognormally distributed variable $\xi_{E,\text{dyn}} \cdot \text{DLF}_E$ is thus characterized by mean $\mu_{\xi_{E,\text{dyn}}} \cdot \text{DLF}_E$ and standard deviation $\sigma_{\xi_{E,\text{dyn}}}$, as schematically shown in Figure 4.23.

Figure 4.23 Lognormal density function for combined basic variable $\xi_{E,\text{dyn}} \cdot \text{DLF}_E$

Based on the reported comparison between the results obtained in, respectively, blast tests and calculations under use of simplified dynamic models (Figure 3.16, Table 3.5, Figure 3.20), a coefficient of variation of 0.2 to 0.3 seems to be a reasonable choice for bending-related action effects. For purpose of the present study the former will be adopted ($\sigma_{\xi_{M,\text{dyn}}} = 0.2$). In view of the comparatively higher

discrepancies between the experimental and analytical results (based on simplified models) for the support reactions of RC members (Figure 3.15, Figure 3.18), a coefficient of variation of 0.3 will be adopted for model uncertainties associated with the determination of dynamic shear forces ($\sigma_{\xi_{V,dyn}} = 0.3$). In turn, the model uncertainty associated with dynamic axial forces is estimated to be significantly smaller. For want of detailed information, the value used for modeling axial forces under static loads (Table 4.7) will be assumed ($\sigma_{\xi_{N,dyn}} = 0.05$). An improvement of these partially subjective estimates would require an extensive test campaign and/or the availability of detailed information on tests published in the literature, including a rigorous characterisation of the relevant properties defining both loading and resistance of the tested members. As emphasized in section 3.3.6, such a characterization is essential for an adequate estimation of the model accuracy.

Strain and displacements

The limit state functions corresponding to bending failure of both beams and columns are function of the internal forces F_s resisted by the longitudinal steel layers, which, in turn, are dependant on the steel strain, ϵ_s . As defined by (4.17), bending failure of the beams arises after yielding of steel layer A_{s1} (Figure 4.10) and strain ϵ_{s1} can be readily obtained from a deterministic structural analysis under dynamic loading conditions (Chapter 5). In case of the columns, the post-yield stage will only be entered for non-buckling critical members (UMLS), as explained before (Figure 4.12). On the contrary, if buckling governs column failure (BLS), either ϵ_{s1} or ϵ_{s2} will equal the yield strain and the associated force in the steel layer $F_{s1,2}$ (4.41) corresponds to the yield force F_y . Irrespective of the governing failure mode, the strains ϵ_{s1} and ϵ_{s2} , and directly related to these (via the curvature), the mid-span displacements δ , required for the determination of the second-order effects, are dependant on the location of the point of equilibrium, $(M_E, N_E) = (M_R, N_R)$, on the interaction diagram (Figure 4.11). The finding of this a priori unknown point requires a specific solution algorithm which accounts for the axial force-dependant formulation of action effects and resistance under consideration of both the geometrical- and material non-linearities involved. This will be addressed in Chapter 5 within the context of the dynamic analysis of the columns. Subsequently, the mentioned algorithm will be coupled to a purpose-developed FOSM-based [67] reliability procedure (section 6.1) in order to obtain the ϵ_s distribution (and the resultant δ) corresponding to the most likely failure point on the LSF (Figure 2.12). Both the ϵ_s and δ are introduced in the reliability analysis as deterministic values.

4.8.3 Material properties

Approach

As mentioned above, probabilistic models for persistent design situations were developed in a former study which represent the uncertainty associated with the rules laid down in the Spanish design codes (Table 4.7). In the context of accidental design situations, the mentioned codes suggest a general reduction of partial safety factors for material strength (section 2.3.4). However, no distinction is made for the type of accidental action involved nor is any explanation given about the circumstances that informed this reduction. In any event, strain rate dependant material behaviour (section 3.2.2) is not explicitly taken into consideration in the design codes. For these reasons, the deduction of a probabilistic model representing the state of uncertainty associated with the design rules for the accidental situation would not provide a realistic estimate of the increase in material properties induced by high loading rates [183].

Figure 4.24 Static (black) and dynamic (red) stress-strain diagrams for concrete under compression (left) and reinforcing steel (right)

The general approach followed in the present study is to represent the relevant material properties under high loading rates, or simply, *dynamic material properties*, m_{dyn} , as the product of the material property associated with quasi-static loading, m_{stat} , and a dynamic increase factor, DIF_m (4.71). The dynamic properties m_{dyn} define the stress-strain relationships under high loading rates, as shown schematically in Figure 4.24 for concrete in compression (left) and the reinforcing steel (right). The DIF_m will be determined by means of simplified models according to [129] and [143], introduced in section 3.3.2. Estimation of the material strain rate is a pre-requisite to applying these models what will be addressed in the context of the dynamic analysis of the structural members (Chapter 5).

$$m_{dyn} = m_{stat} \cdot DIF_m \quad (4.71)$$

Strength

Previously developed probabilistic models, are used to describe the concrete compressive strength $f_{c,stat}$ and the reinforcing steel yield stress $f_{y,stat}$ and $f_{yw,stat}$ under quasi static loading conditions (Table 4.7). The in Table 4.7 represented model for the tensile strength of the reinforcing steel ($f_{t,stat}$) has been derived under the assumption of a lognormal distribution for ratio $k = f_{t,stat}/f_{y,stat}$ with an associated p-fragtile value $k = 1.08$ (Table 4.2) corresponding to a probability of $p = 0.1$ according to [84].

Based on (4.71), the mean values $\mu_{f_{m,dyn}}$ of the dynamic strength variables $f_{m,dyn}$ are derived by multiplying the mean value $\mu_{f_{m,stat}}$ associated with quasi-static loading to the corresponding DIF_m , as illustrated in Figure 4.25. The scatter for $f_{m,dyn}$ is assumed to be the same as in the respective properties under quasi-static loading, $f_{m,stat}$. As far as concrete is concerned, this assumption was theoretically validated by prior research [128].

Figure 4.25 Log-normal density function for dynamic strength variable $f_{m,dyn}$

Strain and modulus of elasticity

In addition to the strength variables, the compressive strain in the concrete at maximum stress (ε_{c1}), the ultimate compressive strain in the concrete (ε_{cu}), the yield strain (ε_y) and the strain at maximum load (ε_u) of the reinforcing steel are required for a complete definition of the uni-axial stress-strain laws (Figure 4.24).

The strains ε_{c1} and ε_{cu} will be adopted from [83] if quasi-static loading conditions are referred to ($\varepsilon_{c1,stat}$, $\varepsilon_{cu,stat}$). Strain ε_{c1} will be converted to a dynamic property ($\varepsilon_{c1,dyn}$) by means of the corresponding DIF (section 3.3.2) before being employed in the derivation of constants a_c and k_c defining the parabola-rectangle shaped stress-strain relationship for concrete under compression. On the contrary, strain ε_{cu} will be adopted as for static loading conditions due to lack of reliable indications which would allow for a different assumption ($\varepsilon_{cu,stat} = \varepsilon_{cu,dyn}$).

The dynamic yield strain of the reinforcing steel $\varepsilon_{y,dyn}$ is implicitly defined by the ratio between the yield strength $f_{y,dyn}$ and the modulus of elasticity E_s . In keeping with the observations in tests (section 3.2.2), the latter is considered as non-strain rate sensitive and will be introduced in the reliability analysis as a deterministic quantity, $E_s = 200.000 \text{ N/mm}^2$ [83]. Due to a lack of experimental evidence (section 3.2.2), a non-strain rate sensitive behavior will be likewise assumed for the strain of the reinforcing steel at maximum load, ε_u ($\varepsilon_{u,stat} = \varepsilon_{u,dyn}$). This material property is defined by (4.72), which assumes that the slope of the post-yield branch of the stress-strain diagram (Figure 4.24) is defined by the characteristic values, f_{yk} , f_{tk} , ε_{yk} and ε_{uk} according to Table 4.2.

$$\varepsilon_u = \frac{f_{y,stat}}{E_s} + \frac{\varepsilon_{uk} - \varepsilon_{yk}}{f_{tk} - f_{yk}} \cdot (f_{t,stat} - f_{y,stat}) \quad (4.72)$$

4.9 Summary

Chapter 4 addresses key issues regarding the approach to the study and the methodology followed in order to attain its objectives. Moreover, it contains basic developments upon which the following chapters are grounded.

The main assumptions for the inference of risk-based requirements for structural safety are described in section 4.1. Acceptable risk is established as the inherent risk set out in existing structural standards, which reflect current *best practice* and are regarded to be acceptable by definition. Risk acceptability therefore depends on the degree of reliability implicitly required by such standards, which is unknown

and which, as previous studies show, might differ fundamentally from nominal target ceilings established in these standards. It was further discussed, how, and under which circumstances, such a level can be defined for accidental situations associated with gas explosions. The need for distinguishing different design situations (accidental or persistent situations) was highlighted (Figure 4.1).

In section 4.2, a mathematical framework for the quantification of structure-related risks is defined. The developments, which are in line with the main principles established in prior studies dedicated to risks in buildings under normal use conditions [9, 10, 13], are based on the state of knowledge regarding explicit approaches for the analysis, assessment and management of risks associated with technical systems, outlined in Chapter 2. In the present context, such a system is described by a particular building structure. Each of the n_j *hazard scenarios* (j) associated with such a structure is represented by a specific collapse scenario triggered by failure of one of its principle loadbearing members, where member failure is characterized by a specific failure mode induced by a particular load arrangement. The hazard scenarios are mathematically described by their occurrence *probability* p_j and the associated *consequences* N_j . Assuming statistical independency between the n_j hazard scenarios, these can be represented in a probability (p_j) -consequence (N_j) diagram, also termed *risk profile* (Figure 4.2). The integral of the risk profile corresponds to the *risk* R associated with the structure in question.

Based on the established mathematical framework, in section 4.3 a procedure (Figure 4.4) is defined which specifies the tasks performed to comply with the established objectives. The procedure comprises the definition of a representative set of ordinary building structures with RC elements (beams and columns) in section 4.4, followed by an identification of the most relevant hazard scenarios (j) associated with these structures in section 4.5. The determination of the occurrence probabilities p_j and the associated consequences N_j forms part of the subsequent quantitative risk analysis. Probabilities p_j are expressed in terms of implicitly acceptable member failure probabilities $p_{f,adm}$, which represent the reliability level implicitly required by the structural standards. The tasks performed for their determination comprise the translation of the defined hazard scenarios into limit state functions (LSF) (sections 4.6), a strict design of the members ($E_d = R_d$) based on these LSF (section 4.7) and the characterisation of the basic variables involved (section 4.8). Moreover, the estimation of probabilities $p_{f,adm}$ calls for the quantification of different dynamic effects associated with the defined member limit states (Chapter 5) and, finally, a reliability analysis (Chapter 6). For the estimation of consequences, N_j , in terms of the number of fatalities associated with the hazard scenario, a model is required what is subject of Chapter 7. As a function of both, p_j and N_j , in Chapter 8 the implicitly acceptable risks associated with the selected structures are established and acceptance criteria deduced from the findings.

A representative set of building structures with RC members (beams and columns) is obtained varying the parameters with the greatest effect on design within practically realistic ranges (section 4.4). The mentioned parameters are related to the geometry of the building structures (section 4.4.2, Figure 4.5, Table 4.1), the properties of the explosion venting panels used in these buildings (section 4.4.3) and the characteristics of the main constitutive structural members (section 4.4.4, Table 4.2). Since it is a conservative approach with regard to the reliability level, only statically determinate members are considered. The representative set of structural members comprises 486 RC beams and 28512 RC columns. Out of these, specific members are selected for the definition of a representative set of 4512 building structures (section 4.4.5).

A qualitative risk analysis is being carried out with the aim to identify the relevant hazard scenarios to the defined structural members (section 4.5). Two subsequent load stages are distinguished (section

4.5.2, Figure 4.8, Figure 4.9). Previously to the explosion event, the structures are subjected to gravitational loads corresponding to the normal use conditions of the buildings (stage 1). In stage 2, a gas explosion occurs, which will generate a blast overpressure that acts simultaneously to these loads. Under the defined load arrangements, bending and diagonal shear failure, in case of the columns influenced by axial forces, are identified as the relevant member failure modes (section 4.5.3). In compliance with the assumption of static determinate members, reasonable hypothesis are established to account for the possibility of a partial- or total system collapse scenario induced by a member failure (section 4.5.4).

Following their qualitative description, the hazard scenarios are represented in terms of limit state functions (LSF), what is addressed in section 4.6. With a few exceptions, the LSFs are based on the codes employed for member design. The mentioned exceptions are introduced in order to accommodate certain aspects, which in the code rules are being addressed in an implicit way or not at all accounted for. This concerns particularly the dynamic effects induced by the relatively high loading rates (inertia forces, energy dissipation and strain rate effects). Of particular interest is the suggested formulation for the RC columns subjected to M-N interaction, where account has been taken of the fact that the reliability level is strongly load-path dependent (section 4.6.3, Figure 4.11). The buckling limit state is distinguished from the ultimate moment limit state (Figure 4.12).

Based on the established LSF's, the selected structural members are designed according to structural safety requirements that reflect current best practice (section 4.7). The design is based on the semi-probabilistic safety format (section 2.3.2) where partial safety factors are employed along with the representative values for load effects and resistance to deduce the corresponding design values (E_d , R_d). Inasmuch as conservative design has a significant effect on the reliability level, it is performed strictly ($E_d = R_d$) so the members comply exactly with the safety requirements laid down in the consistent set of codes employed. In compliance with the considerations in section 4.1, design is based on the mandatory design situation, which demands the higher member resistance according to the employed design rules.

Section 4.8 addresses the statistical characterization of the random variables involved in the defined LSFs. Therefore, probabilistic models in terms of appropriate statistical distributions and their corresponding moments are being established. Special attention is being paid to both, the load effects and structural resistance under explosion load exposure. The dynamic action effects E_{EX} are defined as a function of the respective quantities $E_{EX,peak}$ that would result from a static application of the peak value $p_{EX,peak}$ of the time-dependent explosion pressure $p_{EX}(t)$ (Figure 4.7) and a dynamic load factor DLF_E (section 4.8.2). A probabilistic model for $p_{EX,peak}$ is derived from a statistical analysis of the experimental results which constitute the fundament of the current Eurocode load model (Figure 4.21). The associated scatter, translated into a coefficient of variation of 0.7, matches well indications in [24]. The DLF_E take account of the contribution of inertia forces and energy dissipation on the structural behavior during the blast load exposure. Associated model uncertainties are factored to the DLF_E by means of coefficients $\zeta_{E,dyn}$, characterized by a lognormal distribution, with mean $\mu_{\zeta_{E,dyn}} = 1.0$ and standard deviation $\sigma_{\zeta_{E,dyn}}$ (Figure 4.23), based on test observations (section 3.3.6) and/or subjective estimations ($\sigma_{\xi_{M,dyn}} = 0.2$, $\sigma_{\xi_{V,dyn}} = 0.3$, $\sigma_{\xi_{N,dyn}} = 0.05$). For purpose of a characterization of structural resistance, the mean values $\mu_{m_{dyn}}$ of the relevant mechanical properties (strength, strain and modulus) of concrete and steel under high loading rates are represented as the product of the corresponding mean values associated with quasi-static loading, $\mu_{m_{stat}}$, and dynamic increase factors, DIF_m (section 4.8.3, Figure 4.24, Figure 4.25). Due to a lack of evidence, the corresponding scatter is assumed, as far as strength variables are concerned, as in the respective properties under quasi-static loading conditions. All other properties are treated as deterministic quantities.

Chapter 5 Dynamic analysis

The LSFs defined in section 4.6 account for different dynamic effects, such as the activation of inertia forces, energy dissipation and increased material strain rates, which arise as a consequence of the relatively high loading rates gas explosions might induce in the defined structural members. As a part of step 5 of the established procedure (section 4.3, Figure 5.1), these effects are to be quantified by means of a deterministic dynamic analysis based on simplified models. The following sections 5.1 and 5.2 summarize, respectively, the approach and the procedure adopted to perform this analysis. Subsequently, the implementation of the procedure for the different member types and failure modes is being addressed in section 5.3. A detailed description and discussion of the results obtained is provided in section 5.4. Chapter 5 concludes with a comparative study carried out for the beams, where the employed simplified models are validated by means of non-linear finite element analysis (section 5.5).

Figure 5.1 Procedural flow-chart

5.1 Approach

5.1.1 Introduction

As reported before in section 3.3.1, different levels of approximations may be generally distinguished for structural analysis in association with explosions, with the grounds for calculations ranging from static equivalent loads to complex, numerical simulations of the blast wave and its dynamic effects on the structure. It was stated that the appropriate election of how to model the structural loading and resistance in specific circumstances depends on numerous aspects, as for instance, the type of structural elements involved, the required precision of its response to the explosion loading, or the state of uncertainty associated with this loading.

The fact that both the geometrical properties and boundary conditions of the selected members (section 4.4) are relatively simple and that the dynamic load can be represented as an idealized pressure-time

function (Figure 4.7) enables a level-2 type analysis (section 3.3.1) in the present study. In this analysis, use is made of simplified and computationally efficient dynamic models, such as those described in section 3.3 of the present thesis. In particular, the flexural response of the selected members is computed from a single degree of freedom (SDOF) system (sections 3.3.3, 3.3.4), whereas the reaction forces, representative for the shear forces close to the member supports, are determined from the dynamic equilibrium formulation applied to the members themselves (section 3.3.5). Strain rate effects are accounted for by means of the simplified modeling approaches described in section 3.3.2.

Within the deterministic dynamic analysis, the formulations of action effects and structural resistance of the beams and columns are consistently based on the corresponding Limit State Functions (LSF) defined in section 4.6. The gravitational loads, as well as the geometrical- and static material properties of the members are defined in terms of *mean values* according to Table 4.7.

The main objective of the analysis is to determine the dynamic load- and increase factors (DLF_E and DIF_m) by which means the dynamic action effects (E_{dyn}) and the dynamic material properties (m_{dyn}) are being quantified within the LSF, as expressed by equations (4.66) and (4.71), respectively. The following subsections explain in more detail the approach adopted to deduce DLF 's (section 5.1.2) and DIF 's (section 5.1.3).

5.1.2 Dynamic load factors

The dynamic action effects E_{dyn} (bending moments, axial- and shear forces), to be accounted for within the reliability analysis of the RC members, are quantified by means of dynamic load factors DLF_E (section 4.8.2). The DLF_E (4.66) which depend on the characteristics of both the explosion load and the structural resistance, are time-dependent quantities. However, with regard to the objectives pursued in the present study, only the maximum values, related to the maximum structural response, are of interest.

Beams – Bending failure at mid-span ($M_{(+)}$ or $M_{(-)}$)

In response to load arrangements $p_{EX(+)}$ or $p_{EX(-)}$ (Figure 4.8), the beams may undergo positive or negative flexural failure, respectively (section 4.5.3). In the former case, the explosion-induced deflections and bending moments (δ_{dyn} and $M_{E,dyn}$) are added to the previously present, static action effects (δ_{stat} and $M_{E,stat}$), as schematically shown in Figure 5.2. In the latter case, dynamic and static action effects oppose each other (Figure 5.3). In both cases, the envisaged failure mode is ductile, i.e. energy dissipation due to plastic material deformations may be accounted for. Failure does not occur until the mid-span displacement δ reaches the ultimate value δ_u at time $t = t_u$.

Figure 5.2 Schematic representation of displacement-time (left) and bending moment-time (right) curves for positive bending failure of beams under load-arrangement $p_{EX(+)}$

Figure 5.3 Schematic representation of displacement-time (left) and bending moment-time (right) curves for negative bending failure of beams under load-arrangement $p_{EX(-)}$

Subtracting from the ultimate moment $M_{R,u}$ the contribution $M_{E,stat}$ (4.15) due to the previously acting gravitational loads, yields the available resistance $M_{R,dyn}$ to oppose the explosion-induced action effect $M_{E,dyn}$ (4.15), as expressed per (5.1). Based on the general notation (4.66) and considering the equilibrium condition $M_{E,dyn} = M_{R,dyn}$, the dynamic load factor associated with the bending failure mode of the beams (DLF_M) is defined by (5.2) as the ratio between $M_{R,dyn}$ and moment $M_{EX,peak}$, which would result from a static application of the explosion peak pressure $p_{EX,peak}$ (Figure 5.2, Figure 5.3). This peak pressure is derived from the SDOF analysis assuming that the mid-span cross-section of the beam attains its ultimate deformability ($\delta \cong \delta_u$).

$$M_{R,dyn} = M_{R,u} - M_{E,stat} \quad (5.1)$$

$$DLF_M = \frac{M_{EX}}{M_{EX,peak}} = \frac{M_{R,dyn}}{M_{EX,peak}} \quad (5.2)$$

Beams – Shear failure at support (V_c or V_s)

During the flexural response of the beams to load arrangement $p_{EX(+)}$ (Figure 4.8), reaction forces V_E evolve at the member supports. The maximum value is considered representative for the maximum shear forces acting on the beams, limited by shear resistance V_R associated with, respectively, crushing of the compression strut (V_{Rc}) or yielding of the shear reinforcement (V_{Rs}).

Figure 5.4 Schematic representation of shear force-time curve for beams under load-arrangement $p_{EX(+)}$

In harmony with the previous developments for the flexural failure mode, the dynamic load factor associated with the shear failure mode, DLF_V , is defined by (5.4) as the ratio between the available shear

resistance to face the dynamic action effects, $V_{R,dyn}$ (5.3), and force $V_{EX,peak}$ which would result from a static application of peak pressure $p_{EX,peak}$. This peak pressure is derived from the dynamic equilibrium conditions applied to the members themselves (coupled to the SDOF analysis by means of resistance R) assuming that $V_{E,dyn} \cong V_{R,dyn}$.

$$V_{R,dyn} = V_R - V_{E,stat} \quad (5.3)$$

$$DLF_V = \frac{V_{EX}}{V_{EX,peak}} = \frac{V_{R,dyn}}{V_{EX,peak}} \quad (5.4)$$

It should be stressed that this approach does not account for energy dissipation due to plastic deformations of the transverse reinforcement. A way to consider this is to model the beams by a MDOF-system where both the flexural response and the development of diagonal shear cracks can be represented by separate degree of freedoms, as shown e.g. in [160]. In view of the expected failure probabilities, disregarding the stirrup contribution to the energy dissipation is conservative.

Columns – Failure due to bending-axial force interaction at mid-span ($M-N$)

The structural performance of the blast-exposed columns under load arrangement $N_{EX(-)}$ (Figure 4.9) is characterized by the combined influence of bending moments and axial forces. As previously explained (section 4.6.3), two limit states are distinguished, the ultimate moment limit state (UMLS) and the buckling limit state (BLS). For buckling-critical elements no energy dissipation due to plastic deformations should be accounted for since the occurrence of member instability concurs with initiation of yielding of the longitudinal reinforcement at $M = M_{R,y}$ (Figure 4.12, Figure 5.5 (left)). However, among the strictly designed reinforced concrete columns (section 4.7.3), a large number is not buckling-critical since the member slenderness is small and/or the axial force level in the members is significantly reduced during the explosion event. For these members it can be assumed that, as in case of the beams, the plastic stage of the resistance curve will be entered, and hence a certain amount of energy might be dissipated until cross-section failure arises when the ultimate strain in the concrete or the reinforcing steel is attained at $M = M_{R,u}$ (Figure 4.12, Figure 5.5 (right)).

Figure 5.5 Schematic representation of bending moment-time curves corresponding to buckling limit state (left) and ultimate-moment limit state (right) for columns under load-arrangement $N_{EX(-)}$

Subtracting the moment due to the previously acting gravitational loads $M_{E,stat}$ (4.29) from resistance M_R ($M_{R,u}$ or $M_{R,y}$ depending on whether the UMLS or the BLS is concerned), the available resistance $M_{R,dyn}$ to oppose the explosion-induced action effect $M_{E,dyn}$ (4.34) is given by (5.5).

$$M_{R,dyn} = M_{R,y(R,u)} - M_{E,stat} \quad (5.5)$$

The explosion-induced bending moment $M_{E,dyn}$ can be distinguished into a first order and a second order component. The former, $M_{E,dyn,I}$ (4.35), is attributable to the influence of the horizontal explosion load $\tilde{p}_{EX,h}$ and the eccentrically acting axial force N_{EX} (Figure 4.9). The second order component, $M_{E,dyn,II}$, arises in consequence of the member deflection δ and will be accounted for by means of coefficient $\varphi_{M,II}$ (5.6), previously introduced in the context of the member design (4.64).

$$\varphi_{M,II} = \frac{M_{E,dyn,II}}{M_{E,dyn}} = \frac{M_{E,dyn,II}}{M_{R,dyn}} \quad (5.6)$$

Coefficient $\varphi_{M,II}$, as defined per (5.6), is indicative of the importance of second order effects during the dynamic response of the member. If no such effects influence this response, i.e. $\varphi_{M,II} = 0$, the entire resistance $M_{R,dyn}$ can be allocated to oppose the explosion-induced, first order action effects, $M_{E,dyn,I}$. With increasing second order contribution, hence increasing $\varphi_{M,II}$, the available resistance to withstand these action effects vanishes. In the unrealistic case of $\varphi_{M,II} = 1.0$, the resistance would be entirely consumed by second-order effects and any lateral influence, regardless of its magnitude, would be sufficient to trigger buckling.

Taking into account these considerations, the dynamic load factor DLF_M associated with the flexural failure mode of the columns is defined by (5.7) as the ratio between the available resistance to oppose the explosion-induced, first order moment at mid-span, $M_{R,dyn} - M_{E,dyn,II}$, and moment $(M_{EX} - M_{N,EX})_{peak}$, which would result from a static application of the explosion peak pressure $p_{EX,peak}$. This peak pressure is derived from the SDOF analysis under consideration of axial forces (section 3.3.4) assuming that $\delta \cong \delta_u$ for UMLS members, or $\delta \cong \delta_y$ if the BLS is relevant.

$$DLF_M = \frac{M_{EX} - M_{N,EX}}{(M_{EX} - M_{N,EX})_{peak}} = \frac{M_{R,dyn} \cdot (1 - \varphi_{M,II})}{(M_{EX} - M_{N,EX})_{peak}} \quad (5.7)$$

In Annex A, the implementation of the SDOF analysis under influence of axial forces is illustrated by means of a simplified example-column. Thereby, special attention is being paid to second-order effects and their influence on the explosion peak-load ($p_{EX,peak}$) the member is able to resist. It is shown that an increase of such effects induces an almost proportional decrease of $p_{EX,peak}$, hence resulting in only scarce variations of the DLF_M . The finding applies especially to members with elastic behavior ($\mu_\delta = 1.0$), representative for buckling-critical members (BLS members), where second order effects are important. Non-buckling-critical columns (UMLS members), with certain plastic deformation capacity, are generally associated with comparatively lower compression axial force levels, i.e. lower second order effects. The definition of the member resistance functions in section 5.3.4 duly accounts for the influence of the axial force level on the member deformation capacity.

Finally, it should be stated that oscillations in the longitudinal sense of the columns are of insignificant magnitude. This could be observed during preliminary FE calculations briefly summarized in Annex B. It was shown that the corresponding natural periods T are small in comparison to the load-pulse duration, t_p what is indicative of a quasi-static behavior. Hence, the dynamic load factors for dynamic axial forces affecting the columns can be assumed as $DLF_N = 1.0$.

Columns – Failure due to shear-axial force interaction at support (V_c-N or V_s-N)

In view of a potential column failure due to shear, dynamic load factors DLF_V will be derived and subsequently introduced in the limit state functions corresponding to the tensile-controlled shear failure mechanism (V_s-N) or the failure of the compression strut (V_c-N). The corresponding definitions are equivalent to the developments for the beams, expressed by (5.3) and (5.4), where $V_{E,stat} = 0$: Since in

case of the columns there is no shear force contribution from the gravitational actions, the entire resistance can be allocated to resist the explosion-induced, dynamic action effects, hence $V_R = V_{R,dyn}$. Moreover, it must be considered that axial forces will influence the shear resistance V_R , as defined by (4.46) to (4.48). Dynamic load actors of $DLF_N = 1.0$ will be assumed for the dynamic axial forces (Annex B).

5.1.3 Dynamic increase factors

The reliability analysis of the structural members takes account of the dynamic material properties. As stated in section 4.8.3, these properties are estimated on the grounds of dynamic increase factors, DIF_m (Figure 4.24), which are deduced from simplified models.

For the concrete compressive strength, $f_{c,dyn}$, the DIF model from Model Code (MC) 90 [129] is adopted. As explained in section 3.3.2, this model, expressed by (3.7) to (3.9), seems to be more appropriate for mean value estimations than the model contained in the current MC 2010 [130], which aims a predicting a characteristic value. The same consideration applies to the DIF model for the concrete tensile strength, $f_{ct,dyn}$, which, although it does not appear directly as a basic variable in the limit state function (section 4.6), is involved in the dynamic analysis of the beams subjected to load arrangement $p_{EX(-)}$, as will be shown below (section 5.3.3). The DIF models for the concrete compressive strain at peak-stress, $\varepsilon_{c1,dyn}$, and the modulus of elasticity, $E_{c,dyn}$, are taken from MC 2010 [130]. As stated before, these models did not change with respect to MC 90. Just as the concrete tensile strength $f_{ct,dyn}$, the modulus $E_{c,dyn}$ is not a basic variable in the strict sense but a required parameter for the dynamic analysis of the beams.

The DIF for the reinforcing steel yield- and tensile strength ($f_{y,dyn}$, $f_{t,dyn}$) is deduced from the frequently quoted model proposed by Malvar and Crawford [143], adopted by MC2010 [130] and given by equations (3.12) to (3.14). The model is also adopted for the prediction of reinforcing steel properties under dynamic compression forces.

The estimation of the material strain rate $\dot{\varepsilon}$ is a pre-requisite to applying the described DIF models. This will be accomplished by deriving constant average material strain rates $\dot{\varepsilon}_{m,av}$ (5.8). These are defined as the quotient of strain $\varepsilon_{\sigma,dyn}$ associated with a specific stress state in the material induced by the dynamic explosion loading, divided by the time t_σ needed to attain that stress state [183]. Strain $\varepsilon_{\sigma,dyn}$ can be obtained from cross-sectional analysis or estimated from the adopted constitutive material laws. Time t_σ is computed from the SDOF calculations.

$$\dot{\varepsilon}_{m,av} = \frac{\varepsilon_{\sigma,dyn}}{t_\sigma} \quad (5.8)$$

It should be emphasized that the way by which the material behaviour under dynamic conditions is being accounted for is in several ways a rough simplification only. Yet it is true that the above mentioned DIF equations were empirically derived from tests on specimen exposed to constant strain rates, wherefore their use in association with an average strain rate seems to be at first glance a logical approach. However, it must be kept in mind that material strain rates under blast load influence on structural members are, in fact, not constant quantities, but undergo large variations in both time and space (the strain rate is different in each fiber of each cross-section). Taking account too that the dynamic increase of material properties is influenced by a memory effect [131], i.e. the increase depends on the entire loading history the member has experienced up to a certain point in time, including a possible static stress state prior to the dynamic load exposure [185], casts doubts on the suitability of the followed approach. Moreover, it must be kept in mind that, strictly speaking, the DIF equations adopted here account for the dynamic increase under uniaxial loading conditions, i.e. pure tension or compression stresses in unconfined cross-sections. The use of these equations in the context of explosion-exposed

structural members under bending- or shear stresses, including possible confinement effects, implies additional uncertainties.

In spite of these shortcomings, the herein employed-, or similar DIF models have proven to provide accurate results in a number of explosion- or impact related studies of structures and structural members [144, 155-157, 171, 186, 187], also in association with constant strain-rates [188]. The fact that the assumption of a constant strain rate is actually a sufficiently close approximation for many practical cases, and the observation that the DIF equations are logarithmical, wherefore the prediction of $\dot{\epsilon}$ need not be overly precise [121], explains this to some extent.

5.2 Procedure

For simplified dynamic analysis, an iterative procedure (Figure 5.6) has been developed that factors in material non-linearity, strain rate effects and the gravitational pre-loading of the members prior to the explosion event. The steps comprising the procedure are described below.

Figure 5.6 Dynamic analysis procedure

Since the material strain rates are initially unknown, the first iteration step ($i=1$) is based on the static material properties. For sake of consistency, the constitutive material laws (Figure 4.24, black lines) and the corresponding static material parameters ($\mathbf{m}_{\text{stat}}$) comply with those defined under section 4.8.3.

The stress-strain curves are used to plot a simplified moment-curvature ($M-\chi$) diagram (Figure 5.7, left) at the critical member cross-section where, depending on the member type and the considered load-arrangements, different pre-yield response stages ($M_R \leq M_{R,y}$) and, where relevant, a post-yield stage ($M_{R,y} < M_R \leq M_{R,u}$) are being distinguished. The individual response stages are idealized as linear functions. The curvature ductility ratio μ_χ [189] is obtained from the $M-\chi$ diagram as per (5.9).

$$\mu_\chi = \frac{\chi_u}{\chi_y} \quad (5.9)$$

Figure 5.7 Idealised moment-curvature (left) and resistance-displacement (right) functions assumed for the mid-span cross-section of the beams subjected to load arrangement p_{EX}(+).

The resistance function $R(\delta)$ (Figure 3.9, Figure 5.7, right) is deduced in the following step from the $M-\chi$ diagram. For this purpose resistance values R are obtained from the respective bending moments M as per equation (3.32). The stiffness values k in the pre-yield response stages of the simply supported members are computed from (3.26) using the respective cross-sectional values, EI . Ultimate deflection capacity, δ_u , is deduced from (5.10), adopted from [190], which relates curvature to displacement ductility ratios, respectively μ_χ and μ_δ (3.34). In Annex C, the suitability of (5.10) for purpose of the present study is being proved.

$$\mu_\delta = \frac{\mu_\chi + 1}{2} = \frac{\delta_u}{\delta_y} \quad (5.10)$$

A resistance value, R_{stat} , and the corresponding deflection δ_{stat} must be found for the gravitational loads borne by the beam at the time the explosion occurs (Figure 5.7). Point $(\delta_{stat}, R_{stat})$ defines the origin of the function for effective resistance to the dynamic blast load affecting the members (Figure 5.7, red line). This function is characterised by effective resistances $R_{y,dyn}$ (5.11) and $R_{u,dyn}$ (5.12), stiffness k_{dyn} as well as the effective displacement ductility ratio, $\mu_{\delta,dyn}$ (5.13).

$$R_{y,dyn} = R_y - R_{stat} \quad (5.11)$$

$$R_{u,dyn} = R_u - R_{stat} \quad (5.12)$$

$$\mu_{\delta,dyn} = \frac{\delta_{u,dyn}}{\delta_{y,dyn}} = \frac{\delta_u - \delta_{stat}}{\delta_y - \delta_{stat}} \quad (5.13)$$

It should be noted that Figure 5.7 represents the situation for the beams subjected to positive bending, which will be addressed in detail in section 5.3.2. The figure is anticipated here for purpose of a better

illustration of the involved variables in the development of the dynamic resistance functions of the members. However, equations (5.9) to (5.13) shown before, as well as the corresponding explanations given, are of general validity for all the member types analyzed, i.e. including the beams subjected to negative bending and the columns due to M-N interaction, addressed in sections 5.3.3 and 5.3.4, respectively.

In addition to the resistance function, a dynamic load function must be defined, to which end the triangular load-pulse defined by (3.29) and (3.30), shown in Figure 4.7 is assumed. Load-pulse duration t_p may be assumed to rise with volume of the enclosure where the explosion occurs (section 3.1.4). On these grounds and assuming room volumes smaller than 1000 m³, t_p values from 0.2 s to 0.5 s are adopted depending on the member analysed [183]. Large-span beams (and the columns supporting them), present in larger volume enclosures, are consequently assumed to be exposed to load-pulses of a longer duration (Table 5.1). Moreover, an initial peak value for the pressure-pulse has to be adopted at this stage, $p_{EX,peak,j=1}$ (Figure 4.7, Figure 5.6).

Table 5.1: Load-pulse durations t_p assumed for dynamic analysis of beams and columns

Span l [m]	t_p [s]
5-10	0.20
15-20	0.35
30-40	0.50

Once the parameters of the dynamic resistance- and load functions are defined, the equivalent SDOF quantities are derived by means of transformation factors K_i (Table 3.4) as previously exposed in section 3.3.3. The *constant velocity procedure* [164], is then applied for the stepwise, numerical integration of the equations of motions describing the behaviour of the SDOF system, given by (3.15) to (3.17). The integration yields the maximum explosion-induced deflection, $\delta_{m,dyn}$, which, added to the initial, static deflection, δ_{stat} , gives the total deflection δ_{max} . Where $\delta_{max} - \delta_u$ exceeds the established tolerance, a new explosion peak-pressure $p_{EX,peak,j=2}$ is estimated and the SDOF analysis is repeated (Figure 5.6). This procedure is iterated until the tolerance criterion is met, $\delta_{max} \approx \delta_u$ (section 5.1.2). In addition to δ , the analysis computes the bending moment $M(t)$ at the intermediate member cross-section (Figure 5.2, Figure 5.3, Figure 5.5). The reaction forces $V(t)$ at the supports (Figure 5.4) are found from the dynamic equilibrium conditions (3.41) and (3.42), coupled to the SDOF analysis by means of flexural resistance R . A separate iteration-loop j is required to determine the peak-pressure $p_{EX,peak,j}$ which complies with the condition $V_E = V_R$ according to section 5.1.2.

Strain rate effects are factored into the procedure by deriving DIFs based on a set of average material strain rates $\dot{\epsilon}_{m,av}$ (5.8) according to section 5.1.3. The resulting stress-strain laws characterised by the dynamic material parameters $\mathbf{m}_{dyn}$ (Figure 4.24, red lines) then serve as a basis for a second iteration step ($i=2$). Further to the procedure described, the result of this step is a new set of updated, dynamic material properties $\mathbf{m}_{dyn}$. The procedure reaches conclusion when the difference between the updated and the previous values lies within the established tolerance (1%). Where the difference lies outside the tolerance, further iteration is initiated ($i=3$). A total of two to three steps suffice to obtain sufficiently accurate results.

5.3 Implementation

5.3.1 Introduction

The present section describes the implementation of the described dynamic analysis procedure for the different structural member types and failure modes. The focus is placed on the development of the dynamic resistance function based on the $M-\chi$ analysis (Figure 5.7) and the determination of average strain rates $\dot{\epsilon}_{m,av}$ (5.8) for the estimation of dynamic material properties.

5.3.2 Beams submitted to load arrangement $p_{EX(+)}$

Resistance function

Load arrangement $p_{EX(+)}$ considered for the beams is characterized by two subsequent load stages: *stage 1* due to the static action of gravitational loads and *stage 2* in consequence of the dynamically applied explosion pressure (Figure 4.8). The flexural response of the beams to this load arrangement, represented by the $M-\chi$ diagram at the mid-span cross-section, can be described by three subsequent response functions (RF) (Figure 5.7).

- RF I: uncracked state

$$M_R = \frac{M_{R,cr}}{\chi_{cr}} \cdot \chi \quad \text{if } \chi \leq \chi_{cr} \quad (5.14)$$

- RF II: cracked state prior to yielding of the longitudinal reinforcement

$$M_R = M_{R,cr} + \frac{M_{R,y} - M_{R,cr}}{\chi_y - \chi_{cr}} \cdot (\chi - \chi_{cr}) \quad \text{if } \chi_{cr} < \chi \leq \chi_y \quad (5.15)$$

- RF III: post-yield state

$$M_R = M_{R,y} + \frac{M_{R,u} - M_{R,y}}{\chi_u - \chi_y} \cdot (\chi - \chi_y) \quad \text{if } \chi_y < \chi \leq \chi_u \quad (5.16)$$

The transition between RF I and II is given by the cracking moment $M_{R,cr}$ (5.17) as a function of the concrete tensile-strength and the section modulus W of the mid-span cross-section. Since for the members analyzed here the static action effect $M_{E,stat}$ exceeds $M_{R,cr}$, this transition takes place under the influence of the previously to the explosion event acting gravitational loads. Consequently, the static value of the concrete tensile strength is involved in (5.17), remaining unaffected by strain-rate effects. The same applies to the concrete modulus E_c required for inference of crack-curvature χ_{cr} as the ratio between $M_{R,cr}$ and the cross-sectional stiffness EI_I (Figure 5.7).

$$M_{R,cr} = f_{ct,stat} \cdot W \quad (5.17)$$

Cross-sectional analysis is carried out to compute curvatures χ_y and χ_u corresponding to the yield- and ultimate bending moments, $M_{R,y}$ and $M_{R,u}$, respectively, based on (4.16). Therefore, the relevant dynamic material properties ($f_{c,dyn}$, $\epsilon_{c1,dyn}$, $f_{y,dyn}$ and $f_{t,dyn}$) are employed for all iteration-steps $i > 1$ (Figure 5.6), as will be outlined below. Response functions II and III are thereby completely defined. The curvature ductility, μ_χ (5.9) and the cross-sectional stiffness in the cracked state, EI_{II} (Figure 5.7) are subsequently deduced from the $M-\chi$ diagram. The contribution of the concrete between the cracks (*tension stiffening*) is being neglected. Consequently, a possible increase of the cross-sectional stiffness due to a dynamic increase of the concrete modulus E_c [126, 185] during response stage II is neither accounted for.

The effective (dynamic) resistance function $R_{\text{dyn}}(\delta)$ to the applied explosion load is finally deduced as described in section 5.2. Its point of origin $(\delta_{\text{stat}}, R_{\text{stat}})$ is given by interception of RF II at $R=R_{\text{stat}}$, required to resist the initially present gravitational loads. The corresponding stiffness k_{dyn} is represented by value k_{II} converted from E_{II} (Figure 5.7). Displacement ductility $\mu_{\delta, \text{dyn}}$ is given by (5.13).

Average strain rates

The dynamic increase factors DIF_{fc} and DIF_{ec1} for the determination of the dynamic concrete compressive strength $f_{\text{c, dyn}}$ and the compressive strain at maximum stress $\epsilon_{\text{c1, dyn}}$ (Figure 4.24) are based on the average strain rate $\dot{\epsilon}_{\text{c, av}}$ given by (5.18).

$$\dot{\epsilon}_{\text{c, av}} = \frac{\epsilon_{\text{c, y}} - \epsilon_{\text{c, stat}}}{t_y} \quad (5.18)$$

The numerator of (5.18) represents the compressive strain induced in the most loaded concrete fiber on top of the mid-span cross-section by the dynamic load at time t_y (Figure 5.2). It is determined by subtracting compressive strain $\epsilon_{\text{c, stat}}$ due to the gravitational loads at $t = t_0$ (Figure 5.2) from the total strain $\epsilon_{\text{c, y}}$, both inferred from cross-sectional analysis (Figure 5.8). Time t_y is computed from the SDOF analysis.

Figure 5.8 Strain distribution in the mid-span cross-section due to initially acting static loads at $t = t_0$ (stage 1), and total strain distribution at $t = t_y$

In an equivalent way, average strain rate $\dot{\epsilon}_{\text{s, av}}$ is deduced from (5.19) and employed to compute DIF_{fy} and DIF_{ft} for the yield- and tensile strength of the reinforcing steel layer A_{s1} (Figure 5.8) under dynamic loading conditions, respectively $f_{\text{y, dyn}}$ and $f_{\text{t, dyn}}$. The total strain of the reinforcement layer at yield, $\epsilon_{\text{s1, y}}$, is given by Hook's law, $\epsilon_{\text{s1, y}} = \epsilon_{\text{y, dyn}} = f_{\text{y, dyn}} / E_{\text{s}}$. It should be observed that the formulation for DIF_{ft} (3.12) is a function of the yield strength what seems to justify the use of the yield point as a reference for the derivation of a corresponding average strain rate.

$$\dot{\epsilon}_{\text{s, av}} = \frac{\epsilon_{\text{s1, y}} - \epsilon_{\text{s1, stat}}}{t_y} \quad (5.19)$$

The dynamic increase of the concrete compressive strength involved in the resistance of the compression strut (4.20) close to the beam support cross-sections is based on average strain rate $\dot{\epsilon}_{\text{cv, av}}$ given by (5.20). Within this formulation, ϵ_{cv} is the total strain induced in the concrete strut by shear force V_{E} , approximated by the compressive strain at maximum stress, $\epsilon_{\text{c1, dyn}}$. The contribution of the initially acting static loads $\epsilon_{\text{cv, stat}}$ is approximated as a function of the ratio between the corresponding shear force $V_{\text{E, stat}}$ and total shear force V_{E} . The time t_{V} required to attain V_{E} (Figure 5.4) is deduced from the SDOF analysis.

$$\dot{\varepsilon}_{cv,av} = \frac{\varepsilon_{cv} - \varepsilon_{cv,stat}}{t_v} \approx \frac{\varepsilon_{cv}(1 - \frac{V_{E,stat}}{V_E})}{t_v} \quad (5.20)$$

In an equivalent manner, the DIF_{fyws} corresponding to the yield strength of the transverse reinforcement involved in the resistance of the tension tie (4.21) close to the beam support cross-sections is inferred from average strain rate $\dot{\varepsilon}_{sv,av}$ (5.21). This strain rate is a function of the total strain ε_{sv} in the stirrups due to shear force V_E , given by Hook's law, $\varepsilon_{sv} = f_{yw,dyn} / E_s$ and contribution $\varepsilon_{sv,stat}$ of the gravitational loads.

$$\dot{\varepsilon}_{sv,av} = \frac{\varepsilon_{sv} - \varepsilon_{sv,stat}}{t_v} \approx \frac{\varepsilon_{sv}(1 - \frac{V_{E,stat}}{V_E})}{t_v} \quad (5.21)$$

5.3.3 Beams submitted to load arrangement $p_{EX(-)}$

Resistance function

In contrast to $p_{EX(+)}$, load arrangement $p_{EX(-)}$ (Figure 4.8) is characterized by an explosion load (stage 2) which *opposes* the gravitational loads (stage 1) acting on the beams previously to the accidental scenario. Consequently, the negative bending response mode of the beams under exposure to load-stage 2 is preceded by a positive bending response due to the static action effect $M_{E,stat}$ in consequence of load-stage 1 (Figure 5.3). The tensile force induced by moment $M_{E,stat}$ calls for a bottom reinforcement layer A_{s2} (according to the definition in Figure 4.10, A_{s1} refers to the tensile reinforcement layer under the explosion exposure, i.e. the top reinforcement layer in case of load arrangement $p_{EX(-)}$). For this purpose, the reinforcement obtained in the member design procedure for load arrangement $p_{EX(+)}$ will be adopted.

The structural response to *stage 1* can be described by the following two response functions (RF) of the simplified M - χ relationship at the mid-span cross-section, shown in the left part of Figure 5.9:

- RF I₍₊₎: uncracked state during positive bending response

$$M_R = \frac{M_{R,cr(+)}}{\chi_{cr(+)}} \cdot \chi \quad \text{if } \chi \leq \chi_{cr(+)} \quad (5.22)$$

- RF II₍₊₎: cracked state prior to yielding of the bottom reinforcement layer A_{s2} during positive bending response

$$M_R = M_{R,cr(+)} + \frac{M_{R,y(+)} - M_{R,cr(+)}}{\chi_{y(+)} - \chi_{cr(+)}} \cdot (\chi - \chi_{cr(+)}) \quad \text{if } \chi_{cr(+)} < \chi \leq \chi_{stat} \quad (5.23)$$

Within the preceding M - χ relationships, cracking moment $M_{R,cr(+)}$ is defined by (5.17) whereas the yield moment $M_{R,y(+)}$ and the corresponding yield curvature $\chi_{y(+)}$ are deduced from cross-sectional analysis under assumption of the bottom reinforcement layer A_{s2} as described above. Since the positive bending response is induced by the gravitational loads, the static material properties are used for the description of RF I₍₊₎ and II₍₊₎ and are hence exempt from the iterative updating procedure of the dynamic material characteristics (Figure 5.6).

The interception of RF II₍₊₎ at $M_R = M_{E,stat}$ delivers the point of origin of the structural response to the explosion load (*stage 2*). Four linear M - χ functions represent this response (Figure 5.9, left):

- RF 0: decompression of the cross-section

$$M_R = \frac{M_{E,stat}}{\chi_{stat}} \cdot \chi \quad \text{if } 0 < \chi \leq \chi_{stat} \quad (5.24)$$

- RF I₍₋₎: uncracked state during negative bending response

$$M_R = \frac{M_{R,cr(-)}}{\chi_{cr(-)}} \cdot \chi \quad \text{if } \chi_{cr(-)} < \chi \leq 0 \quad (5.25)$$

- RF II₍₋₎: cracked state prior to yielding of top reinforcement layer A_{s1} during negative bending

$$M_R = M_{R,cr(-)} + \frac{M_{R,y(-)} - M_{R,cr(-)}}{\chi_{y(-)} - \chi_{cr(-)}} \cdot (\chi - \chi_{cr(-)}) \quad \text{if } \chi_{y(-)} < \chi \leq \chi_{cr(-)} \quad (5.26)$$

- RF III₍₋₎: post-yield state during negative bending response

$$M_R = M_{R,y(-)} + \frac{M_{R,u(-)} - M_{R,y(-)}}{\chi_{u(-)} - \chi_{y(-)}} \cdot (\chi - \chi_{y(-)}) \quad \text{if } \chi_{u(-)} < \chi \leq \chi_{y(-)} \quad (5.27)$$

Response function 0 describes the member response from the initiation of the explosion event at $t = t_0$ to the moment of the complete decompression of the cross-section at $t = t_{dec}$ (Figure 5.3). It is assumed that at the end of this stage, the initial curvature χ_{stat} will have been reduced to zero. In the following, the negative bending response stage is entered with initially uncracked concrete in tension, until, at the end of RF I₍₋₎, cracking moment $M_{R,cr(-)}$ is reached. The dynamic concrete properties ($f_{ct,dyn}$ and $E_{c,dyn}$) are employed to deduce $M_{R,cr(-)}$ and the corresponding curvature $\chi_{cr(-)}$ for all iteration steps $i > 1$ (Figure 5.6).

The transition from RF I₍₋₎ to II₍₋₎ is characterized by a decrease of the cross-sectional stiffness EI . Response RF II₍₋₎ concludes with the initiation of plastic deformations of the longitudinal reinforcement upon reaching the yield curvature $\chi_{y(-)}$. Both the yield- and the ultimate curvature ($\chi_{u(-)}$) are deduced from cross-sectional analysis under use of the relevant dynamic properties of concrete ($f_{ct,dyn}$, $E_{c,dyn}$, $f_{c,dyn}$, $\epsilon_{c1,dyn}$) and steel ($f_{y,dyn}$ and $f_{t,dyn}$) for all iteration steps $i > 1$ (Figure 5.6). Response functions II₍₋₎ and III₍₋₎ are thereby fully described and the curvature ductility deduced from (5.9) where χ_y and χ_u are replaced by $\chi_{y(-)}$ and $\chi_{u(-)}$, respectively. Based on the established M - χ diagram (RF 0 to RF III₍₋₎), the resistance function $R(\delta)$ (Figure 5.9, right) is subsequently derived as described under section 5.2.

Figure 5.9 Idealised moment-curvature (left) and resistance-displacement (right) functions assumed for the mid-span cross-section of the beams subjected to load arrangement p_{EX(-)}

For sake of a simplicity, the derived, multilinear R - δ function is then converted to a strain energy-equivalent bilinear function, shown in the right part of Figure 5.10. Both the point of origin (δ_{stat} , R_{stat}) and the failure point ($\delta_{u(-)}$, $R_{u(-)}$) of this bilinear function are adopted from the multilinear graph (Figure

5.10, left). The equivalent yield point ($\delta_{y(-),eq}$, $R_{y(-),eq}$) is determined so that the dissipated strain energy (red coloured area $W_{s,BL}$) is equal as under the multilinear approach (blue coloured area $W_{s,ML}$).

The effective resistances $R_{y,dyn,eq}$ and $R_{u,dyn,eq}$ of the bilinear function are then obtained from (5.11) and (5.12), respectively, where $R_y = R_{y(-),eq}$, $R_u = R_{u(-)}$ and $R_{stat} < 0$. Moreover, this function is characterized by an effective stiffness $k_{dyn,eq}$ prior to yielding (Figure 5.10, right) and an effective displacement ductility $\mu_{\delta,dyn,eq}$ given by (5.13) where $\delta_y = \delta_{y(-),eq}$, $\delta_u = \delta_{u(-)}$.

Figure 5.10 Idealised multilinear (left) and equivalent, bilinear (right) resistance-displacement functions assumed for the mid-span cross-section of the beams subjected to load arrangement $p_{EX(-)}$

Average strain rates

The dynamic increase factors $DIF_{f_{ct}}$ and DIF_{E_c} for the determination of the dynamic concrete tensile strength $f_{ct,dyn}$ and the elastic modulus $E_{c,dyn}$ (Figure 5.11) are based on the average strain rate $\dot{\epsilon}_{ct,av}$ given by (5.28).

$$\dot{\epsilon}_{ct,av} = \frac{\epsilon_{ct}}{t_{cr} - t_{dec}} \quad (5.28)$$

Strain ϵ_{ct} corresponds the most loaded concrete cross-section fiber in tension, induced by the dynamic load at time t_{cr} when cracking occurs. Keeping in mind that this fiber is initially under compression until $t = t_{dec}$ (Figure 5.3), the effective time span for ϵ_{ct} to be reached is given by $t_{cr} - t_{dec}$. Both t_{cr} and t_{dec} are deduced from the SDOF analysis.

Figure 5.11 Stress-strain relation for concrete in tension under static (black) and dynamic (red) conditions

The dynamic increase of the concrete compressive strength $f_{c,dyn}$ and the compressive strain at maximum stress $\epsilon_{c1,dyn}$ is determined from the average strain rate $\dot{\epsilon}_{c,av}$ (5.29). This strain rate is defined by

the ratio of the total compressive strain $\varepsilon_{c,y}$ in the most loaded concrete cross-section fiber at time $t = t_y$ to the effective time span $(t_y - t_{dec})$, which considers that this fiber is initially exposed to tensile stress. In a similar vein, equation (5.30) defines the average strain rate for estimation of the dynamic increase factors for the reinforcing steel yield- and tensile strength.

$$\dot{\varepsilon}_{c,av} = \frac{\varepsilon_{c,y}}{t_y - t_{dec}} \quad (5.29)$$

$$\dot{\varepsilon}_{s,av} = \frac{\varepsilon_{s1,y}}{t_y - t_{dec}} \quad (5.30)$$

Figure 5.12 Strain distribution in the mid-span cross-section due to initially acting static loads at $t = t_0$ (stage 1), and total strain distribution at $t = t_y$

5.3.4 Columns

Resistance function

As previously described in section 4.5, load arrangement $N_{EX(-)}$ considered for the columns is characterized by two subsequent load stages (Figure 4.9). In *stage 1*, the gravitational loads impose an eccentric axial compression force in the columns, whereas in *stage 2* the explosion-induced actions reduce this axial force under simultaneous increase of the bending moment. The development of the resistance function, which adequately describes the structural response of the columns under these particular load assumptions, is far more complex than in case of the beams. In particular, the following problems must be dealt with:

- The member failure mode is unknown. Depending on the load- and resistance characteristics in each case, members might fail due to exceedance of the UMLS or the BLS (Figure 4.12).
- The location of the failure point ($M_E=M_R$, $N_E=N_R$) on the M-N interaction diagram (Figure 4.11, Figure 4.12) is unknown.
- The moment-curvature relationship, based on which the resistance function is to be developed, depends on the (variable) axial force level.

The determination of the relevant failure mode and the corresponding failure point calls for a numerical solution algorithm, which accounts for the axial force-dependent formulation of action effects and resistance under consideration of both the geometrical and material non-linearity involved. Within this algorithm, the explosion pressure and the corresponding statically equivalent load-effects are stepwise incremented and the corresponding structural response computed accordingly, until member failure. The algorithm is described below.

The first step of the numerical solution procedure ($s=1$), describes the initial situation under the influence of the gravitational loads (load stage 1). The eccentric action of the corresponding axial compression force $N_{E,s=1}$ (5.31) causes a bending moment $M_{E,s=1}$ (5.32), with a second order eccentricity $e_{II,s=1}$ inferred from curvature $\chi_{s=1} = \chi_{stat}$ under the assumption of a sinusoidal curvature distribution along the member length, l_0 (Figure 5.13).

$$N_{E,s=1} = N_{E,stat} \quad (5.31)$$

$$M_{E,s=1} = N_{E,s=1} \cdot (e_{min} + e_{II,s=1}) = N_{E,s=1} \cdot (e_{min} + 0.1 \cdot l_0^2 \cdot \chi_{s=1}) \quad (5.32)$$

Curvature $\chi_{s=1}$ can be found from the equilibrium condition (5.33) where resistance moment $M_{R,s=1}$ is given by (5.34) or (5.35), depending on whether $\chi_{s=1} > \text{or} < \chi_{cr}$. In the former case, represented in Figure 5.13, $M_{R,s}$ is function of the yield moment $M_{R,y,s}$ and the corresponding yield curvature $\chi_{y,s}$, obtained from cross-sectional analysis under consideration of axial force $N_{E,s=1}$ (5.31). The static material properties are used for the derivation of $M_{R,s=1}$, which is hence exempt from the iterative updating of the dynamic material properties (Figure 5.6). With $\chi_{s=1}$ being determined, the point of origin of the M - χ function describing the structural response to the explosion event is defined.

$$M_{E,s=1} = M_{R,s=1} \quad (5.33)$$

$$M_{R,s} = \frac{M_{R,cr}}{\chi_{cr}} \cdot \chi_s \quad \text{if } \chi_s \leq \chi_{cr} \quad (5.34)$$

$$M_{R,s} = M_{R,cr} + \frac{M_{R,y,s} - M_{R,cr}}{\chi_{y,s} - \chi_{cr}} \cdot (\chi_s - \chi_{cr}) \quad \text{if } \chi_s > \chi_{cr} \quad (5.35)$$

Figure 5.13 Idealised moment-curvature function for the intermediate cross-section of the columns and corresponding action effects (blue line) – step $s=1$ (gravitational loads)

In the following steps ($s=2$ to n) of the solution algorithm, the explosion pressure is being progressively increased. Each pressure increment entails an increment $\Delta N_{EX,s}$ of the tensile axial force component N_{EX} and consequently a reduction $\Delta N_{E,s} (= \Delta N_{EX,s})$ of the resulting axial compression force N_E with respect to the previous load step $s-1$ (5.36). Moreover, bending moment M_E is being incremented by $\Delta M_{E,s}$ (5.37).

$$N_{E,s} = N_{E,s-1} - \Delta N_{E,s} \quad (5.36)$$

$$M_{E,s} = M_{E,s-1} + \Delta M_{E,s} \quad (5.37)$$

Increment $\Delta M_{E,s}$ (5.38) is composed by a first order and second order term, respectively, $\Delta M_{E,I,s}$ and $\Delta M_{E,II,s}$ (Figure 5.14). The former (5.39), based on (4.35), expresses the difference between increment

$\Delta M_{EX,s}$ of moment M_{EX} due to the horizontal explosion-load component $\tilde{p}_{EX,h}$ and increment $\Delta M_{N,EX,s}$ of moment $M_{N,EX}$ due to the eccentricity (e_{min}) of the dynamic axial force $N_{E,dyn} = N_{EX}$. The latter (5.40), based on (4.36), accounts for changes in the deformed state of the columns. Formally it represents the difference between second order moments ($N_E \cdot \delta$) corresponding to incremental steps s and $s-1$, respectively, expressed as a function of the corresponding curvatures χ_s and χ_{s-1} at the intermediate cross-section of the columns.

$$\Delta M_{E,s} = \Delta M_{E,I,s} + \Delta M_{E,II,s} \quad (5.38)$$

$$\Delta M_{E,I,s} = \Delta M_{EX,s} - \Delta M_{N,EX,s} \quad (5.39)$$

$$\Delta M_{E,II,s} = N_{E,s} \cdot \delta_s - N_{E,s-1} \cdot \delta_{s-1} = 0.1 \cdot l_0^2 \cdot (N_{E,s} \cdot \chi_s - N_{E,s-1} \cdot \chi_{s-1}) \quad (5.40)$$

Curvature χ_s is finally deduced from condition (5.41) where resistance $M_{R,s}$ is given by (5.34) or (5.35), depending on whether $\chi_s > \text{or} < \chi_{cr}$. In the former case, represented in Figure 5.14 (for $s=2$), $M_{R,s}$ is function of the yield moment $M_{R,y,s}$ and the corresponding curvature $\chi_{y,s}$. Both $M_{R,y,s}$ and $\chi_{y,s}$ are obtained from cross-sectional analysis under consideration of axial force $N_{E,s}$ (5.36) and under use of the relevant dynamic properties of concrete ($f_{ct,dyn}$, $E_{c,dyn}$, $f_{c,dyn}$, $\epsilon_{c1,dyn}$) and steel ($f_{y,dyn}$ and $f_{t,dyn}$) for all steps $i > 1$ of the established iteration procedure (Figure 5.6).

$$M_{E,s} = M_{R,s} \quad (5.41)$$

Figure 5.14 Development of the idealised moment-curvature function for the intermediate cross-section of the columns and corresponding action effects (blue+red lines) – step $s = 2$.

The successive increase of the explosion pressure and the corresponding statically equivalent action effects delivers, through application of equations (5.36) to (5.41), the points of equilibrium (blue dots in Figure 5.14) which characterize the pre-yield response stage of the member in question. This stage concludes when curvature χ_s equals the yield curvature ($\chi_s = \chi_{y,s}$). At this point, where the yield strength of the tensile- or compression reinforcement layer is being reached, the member might buckle in case the second order effects would be still sufficiently high, despite the reduction of the compression axial force the column has experienced up to that point. As explained in section 4.6.3 (Figure 4.12), the corresponding limit state (BLS) is attained when, upon reaching $\chi_{y,s}$, the step-function representing the action effects $M_{E,s}(\chi)$ is tangent to the moment-curvature diagram $M_{R,s}(\chi)$, as shown in Figure 5.15, left (for $s=3$). This situation, where any further load increase would imply an immediate stability loss of the member, is formally described by condition (5.42), which expresses that the slope $m_{II,s}$ of $M_{E,s}(\chi)$ at $\chi_s =$

$\chi_{y,s}$ exceeds slope $m_{R,s}$ of the plastic response stage of $M_{R,s}(\chi)$. Hence, members for which the inequality (5.42) is fulfilled, are considered buckling-critical. On the contrary, the member will not buckle upon initiation of reinforcement yielding and the plastic response stage will be entered, as represented in Figure 5.15, right (for $s = 3$). In this stage, resistance $M_{R,s}$ is defined by (5.43), as a function of the moments and curvatures at yield- and ultimate failure, inferred from cross-sectional analysis under influence of axial force $N_{E,s}$ (5.36). Failure will arise, after further load increase, in consequence of excessive cross-sectional deformations, i.e. exceedance of the UMLS beyond $M_{E,s} = M_{R,u,s}$.

$$m_{II,s} = \frac{\Delta M_{E,II,s}}{\chi_s - \chi_{s-1}} > m_{R,s} = \frac{M_{R,u,s} - M_{R,y,s}}{\chi_{u,s} - \chi_{y,s}} \quad \text{and } \chi_s \approx \chi_{y,s} \quad (5.42)$$

$$M_{R,s} = M_{R,y} + \frac{M_{R,u,s} - M_{R,y,s}}{\chi_{u,s} - \chi_{y,s}} \cdot (\chi_s - \chi_{y,s}) \quad \text{if } \chi_s > \chi_y \quad (5.43)$$

It should be noted that the presented algorithm is computed with a variable load step-size Δs . While the first steps are kept comparatively large, Δs is reduced when approaching the yield curvature $\chi_{y,s}$ in order to assure that $\chi_s \approx \chi_{y,s}$ with an accurate precision when checking the inequality (5.42). In Annex D, the algorithm is checked for its correct functioning and implementation by means of a case study under the assumption of simplified loading conditions.

Figure 5.15 Development of the idealised moment-curvature function for the intermediate cross-section of the columns and corresponding action effects (blue+red lines) – step $s = 3$: BLS (left) and UMLS (right).

A linear connection of the points of equilibrium (blue dots) found for each of the load steps s delivers the multilinear M - χ relationship which represents the structural response of the member to the imposed (statically equivalent) load effects. Subsequently, the multilinear M - χ diagram (Figure 5.16, left) is being converted to an effective, dynamic resistance function $R_{dyn}(\delta)$ taking account of the static load level immediately prior to the explosion (section 5.2). A simplified, bilinear function is inferred (Figure 5.16, right), characterised by effective resistances $R_{y,dyn}$ and $R_{u,dyn}$ (the latter is relevant only for UMLS members), an effective stiffness prior to yielding, k_{dyn} and the effective ductility ratio $\mu_{\delta,dyn}$ (5.13) (in case of BLS members $\mu_{\delta,dyn} = 1.0$). Effective resistances $R_{y,dyn}$ and $R_{u,dyn}$ are deduced from (5.11) and (5.12) as a function of resistance R_{stat} , required to withstand the initially present gravitational loads. Stiffness k_{dyn} represents the slope of the pre-yield response function, defined by a linear connection between the point of origin (R_{stat} , δ_{stat}) and the yield point (R_y , δ_y), secant to the R - δ diagram (Figure 5.16, right). Conducted tests showed that a strain energy-based inference of an equivalent stiffness $k_{dyn,eq}$, in a similar vein as done for the beams under load arrangement $p_{EX(-)}$ (section 5.3.3), would not entail significant changes with respect to the adopted secant approach.

Figure 5.16 Multilinear moment-curvature (left) and resistance-displacement (right) function for the intermediate cross-section of the columns (UMLS members)

The effective resistance function can then be employed to determine the dynamic load- and increase factors following the established, iterative analysis procedure (Figure 5.6). It should be observed that the dynamic load factor DLF_M (5.7), obtained after each iteration step i of this procedure, will entail an alteration of the first-order term of the statically equivalent bending moment, $M_{E,i}$. The corresponding increments $\Delta M_{E,i,s}$ (5.39) are to be updated accordingly. On the other hand, since $DLF_N = 1.0$ (section 5.1.2), no such updating of the axial force increment $\Delta N_{E,s}$ is necessary.

Average strain rates

Although this applies to the minority of the studied members, the initial moment due to the gravitational loads acting prior to the explosion event, $M_{E,stat}$, might not exceed the cracking moment, $M_{R,cr}$. In these cases, concrete cracking is supposed to occur under dynamic loading conditions. Hence, dynamic increase factors (DIF) for the material parameters characterizing cracking are to be derived, i.e., the concrete tensile strength f_{ct} and the concrete modulus E_c . The corresponding DIF ($DIF_{f_{ct}}$, DIF_{E_c}) are based on the average strain rate $\dot{\epsilon}_{ct,av}$ given by (5.44). Strain ϵ_{ct} corresponds to the tensile stress-state in the most loaded concrete cross-section fiber, induced by the dynamic load at time t_{cr} , when the concrete tensile strength is reached. Strain component $\epsilon_{ct,stat}$ is approximated as a function of the ratio between $M_{E,stat}$ and $M_{R,cr}$. Time t_{cr} is deduced from the SDOF-analysis.

$$\dot{\epsilon}_{ct,av} = \frac{\epsilon_{ct} - \epsilon_{ct,stat}}{t_{cr}} = \frac{\epsilon_{ct}(1 - \frac{M_{E,stat}}{M_{R,cr}})}{t_{cr}} \quad (5.44)$$

The determination of the structural response in the pre- and post-yield stage requires dynamic increase factors DIF_{fc} and $DIF_{\epsilon_{c1}}$ for the dynamic concrete compressive strength $f_{c,dyn}$ and the compressive strain at maximum stress $\epsilon_{c1,dyn}$ (Figure 4.24). Moreover DIF_{fy} and DIF_{ft} for the yield- and tensile strength of the reinforcing steel layers A_{s1} and A_{s2} (Figure 4.10) are needed. The corresponding average strain rates $\dot{\epsilon}_{c,av}$ and $\dot{\epsilon}_{s,av}$ are given by (5.18) and (5.19), respectively, introduced before for the beams. In Annex E, the herein used DIF equations for f_c , ϵ_{c1} , f_y and f_t are applied to formulate the resistance of a column tested under dynamic exposure of interacting bending moment and axial force [191]. It is found that the resistance of the column can be accurately reproduced.

The dynamic increase of the concrete compressive strength and the yield strength of the transverse reinforcement involved in the shear resistance of, respectively, the compression strut and the tension

tie at the beam's support cross-section, is also computed in an analog manner to the beams. Equations (5.20) and (5.21) apply for estimation of the corresponding average strain rates, where $V_{E,stat} = 0$.

5.4 Results

5.4.1 Introduction

The defined RC beams and columns are submitted to the dynamic analysis procedure described in section 5.2 under use of the corresponding resistance function and the average strain rates defined in section 5.3. The results for the beams submitted to load arrangements $p_{EX(+)}$ and $p_{EX(-)}$ (Figure 4.8) are presented in sections 5.4.2 and 5.4.3, respectively. Section 5.4.4 contains the most relevant results obtained for the 28512 columns submitted to load arrangement $N_{EX(-)}$ (Figure 4.9).

5.4.2 Beams submitted to load arrangement $p_{EX(+)}$

Dynamic load factors

As a measure for their deformation capacity, Figure 5.17 (left) presents the displacement ductility ratios μ_δ (5.10) corresponding to the 486 RC beams subjected to load arrangement $p_{EX(+)}$. The Figure shows that μ_δ strongly depends on the tensile reinforcement index ω ($\omega = \rho_l \cdot f_{yk}/f_{ck}$). In line with the findings from other studies, e.g. [192], the plastic deformation capacity of the beams experiences a decay with increasing ω . The tendency line included in the figure suggests that this decay follows a power-law function. The minimum ductility ratio $\mu_\delta \approx 2$ is associated with reinforcement indices ω larger than 0.2. On the contrary, the maximum μ_δ of nearly 8 corresponds to a ω below 0.05.

Figure 5.17 Displacement ductility ratio μ_δ versus tensile reinforcement index ω (left); Ratio between μ_δ and $\mu_{\delta,dyn}$ versus explosion-load ratio v_{acc} (4.62) (right); 486 beams subjected to load-arrangement $p_{EX(+)}$

The displacement ductility ratio μ_δ (5.10) is related in Figure 5.17 (right) to the effective ductility ratio $\mu_{\delta,dyn}$ (5.13), which takes account of the initially present gravitational loads. Ratio $\mu_\delta / \mu_{\delta,dyn}$ is shown as a function of the accidental load ratio v_{acc} (4.62), which represents the ratio between the blast load and the total loads acting on the members. It can be observed that the higher the gravitational loads, represented by smaller v_{acc} , the higher is the effective ductility ratio $\mu_{\delta,dyn}$ with respect to μ_δ . The point is that higher gravitational load R_{stat} levels cause a shorter elastic response stage ($\delta < \delta_y$) in relation to the plastic range ($\delta_y \leq \delta < \delta_u$) and consequently a larger effective ductility $\mu_{\delta,dyn}$ in comparison to μ_δ (Figure 5.7).

In line with the explanations in section 3.3.3 (Figure 3.11), the dynamic load factors DLF_M corresponding to the flexural failure mode of the beams diminish with increasing deformation capacity of the members. This can be drawn from Figure 5.18 (left), where the obtained DLF_M (red dots) are shown as a function

of the effective displacement ductility ratio $\mu_{\delta,dyn}$ (5.13). The results manifest that the required resistance to face the applied blast load decreases as the deformation capacity increases. From an alternative point of view, it might be stated that the resisted explosion pressure increases with rising deformation capacity. In any case, the results clearly evidence the beneficial effect of energy dissipation through plastic deformations. The maximum DLF_M of 0.96 corresponds to a $\mu_{\delta,dyn}$ close to 2 whereas the minimum DLF_M of 0.48 is obtained for an effective ductility ratio larger than 7.

The ductile flexural response mode can only be developed if the members are able to resist the dynamic shear forces that arise concomitantly close to the member supports. The green dots reflected in Figure 5.18 (left) indicate that this is highly unlikely however. They represent the maximum dynamic load factors DLF_M that are obtained if the peak pressure applied to the beams would not exceed the value the member may sustain without undergoing a tension-controlled shear failure scenario at the member support, characterized by failure of the stirrups. As indicated by the comparatively higher DLF_M , this peak pressure is significantly smaller than in case of the flexural failure mode, represented by the red dots. The comparatively smaller $\mu_{\delta,dyn}$ ratios point to the fact that the energy dissipation capacity of the members is not fully taken account of at the time of shear failure. This is being confirmed in the right part of Figure 5.18 which compares the strain energy W_s dissipated during the explosion event until the point of bending failure ($W_{s,M}$) and tension tie shear failure ($W_{s,Vs}$), respectively. The represented tendency line shows that at the time of shear failure only about 1/3 of the energy dissipation capacity corresponding to the flexural failure mode is taken advantage of.

Figure 5.18 Dynamic load factor DLF_M for flexural (red) and tension tie shear failure mode (green) versus effective displacement ductility $\mu_{\delta,dyn}$ (left); Dissipated strain energy W_s : Tension tie shear failure mode, $W_{s,Vs}$ vs. flexural failure mode, $W_{s,M}$ (right); 486 beams strictly designed for acc. load arrangement $p_{EX(+)}$

The DLF_M shown in Figure 5.18 are again plotted in Figure 5.19 (left) as a function of the ratio between the load-pulse duration t_p and the member natural period T (3.27). It can be observed that the beams perform in the dynamic regime (section 3.3.3). The mayor discrepancies between the represented DLF_M (red and green dots), and hence the sustained explosion peak pressure until member failure in the, respectively, flexural- and shear mode, arise at non-dimensional time ratios t_p/T around 1.0. Towards the quasi-static regime (section 3.3.3) these discrepancies logically vanish.

The fact that the resisted peak pressure until occurrence of a tension-induced shear failure is comparatively small is also being reflected in relatively high dynamic load factors DLF_{Vs} , defined according to (5.4). This can be drawn from Figure 5.19 (right), where the mentioned DLF_{Vs} are plotted as a function of ratio t_p/T (green dots). The maximum DLF_{Vs} of about 1.3 indicates that the required resistance to

sustain the dynamic shear force may be up to 30% higher than the shear force due to the statically applied peak pressure. In the most favorable case, corresponding to a relatively small t_p/T ratio of about 0.6, the required resistance amounts to approximately 70% of this force ($DLF_{Vs} \approx 0.7$).

Figure 5.19 Dynamic load factor DLF_M for flexural (red) and tension tie shear failure mode (green) versus non-dimensional time-ratio t_p/T (left); Dynamic load factor DLF_V for tension- (green) and compression- (blue) shear failure mode versus t_p/T (right); 486 beams strictly designed for acc. load arrangement $p_{EX(+)}$

In comparison to the tension-induced shear failure, a shear failure associated with crushing of the compression strut seems to be highly unlikely to occur. Precisely speaking, it is found that the peak pressure the members are able to sustain until this type of failure occurs largely exceeds the corresponding value associated with the tension-controlled shear mode. Likewise, it amply exceeds the sustained peak pressure of the members performing in the ductile, flexural mode. In other words, in order to attain the compression-controlled shear force resistance, the member would have to undergo displacements far beyond the ultimate value δ_u associated with flexural failure. Since this is not reasonable, the expected dynamic shear forces corresponding to the compression-controlled shear failure mode are represented by the maximum member reaction forces that arise concomitantly to the flexural response mode. The corresponding DLF_{Vc} are shown in the right part of Figure 5.19 (blue dots). It can be observed that the DLF_{Vc} are substantially smaller than the DLF_{Vs} values, corresponding to the tension-induced shear failure (green dots), especially when t_p/T ratios around unity are concerned. The minimum DLF_{Vc} of 0.55, corresponding to a t_p/T ratio of about 0.6, represents a dynamic shear force which amounts to 55% of the shear force resulting from the statically applied explosion peak-load.

The minimum, mean and maximum dynamic load factors of the 486 beams designed according to the *accidental situation* are summarized in Table 5.2. It must be recalled at that point that this design situation is not mandatory in all 486 cases however. As explained in section 4.7 (Table 4.3), according to the hypothesis adopted in the present study, the design is to be based on the *persistent situation* in 273 out of the 486 members, since in these cases the normal load conditions demand a higher resistance than the accidental load scenario. A higher resistance, in turn, is reflected in a comparatively lower member deformation capacity and, hence, in a reduced energy dissipation during the dynamic response under the explosion load scenario. Due to this reason, the DLF's for members strictly designed to the persistent situation are comparatively higher, as can be inferred from Table 5.2. The indicated minimum, mean and maximum values for the 486 beams designed according to the *mandatory situation* (213 results for the *accidental situation* and 273 results for the *persistent situation*) exceed the values obtained if exclusively the accidental situation is considered in the design stage.

Table 5.2: Statistical evaluation of dynamic load factors (minimum, mean, maximum value and coefficient of variation CoV) for 486 beams designed strictly according to accidental and mandatory situation.

Failure mode	Bending		Shear (tension)		Shear (compression)	
Dynamic load factor	DLF _M		DLF _{Vs}		DLF _{Vc} *	
Design situation	<i>Accidental</i>	<i>Mandatory</i>	<i>Accidental</i>	<i>Mandatory</i>	<i>Accidental</i>	<i>Mandatory</i>
Min	0.48	0.59	0.69	0.75	0.55	0.64
Mean	0.79	0.86	1.03	1.09	0.78	0.84
Max	0.96	1.05	1.29	1.31	0.94	0.98

* Concomitant to bending failure

Dynamic increase factors

Table 5.3 shows the mean, maximum and minimum dynamic increase factors (DIF) involved in the bending- (M) and shear- (V) response of the beams subjected to load arrangement $p_{EX(+)}$. It is shown that the concrete compressive strength f_c experiences the comparatively largest dynamic increase with DIF's up to 1.23. Such high DIF's correspond to concretes of lower strength class ($f_{ck} = 25 \text{ N/mm}^2$), as exposed in section 3.2.2. The mean increase of f_c is about 13-15%, only slightly above the mean value for the concrete compressive strain ϵ_{c1} (11-12%) and the reinforcing steel yield strength f_y or f_{yw} (10%). The smallest dynamic increase corresponds to the tensile strength of the reinforcing steel f_t with a mean value of about 3%.

Table 5.3: Dynamic increase factors (DIF) for 486 beams submitted to load arrangement $p_{EX(+)}$

Failure mode	DIF	Mean	Max	Min
M	f_c	1.13	1.22	1.07
	ϵ_{c1}	1.11	1.13	1.09
	f_y	1.10	1.12	1.08
	f_t	1.03	1.04	1.03
V	f_c	1.15	1.23	1.09
	ϵ_{c1}	1.12	1.14	1.11
	f_{yw}	1.10	1.12	1.07

Final observations

In relation to the presented results, it is interesting to recall that the mid-span and the support cross-sections of the beams have been *strictly* designed, respectively for bending and shear, to resist the nominal explosion pressure specified in the applied code (section 4.7). In the flexural design, it was envisaged that the mid-span cross-section of the beams behaves in a ductile manner, by limiting the neutral axis depth x/d (4.50). Nevertheless, according to the results presented above, it seems likely that the beams, as designed, will fail in the more brittle shear (tension tie) mode, before the full flexural response might be developed. This finding is consistent with the frequently quoted observation that structural members, when exposed to blast loads, might fail in shear, even if they were designed to exhibit a ductile, flexural behavior under quasi-static loading conditions (section 3.2.3). In order to perform in the desired ductile flexural mode, the beams would require either a higher shear resistance (for instance by providing a higher transverse reinforcement degree), or a lower bending resistance and/or stiffness, what constitutes a parallelism to the capacity design approach for seismic actions. The latter option could be achieved by considering the beneficial effect of energy dissipation explicitly in the flexural design stage

of explosion-exposed members, for instance by introducing dynamic load factors such as those presented before*. In addition, the current definition of the design material properties for accidental situations should be analyzed in detail, as do suggest the findings presented in Table 5.4. The table shows the minimum, mean and maximum values of the ratios between the structural resistance of the members based on, respectively, mean (including strain-rate effects) and design values. The ratios for bending (M)-, tension-induced shear (V_s)- and compression-controlled shear- (V_c) failure are distinguished. As can be observed, the resisting bending moments based on mean values, M_{Rm} , exceed the design moments M_{Rd} between 1.29 and 1.38 times. On the contrary, the corresponding ratios for the tension-induced shear failure mode, $V_{Rm,s}/V_{Rd,s}$ oscillate between far lower boundaries of 1.20 and 1.25, whereas in case of the compression-controlled shear failure the ratios ($V_{Rm,c}/V_{Rd,c}$) are significantly higher as denoted by lower and upper bounds of 1.75 and 1.98, respectively.

Table 5.4: Minimum, mean and maximum values of ratios between mean (including strain rate effects) and design resistance of 486 RC beams designed for load arrangement pEX(+)

Ratio	M_{Rm} / M_{Rd}	$V_{Rm,s} / V_{Rd,s}$	$V_{Rm,c} / V_{Rd,c}$
Min	1.29	1.20	1.75
Mean	1.34	1.23	1.85
Max	1.38	1.25	1.98

The mentioned discrepancies, which are indicative of the above detected proneness of the beams to fail in tension-induced shear, on one hand, and their highly unlikely failure in compression-controlled shear, on the other, can be traced back to different circumstances. Among them, the following two should be highlighted here: the partial safety factors applied in the member design for concrete and reinforcing steel, and the strain rate effects. In line with the current approach in the codes, the latter have not been explicitly accounted for in the design stage. As the mentioned results show, this might not only contribute to an underestimation of structural resistance, but also to an undesired, brittle shear failure mode, characterized by a reduced energy absorption capacity, and consequently, to a non-efficient allocation of structural resources to face the explosion-induced loads. In this context, it is also suggested to analyze in detail to which extent the current partial safety factors for concrete and steel, to be employed in the accidental design situation, are based on rational grounds. The fact that there seems to be no clear evidence that the scatter associated with the dynamic resistance of concrete is significantly different from concrete under static loading [128], and the fact that model uncertainties associated with dynamic load effects are generally higher than under static conditions, do certainly not deliver a rational basis which justifies the reduction of the partial factor for concrete γ_c from 1.5, in persistent situations, to 1.3 [72], or 1.2 [84], in accidental design scenarios. In case this reduction would have been introduced to account indirectly for enhanced material resistance under dynamic loading conditions, i.e. due to strain rate effects, it seems to be questionable if this would be generally the appropriate way to do so. Besides from the fact that in this case a unified reduction of γ_c , independent of the dynamic load type, would not seem to be reasonable from a physical point of view, due to fundamentally different orders of magnitude the material strain rates might attain in specific accidental situations involving different types of dynamic load exposures (Figure 3.5). The introduction of material- and load-specific, dynamic increase factors in the design codes would seem to be a more rational approach in this regard.

* Clause 5.3 (3) in the current Eurocode for accidental actions [17] states that gas explosions might involve peak pressures which are higher than the nominal static equivalent pressure p_d according to Annex D.2 of the mentioned code, adopted for member design in the present study. It further states that such peak pressures should be considered in the context of a maximum load duration of 0.2 s and plastic material deformations. No model is included in [17] for estimation of such peak pressures, however. In [30], where a possible practical implementation of the quoted clause is illustrated, the nominal pressure p_d has been adopted as a reasonable approximation.

In summary, the previously discussed results point to certain scope for improving the current design rules for accidental situations in view of a more consistent and rational treatment of structural safety. The development of a corresponding, explicit proposal is out of scope of the present thesis. In any case, such a proposal should be preferably based on the results of a consistent risk analysis including both probabilities of explosion-induced failure scenarios and their consequences, what is the purpose of chapters 6 to 8 of the present thesis.

5.4.3 Beams submitted to load arrangement $p_{EX(-)}$

Dynamic load factors

As shown before in the context of the beams exposed to load arrangement $p_{EX(+)}$, the displacement ductility ratios μ_δ for the 136 beams strictly designed to resist load arrangement $p_{EX(-)}$ are found to decrease with rising tensile reinforcement index ω , as shown in Figure 5.20 (left). However, unlike observed for load arrangement $p_{EX(+)}$, the $\mu_\delta - \omega$ relation shown here is subject to a considerable scatter, in particular for small tensile reinforcement indices ω . The unusually small ω ($\omega_{\min} \approx 0.01$), which are a consequence of the opposing directions of gravitational- and explosion design loads and the correspondingly small design action effect M_{Ed} , cause displacement ductility's μ_δ far larger than found for the beams under $p_{EX(+)}$, with maximum values of $\mu_\delta \approx 19$.

Figure 5.20 Displacement ductility ratio μ_δ (multilinear resistance function) versus tensile reinforcement index ω (left); Inverse of ratio between displacement ductility μ_δ and effective displacement ductility $\mu_{\delta, dyn}$ versus inverse of explosion-load parameter v_{acc} ; 136 beams strictly designed for acc. load arrangement $p_{EX(-)}$

In section 5.4.2 it was shown that the effective ductility ratio $\mu_{\delta, dyn}$ (5.13) of the beams under load arrangement $p_{EX(+)}$ depends significantly on the relation v_{acc} (4.62) between the explosion load and the total loads. This is being confirmed in Figure 5.20 (right) for the members exposed to load configuration $p_{EX(-)}$. With increasing influence of the gravitational loads, represented by decreasing ratios v_{acc}^{-1} (exponent -1 is introduced in Figure 5.20 since under configuration $p_{EX(-)}$ the explosion load (EX) exceeds the total loads (EX-G)), the effective ductility ratio $\mu_{\delta, dyn}$ diminishes in comparison to the “potential” ductility ratio μ_δ . In contrast to load arrangement $p_{EX(+)}$, the counteracting effect of the gravitational loads entails pronounced elastic deformations ($\delta < \delta_y$) during the explosion-induced member response and hence a reduction of the effective ductility ratio $\mu_{\delta, dyn}$. In the most unfavorable cases ($v_{acc}^{-1} \approx 0$), $\mu_{\delta, dyn}$ approaches only 70% of μ_δ .

In the following, Figure 5.21 (left) compares the effective ductility ratios corresponding to the multilinear and the equivalent, bilinear resistance function (Figure 5.10), respectively $\mu_{\delta, dyn}$ and $\mu_{\delta, dyn, eq}$. It is ev-

identified, that the former is generally smaller than the latter. The regression line suggests that, as a general tendency, the higher is $\mu_{\delta,dyn}$ the larger is the difference to $\mu_{\delta,dyn,eq}$. In any case, it should be kept in mind that $\mu_{\delta,dyn}$ and $\mu_{\delta,dyn,eq}$ are consistently related to each other on the basis of the total strain energy dissipated by the structural deformations during the dynamic response. Hence, the adoption of the equivalent parameter $\mu_{\delta,dyn,eq}$, in addition to the equivalent resistance $R_{y,dyn,eq}$ and stiffness $k_{dyn,eq}$ (Figure 5.10), is supposed to entail a similar dynamic behaviour as under use of the multilinear resistance function. This behaviour is represented by the dynamic load factors DLF_M , shown in Figure 5.21 (right) as a function of $\mu_{\delta,dyn,eq}$. As observed before for situation $p_{EX(+)}$, the DLF_M , and hence the required resistance to oppose the blast, diminishes with increasing ductility capacity of the members. Since due to the rather small tensile reinforcement indices (Figure 5.20), this capacity, and hence the energy dissipation, is comparatively large, the DLF_M are generally smaller than those found for the beams under load arrangement $p_{EX(+)}$. The minimum DLF_M of 0.28 is obtained for an equivalent ductility ratio of about 28, whereas the maximum DLF_M of about 0.8 corresponds to a $\mu_{\delta,dyn,eq}$ close to 3. The mean value of the 136 represented DLF_M is situated at approximately 0.53.

Figure 5.21 Effective displacement ductility ratios corresponding to the multilinear ($\mu_{\delta,dyn}$, x-axis) and the equivalent bilinear resistance function ($\mu_{\delta,dyn,eq}$, y-axis) (left); Dynamic load factor DLF_M for flexural failure mode versus $\mu_{\delta,dyn,eq}$ (right); 136 beams strictly designed for acc. load arrangement $p_{EX(-)}$

Dynamic increase factors

Table 5.5 shows the mean, maximum and minimum dynamic increase factors (DIF) involved in the dynamic response of the beams subjected to load arrangement $p_{EX(-)}$. The DIF for the parameters characterising the behaviour of concrete in compression (f_c and ε_{c1}) as well as the strength of the reinforcing steel (f_y and f_t) are of the same order as those for the beams exposed to load arrangement $p_{EX(+)}$ (Table 5.4). Interesting to stress here is the very considerable dynamic increase of the concrete tensile strength f_{ct} and the modulus E_c . According to the results obtained, the latter experiences an increase of 20-30% when exposed to the explosion-induced action effects, while the former might even rise to about 40%.

Table 5.5: Dynamic increase factors (DIF) for 136 beams submitted to load arrangement $p_{EX(-)}$

Failure mode	DIF	Mean	Max	Min
M	f_c	1.17	1.26	1.09
	ε_{c1}	1.14	1.17	1.11
	E_c	1.24	1.29	1.21
	f_{ct}	1.26	1.42	1.18
	f_y	1.16	1.21	1.14
	f_t	1.05	1.07	1.05

5.4.4 Columns

Failure modes and domains due to M-N interaction

The structural behavior of the 28512 defined RC columns subjected to the accidental load arrangement $N_{EX(-)}$ is characterized by interaction of bending moments (M) - or shear forces (V), with axial forces (N). The present subsection refers to the M-N interaction. Figure 5.22 illustrates the possible associated failure domains according to the distinction into members that fail in the buckling mode (BLS, left) and those where cross-section failure concurs with exceedance of the ultimate moment (UMLS, right). For BLS members, domains B_T (tension) or B_C (compression) apply depending on whether the yield strength of the tensile- (A_{s1}) or the compression-reinforcement layer (A_{s2}) is being exceeded. In case of the UMLS columns, five possible failure domains can be distinguished (U_1 to U_5), consistent with the definition of the strain limits for ULS-design shown in Figure 4.13 (right).

Figure 5.22 Possible failure domains B_C (compression) and B_T (tension) corresponding to buckling-limit state (BLS) members (left) and U_1 to U_5 of Ultimate Moment Limit state (UMLS) members (right).

Figure 5.23 shows, for each of the 28512 analyzed members, the ratio between the axial force N_E at failure of the intermediate cross-section and the maximum axial force $N_{R,max}$ this cross-section is able to resist. Ratio $N_E/N_{R,max}$ is represented as a function of eccentricity e (2.51, 2.54), normalized for the cross-section depth h , given by (5.45). It should be stressed that ratio e/h does not represent a “real” constant eccentricity. It is introduced here as a measure for the location of the failure point (M_E, N_E) on the M-N interaction diagram in consequence of the two sequential load-paths due to gravitational loads (path 1) and the explosion-induced loads (path 2), respectively (Figure 4.11, Figure 5.23).

$$\frac{e}{h} = \frac{M_E}{N_E \cdot h} = \frac{M_R}{N_R \cdot h} \quad (5.45)$$

From Figure 5.23 it can be deduced that the study of the columns covers a wide range of failure domains. Positive $N_E/N_{R,max}$ ratios are indicative of failure under compression axial forces whereas negative $N_E/N_{R,max}$ ratios represent cases where failure arises under a tensile axial force. In all cases, a bending moment interacts with the axial force in a more or less important way, reaching from negligible ($e/h \approx 0$) to totally prevailing ($e/h \rightarrow \pm\infty$) contributions.

Figure 5.23 Ratio between axial force N_E at failure and maximum resisting axial force $N_{R,max}$ versus normalized eccentricity e/h ; 28512 columns strictly designed for accidental load arrangement $N_{EX(-)}$.

From Table 5.6 it can be deduced that the vast majority of the analysed members fails due to exceedance of the UMLS (92.5 %), where failure domain U_3 , characterised by crushing of the concrete under simultaneous yielding of the tensile reinforcement layer, is most frequent. With only very few exceptions, yielding of the tensile reinforcement characterises also the failure of the generally less frequent buckling critical members (7.5 %). The fact that the explosion entails a reduction of the axial force (Figure 5.23, path 2) explains why failure characterized by yielding of the reinforcement in compression occurs in comparatively few members.

Table 5.6: Percentages corresponding to failure domains of the 28512 analysed columns

Limit state	Failure domain (Figure 5.22)	%	Examples
BLS	B_T	7.5	E5
	B_C	≈ 0	-
UMLS	U_1	0.3	-
	U_2	11.1	E4
	U_3	52.5	E1, E3
	U_4	28.6	E2
	U_5	0	-
Total		100	-

In the following, the structural behaviour and the resulting failure modes of the analysed columns will be illustrated by means of examples E1 to E5 (see Figure 5.23 and Table 5.6). The first example-column (E1), is situated on the top floor ($n_s = 0$) of a building characterised by relatively small spacing between columns of $l = 5$ m (Figure 4.6). Hence it is characterized by an initially low compression axial force level due to gravitational loads ($N_{E,stat}$) in relation to the maximum cross-section capacity $N_{R,max}$. This can be observed in the left part of Figure 5.24 (left), which shows the idealised action effects corresponding to load-paths 1 and 2 (Figure 5.23) as well as the situation of the corresponding failure point (M_E, N_E) on the M-N interaction diagram as obtained after the final iteration-step of the established dynamic analysis procedure (section 5.2). The figure reveals that the explosion scenario reduces the initial axial force level only scanty while the corresponding moment, mainly due to the horizontal blast load $\tilde{p}_{EX}(t)$ (Fig-

ure 4.9), is increased until cross-section failure arises in domain U_3 under almost pure bending exposure, i.e. $N_E \approx 0$, $e/h \approx -39$ (Figure 5.23). In the right part of Figure 5.24, which shows the accidental *design situation* of example E1, it can be deduced that the scarce contribution of the axial force was anticipated in the design stage.

Figure 5.24 Example E1: M-N interaction diagrams (red) and sequential action effects (black). Mean value analysis (left) versus design situation (right).

In comparison to the previous case, the initial axial force N_{Estat} corresponding to example E2 is higher in relation to the maximum capacity $N_{R,max}$, what can be drawn from Figure 5.25. The reason for this is that example E2 supports the corresponding weight of $n_s = 10$ floors (E1: $n_s = 0$). Moreover, the spacing between columns ($l = 15$ m) is significantly higher than in E1. The latter aspect contributes also to a large influence area of the explosion pressure on floor and ceiling, and hence to an important axial force reduction due to component N_{EX} in relation to the increase of the bending moment M_{EX} stemming from the horizontal explosion load (Figure 4.16, left). The failure point of column E2 is situated in domain U_4 , just above the balance point ($N_E/N_{R,max} \approx 0.3$, $e/h \approx 0.7$). Interesting to observe here is that this failure mode was not predicted in the member design stage. In the right part of Figure 5.25 it is shown that the design point falls into failure domain U_5 ($x > h$, $(e/h)_d \approx 0.1$). The explanation for this discrepancy is twofold: On one hand, and more important, the consideration of more realistic material properties, in terms of mean values under inclusion of strain-rate effects, causes load-path 2 to intercept the M-N interaction diagram at a far lower axial force level than in the design situation. Secondly, and although far less influent, the fact that second-order effects are over-estimated in the design stage due to conservative design assumptions causes a more pronounced slope of load-path 2 in comparison to the design situation, what likewise results in an interception point with the interaction diagram at a comparatively lower axial force level.

Figure 5.25 Example E2: M-N interaction diagrams (red) and sequential action effects (black). Mean value analysis (left) versus design situation (right).

Figure 5.26 Example E3: M-N interaction diagrams (red) and sequential action effects (black). Mean value analysis (left) versus design situation (right).

The fact that the failure domain of the member differs from the design assumptions applies also to example E3, shown in Figure 5.26. However, in contrast to E2 the compression axial force reduction during the dynamic response of E3 is far more important, what may be mainly attributed to a larger column spacing l (E3: $l = 30$ m). In consequence of the important reduction of the compression axial force, second order effects are mitigated and failure occurs, as before in example E1, in domain U_3 under crushing of the concrete and simultaneous yielding of the tensile reinforcement layer in the intermediate cross-section ($N_E/N_{R,max} \approx 0.08$, $e/h \approx 2.7$). In the design stage, on the contrary, failure was associated with concrete crushing and yielding of the compression reinforcement in domain U_4 (Figure 5.26, right). As described before, the particular characteristics of load-path 2, given by a descending axial force and an increasing bending moment, along with the assumption of more realistic values for the material properties and, less important, the conservative estimation of second order effects in the design stage, are in the origin of this discrepancy.

Table 5.6 shows that in about 11% of the analysed members, the explosion-induced axial force reduction due to component N_{EX} causes a failure of the tensile reinforcement in failure domain U_2 . Example E4, characterised by an initially high compression axial force $N_{E,stat}$ due to significant gravitational loads ($n_s = 15$, $l = 40$ m), represents such members. As can be observed in Figure 5.27 (left), failure of E4 arises under the influence of a resulting tensile axial force ($N_E < 0$, $e/h \approx -0.84$). As shown in the right part of this figure, such structural behaviour, which is associated with extremely large column spacing l , was

neither foreseen in the member design stage, where the failure point was situated in domain U_5 ($x > h$). Besides, it should be stressed that the design for the accidental load arrangement of column E4 demands a cross-section resistance that is not sufficient to resist the design values of the action effects due to the gravitational loads ($M_{E,stat,d}$, $N_{E,stat,d}$), acting immediately before the explosion event (Figure 5.27, right). Consequently, without any need for further checking, it is clear that column E4, as designed for the accidental situation, is not a practically relevant member. In this case, the persistent design situation calls for a comparatively higher resistance (Figure 4.16, left). It is recalled here that in more than 80% of the analyzed 28512 columns, the persistent design situation would be mandatory for design of the intermediate cross-section (Figure 4.17). At a later stage of this section this issue will be resumed.

Figure 5.27 Example E4: M-N interaction diagrams (red) and sequential action effects (black). Mean value analysis (left) versus design situation (right).

The previous examples are all characterized by cross-section failure due to exceedance of the UMLS, where second-order effects are negligible. Under the assumptions met in the present study, the following three conditions must be simultaneously fulfilled for a member to be buckling-critical: a certain, initial compression axial force level in the members due to gravitational loads ($N_{E,stat}$), a relatively small reduction of this level during the explosion response (what according to the assumptions adopted implies a relatively small distance l between columns) and, most importantly, a considerable member slenderness ratio λ . One such member is example-column E5 ($N_{E,stat} / N_{R,max} \approx 0.15$, $l = 5$ m, $\lambda = 69$). In the left part of Figure 5.28 it can be observed that the explosion entails an insignificant reduction of the initial axial force $N_{E,stat}$ while the bending moment is increased until buckling failure occurs. This failure, which concurs with yielding of the tensile reinforcement layer (failure domain B_T), manifests an unstable equilibrium state before the plastic structural response stage can be entered (section 4.6.3). Figure 5.28 (right) shows that the important second order effects, of about 30%, were anticipated in the design stage, although slightly over-predicted (47%). The fact that the failure mode changes from a compression failure in design (domain B_4) to a tensile failure in the mean-value analysis is attributable, once more, to the particular characteristics of load path 2 in conjunction with more realistic assumptions for the material properties.

Figure 5.28 Example E5: M-N interaction diagrams (red) and sequential action effects (black). Mean value analysis (left) versus design situation (right).

Dynamic load factors

It was previously stated that the vast majority of the columns designed for M-N interaction is predicted to fail due to exceedance of the ultimate moment of the intermediate cross-section (UMLS). For these members, energy dissipation during the dynamic structural response can be accounted for to a more or less extent, depending on the member deformation capacity in the plastic regime. As a measure for this capacity, the effective displacement ductility $\mu_{\delta, \text{dyn}}$ (5.13) is shown in Figure 5.29 (left) as a function of the normalised eccentricity (5.45) for the totality of the 28512 members strictly designed for the accidental situation $N_{\text{EX}(-)}$ and analysed according to the procedure described in section 5.2. The figure reveals that under the influence of a resulting compression axial force ($e/h > 0$), the effective displacement ductility and hence, the potential energy absorption capacity of the members during the response to the explosion event, tendentially increases with increasing e/h , what is a logical finding. As stated before, relatively small e/h are representative of comparatively high compression axial forces N_E at failure, what implies a reduced deformation capacity. An example is column E2 where a normalized eccentricity of $e/h \approx 0.7$ is associated with almost zero plastic deformation capacity ($\mu_{\delta, \text{dyn}} \approx 1.05$). It is also recalled here that in buckling-critical members (Example E5), no such capacity can be activated ($\mu_{\delta, \text{dyn}} = 1.0$).

Towards pure bending failure ($e/h \rightarrow \pm\infty$), the effective displacement ductility $\mu_{\delta, \text{dyn}}$ converges to a constant value, which as can be observed, remains below 10 in all cases. This observation is in good agreement with the findings for the bending failure mode of the beams subjected to load arrangement $p_{\text{EX}(+)}$ (Figure 5.18). Figure 5.29 (left) further shows that under the prevailing influence of a tensile axial force ($e/h \rightarrow -0$), the effective displacement ductilities are boosted to values of up to 50. Such large plastic deformation capacities correspond to extremely ductile cross-sectional behavior, characterized by failure of the tensile reinforcement upon reaching the ultimate strain ϵ_u in failure domain U_2 (Example E4: $e/h \approx -0.8$, $\mu_{\delta, \text{dyn}} \approx 21$).

Figure 5.29 Effective displacement ductility ratio $\mu_{\delta,dyn}$ versus normalized eccentricity e/h (left); Stiffness k_{dyn} versus compression axial force due to gravitational loads $N_{E,stat}$ (right); 28512 columns strictly designed for acc. load arrangement $N_{EX(-)}$.

In summary, the results shown in Figure 5.29 (left) illustrate that a large fraction of the studied columns disposes of a potentially considerable energy absorption capacity. However, it must be taken into account that most of these members are initially subjected to large gravitational loads what has significant consequences on the member characteristics and hence on its dynamic structural performance. According to the adopted design procedure (section 4.7), the larger the axial force level $N_{E,stat}$ due to such loads, the larger are the member cross-section dimensions and hence, the higher is the member bending stiffness k_{dyn} (Figure 5.16). This is shown in Figure 5.29 (right). A larger bending stiffness, in turn, involves smaller natural periods T (3.27) in relation to the load-pulse duration t_p what can be observed in Figure 5.30 (left), which represents ratio t_p/T as a function of k_{dyn} . It is revealed that with a few exceptions, the studied members perform in the quasi-static regime (section 3.3.3), where the t_p/T ratios are too high to cause a notable influence of inertia effects and energy dissipation on the structural behavior (for instance E2, E3, E4). Consequently, the potentially large energy absorption capacity, expressed by the considerably high $\mu_{\delta,dyn}$ ratios of many members, is clearly put into perspective for in most of the cases it can be taken advantage of only to a very small extent.

Figure 5.30 Ratio between load-pulse duration t_p and natural period T versus stiffness k_{dyn} (left); Dynamic load factor DLF for flexural (red) and shear failure mode (blue) versus ratio t_p/T ; 28512 columns strictly designed for accidental load arrangement $N_{EX(-)}$

The ultimate consequence of the generally high t_p/T ratios is, that in contrast to the findings for the beams, the dynamic load factors DLF_M for most of the members do not fall significantly below unity, as illustrated by the red dots in Figure 5.30 (right). In case of members with extremely limited- or zero

plastic deformation capacity, the DLF_M factors may even amount to values larger than unity, in the most unfavourable case of up to 1.2. Among these members are the buckling-critical columns which, due to their comparatively high slenderness, are characterised by relatively small t_p/T ratios.

In addition to the DLF_M , Figure 5.30 (right) shows the dynamic load factors corresponding to the shear failure mode of the columns, DLF_V . It can be observed that, due to the quasi-static behaviour of most of the columns, the difference between the DLF_V and DLF_M ratios is generally small and vanishes with increasing t_p/T ratios. It should also be noted that due to the generally large t_p/T ratios, the DLF_V factors corresponding to the tension- (DLF_{Vs}) and compression- (DLF_{Vc}) controlled shear failure at the support cross-sections of the columns are almost identical, wherefore they are not explicitly distinguished here. This could be observed before for the beams performing under comparatively high t_p/T ratios (Figure 5.19, right).

The previous results refer to the 28512 columns strictly designed according to the *accidental situation*. As was described in section 4.7 (Table 4.3, Figure 4.17), this design situation is mandatory in less than 20% of the studied cases however. According to the hypothesis adopted in the present study, in 23757 of the 28512 members the design is to be based on the *persistent situation* since in these cases the normal load conditions demand a higher resistance than the accidental load scenario. This was illustrated by example E5 presented before (Figure 5.27, right). It should be added here that for these members, the increase in resistance with respect to the accidental design situation reflects in even higher t_p/T ratios. Hence, the before described findings concerning the quasi-static behavior likewise apply to the members designed for the persistent situation and the corresponding dynamic load factors are almost identical to those presented above. Unlike could be observed for the beams, the mentioned increase in resistance does not imply a notable decrease of the member deformation capacity. As explained before, the generally high t_p/T ratios entail that the energy dissipation capacity of the members can be taken account of only to a very small extent, in most cases of any practical relevance.

Dynamic increase factors

Table 5.7 shows the mean, maximum and minimum dynamic increase factors (DIF) involved in the dynamic response of the columns subjected to M-N or V-N interaction. Concerning the V-N response mode, the results are generally similar to those found for the beams under pure shear exposure. Some differences between beams and columns are detected for bending-influenced member response. The wide range of M-N failure domains covered in the study of the columns (Figure Figure 5.23) reflects in comparatively large scattered DIF's. While the maximum values only slightly exceed the values found for the beams under pure bending, the minimum values ($DIF \approx 1.0$) are situated far below those obtained for the beams. The mean DIF's of bending-influenced columns and beams are of the same order.

Table 5.7: Dynamic increase factors (DIF) for 28152 columns submitted to load arrangement $N_{EX(-)}$

Failure mode	DIF	Mean	Max	Min
M-N	f_c	1.12	1.25	1.00
	ε_{c1}	1.10	1.15	1.00
	f_y	1.10	1.13	1.00
	f_t	1.03	1.04	1.00
V-N	f_c	1.15	1.23	1.09
	ε_{c1}	1.12	1.14	1.11
	f_{yw}	1.11	1.12	1.10

Final observations

The analysis of the example-columns presented above revealed an interesting finding: The failure domains of the members might strongly differ from those anticipated in the design stage what, ultimately, could contribute to irrational design solutions. In particular, it was shown that the failure point on the interaction diagram based on mean values for the material properties (including strain rate effects) might be situated in a failure domain that completely differs from the design predictions, where characteristic values of the material properties are employed along with partial safety factors to define the interaction diagrams. It should be highlighted here that only the particular characteristics of the explosion-induced load-path 2, described by a simultaneous increase of bending moment and decrease of axial force, make this phenomenon possible. In other words, the observed discrepancy would not appear for the assumption of normal use conditions, i.e. under the influence of gravitational loads only. For such conditions a constant eccentricity can be assumed in many cases (Figure 2.18, path E_2^*) and hence the consideration of more realistic material properties for the definition of the M-N interaction diagram does not imply significant changes of the failure point location with respect to the design situation (see e.g. Figure 2.19).

In order to improve the described situation, it would seem advisable to consider more realistic material properties in the design stage of columns involving M-N interaction associated with gas explosions or other accidental situations involving similar load path characteristics. Important seems an appropriate definition of the representative values of the material properties, including strain rate effects. Moreover, as already suggested for the beams (section 5.4.2), it should be analysed whether the current partial safety factors constitute a suitable solution for the cross-section design under the combined influence of explosion-induced bending moments and axial forces. In the light of the herein obtained results it could be even convenient to envisage partial factors of $\gamma = 1.0$ for both concrete and steel in order to shift the design M-N interaction diagrams towards more realistic boundaries and thereby to avoid the above described inconsistencies concerning the failure point location. In any case, a consistent calibration of the design rules should be based on rationally established target reliability levels. Such levels should be derived taking account of the occurrence probabilities of explosion-induced column failure under consideration of the load-path dependant structural behaviour and failure modes (Chapter 6). Moreover, they should account for the failure consequences what calls for suitable consequence models (Chapter 7). The derivation of such target reliability levels is objective of Chapter 8.

5.5 Validation of simplified models

5.5.1 Introduction

The dynamic analysis procedure described in sections 5.2 and 5.3 is based on a simplified characterisation of both dynamic loading and structural resistance. While in view of available test results the idealization of the explosion loading by means of a pressure-time function seems to be a reasonable approach (section 3.1.4), the representation of structural resistance under such loading implies a number of simplifying hypotheses and assumptions that require a validation. For instance, the moment-curvature-based definition of the effective resistance function called for a number of assumptions related to the member stiffness, deformation capacity, the dynamic properties of its constituent materials and the effect of static preloading. Moreover, simplifying assumptions were adopted in connection with the conversion of the real system into an equivalent SDOF system by means of transformation factors (section 3.3.3), e.g. assumptions regarding the deformed shape of the members in the elastic and plastic response stage (Figure 3.10). Finally, hypothesis had to be established

concerning the distribution of inertia forces (Figure 3.14) along the members in order to estimate their contribution to the dynamic equilibrium formulated for the determination of dynamic shear forces close to the member's support cross-section.

The mentioned simplifying hypotheses and assumptions were validated with a finite element model that factors in the many complex features of structural behaviour under explosive loading. A numerical model based on a plane stress assumption was developed with the FE-code DIANA [193] and its structural performance compared to test results reported in the literature. That model was subsequently used to explore the behaviour of 18 RC beams previously analysed with the simplified procedure and models presented in sections 5.2 and 5.3.

The numerical study was carried out on the occasion of a researcher exchange programme at Delft University of Technology (TU Delft), divided into two periods of 6 weeks each. The first stay included the model development and validation by means of extensive comparisons to beam tests under both static and dynamic loads, taken from the literature. In the further continuation of the work, the numerical model was adapted to the geometry, the material characteristics and the loading conditions of the 18 beams selected for purpose of the comparative study between simplified analysis (SA) and numerical analysis (NA). However, the first analysis trials carried out before performing this study, revealed significant plastic strain concentrations in the tensile reinforcing steel cross-sections, situated at the locations of the main bending cracks close to the mid-span cross section of the beams. The influence of certain modeling parameters on the mentioned strain concentrations and their impact on the structural behaviour and failure mode of the beams was subsequently investigated. A particular aim of this preliminary study, summarized in Annex F, was to judge the possibility to establish a failure criteria based on the ultimate strain of the materials. Besides from influences of the material model for the reinforcing steel and bond-slip relationships in the steel-concrete interface, the strain concentrations were found to be strongly mesh-size dependent what precluded the establishment of an ultimate strain-based failure criterion. On the contrary, the global structural response of the members was found to converge reasonably well upon refining mesh-size. It was thus decided to conduct the subsequent comparative study between NA and SA in terms of forces and deflections rather than in terms of material strain and curvatures.

After a description of the members selected for the comparative study (section 5.5.2), the characteristics of the numerical model finally adopted for this purpose are described in section 5.5.3. This is followed by a brief summary of the model validation in section 5.5.4. The numerical analysis procedure is presented in in section 5.5.5, before some of the representative results are discussed in section 5.5.6. The content of sections 5.5.2 to 5.5.6 is partly summarized in [194, 195].

5.5.2 Members studied

The 18 beams selected out of a total of 486 include members with spans of $l = 5$ and 15 m (Table 5.8). The variations in beam slenderness ($11.4 < l/h < 15$) and the aspect ratios ($1.7 < h/b < 2.0$) of their rectangular cross-sections (Figure 4.10) are nearly negligible. The members are subjected to load-arrangement $p_{EX(+)}$ (Figure 4.8). A wide range of nominal explosion load to total load ratios, v_{acc} (4.62), is considered ($0.23 < v_{acc} < 0.84$), as shown in Table 5.8. The members analysed are designed for characteristic concrete compressive strengths of $f_{ck} = 25$ or 50 N/mm², and a reinforcing steel yield strength of $f_{yk} = 500$ N/mm². The provided longitudinal- and transverse reinforcement, respectively, A_{s1} and A_{sw}/s , resulted from the strict design at the beams mid-span- and support cross sections (section 4.7). The numerical model required also a longitudinal compression reinforcement layer, assumed as $A_{s,2} = 0.2 \cdot A_{s,1}$. The reinforcement ratios for the longitudinal tensile reinforcement, $\rho_l = A_{s1}/(b \cdot d)$, ranged

from 0.004 to 0.02 and the resulting tensile reinforcement index, $\omega = \rho_l \cdot f_{yk} / f_{ck}$, from 0.05 to 0.21 (Table 5.8). The ratio for shear reinforcement, $\rho_w = A_{sw} / (s \cdot b)$, fluctuated from 0.001 to 0.005.

Table 5.8: Some characteristics of the 18 beams selected for the comparative study

Beam n ^o	l (m)	V _{acc}	ρ_l	ω	Beam n ^o	l (m)	V _{acc}	ρ_l	ω	Beam n ^o	l (m)	V _{acc}	ρ_l	ω
1	5	0.23	0.010	0.21	7	5	0.61	0.005	0.09	13	15	0.34	0.009	0.18
2	5	0.24	0.012	0.12	8	5	0.61	0.004	0.04	14	15	0.34	0.008	0.08
3	5	0.30	0.010	0.21	9	5	0.68	0.006	0.12	15	15	0.42	0.010	0.21
4	5	0.30	0.013	0.13	10	5	0.68	0.005	0.05	16	15	0.42	0.009	0.09
5	5	0.51	0.010	0.21	11	5	0.84	0.010	0.21	17	15	0.62	0.010	0.21
6	5	0.52	0.020	0.20	12	5	0.84	0.011	0.11	18	15	0.64	0.016	0.16

5.5.3 Development of a numerical model

A finite element model was developed using DIANA [193]. Capitalising on the symmetry in the approach, a half-beam model simply supported at $l/50$ from the free end and with constrained horizontal displacement at mid-span was defined (Figure 5.31). The concrete is represented by quadratic, quadrilateral plane stress elements whose size declines gradually from $e=h/8$ at the free end of the beams to $e=h/32$ at mid-span. The model envisages truss-type embedded reinforcement [193] and a perfect bond. Three-node linear elements are applied at the top of the beams to add a distributed mass which accounts for the permanent loads acting on the members, likewise considered in the simplified analysis.

Figure 5.31 Numerical model

The model factors in material non-linearity. Concrete cracking is modelled from a smeared, rotating crack approach which aligns the axes of principle strain with the rotating crack orientations during the entire loading process [193]. The rotating crack approach was employed in conjunction with a fracture-energy based, linear tension softening law (Figure 5.32, left). For compression, uniaxial concrete stresses are deduced from the parabolic law proposed by Feenstra [196] (Figure 5.32, right). The concrete fracture energies in tension (G_f^I) and compression (G_c), required to define those constitutive models, are derived from the static compressive strength according to [129] and [197], respectively. The equations used to factor in multiaxial stress states are based on Selby's and Vecchio's studies [198]. Reinforcement behaviour is modelled by a uniaxial, bilinear stress-strain law that includes strain hardening, likewise considered in the simplified analysis (Figure 4.24).

Figure 5.32 Stress-strain relationships for concrete: Linear tension softening law (left) and parabolic law for compression [196] (right).

The increase in dynamic material properties is factored into the model in the same way as in the simplified procedure (section 4.8.3, 5.1.3), i.e. by applying DIF's to concrete compressive strength [129] and reinforcing steel yield- and tensile strengths [143] to modify the uniaxial stress-strain laws. DIF's are obtained for the average strain rates in the concrete elements under compressive stress and the steel elements under tensile stress, deduced from the numerical analysis. The approach, described in Annex G, is iterative but converges quickly since, as mentioned above, the DIF equations are logarithmical. From Annex G it can also be inferred that the assumption of a constant, average strain rate is indeed defensible within the scope of the present study.

5.5.4 Model validation

The numerical model was validated by comparing its behaviour to test results reported by Seabold [146], who studied the response of simply supported RC beams ($l = 3.91$ m) subjected to both quasi-static- and blast loading. The rectangular beam cross-sections ($h = 381$ mm, $b = 197$ mm) were reinforced in the longitudinal direction by two bars in, respectively, the tensile ($A_{s1} \approx 1289$ mm²) and the compression zone ($A_{s2} \approx 776$ mm²). In transverse direction, box-type stirrups ($A_{sw} \approx 22$ mm²) were hooked to the compression steel in some of the tested beams. Figure 5.33 shows the half-beam model of test beam WE-5 [146], with a 76 mm spacing between these stirrups. The fact that the stirrups were spaced closer (51 mm) in the other half of test beam WE-5 (in order to preclude shear failure in that region) could be disregarded in the model since the tests showed that the departure from symmetry was not large enough to cause unsymmetrical flexural response [146].

Figure 5.33 Numerical model of beam WE-5

Table 5.9 gives the static and dynamic material properties of test beam WE-5 [146] used in the present study to define the constitutive materials laws described in section 5.5.3. With the exception of E_c , G_f^I and G_c , which were estimated (as a function of f_c) from [72], [129] and [197], respectively, the quasi-static properties were derived from material tests on concrete and steel specimen [146]. Moreover, dynamic tests were carried out in order to characterize the material properties under the rapidly applied explosion load. From these tests, DIF formulations were derived as a function of strain rate [146]. Factoring average strain rates measured during the beam tests into these formulations, the dynamic material properties for f_{ct} , f_y and f_{yw} given in Table 5.9 were obtained. The indicated dynamic values for f_c , E_c and f_t , were estimated using DIF's based on [129, 143]. Tensile (G_f) and compression (G_c) fracture energies are assumed as under static loading conditions. As far as G_f is referred to, this

assumption is validated by [139], where a significant dynamic increase was recorded only under very high tensile loading rates (> 15 GPa/s), far higher than the estimated loading rate of about 1-2 GPa/s characterising the test of beam WE-5.

Table 5.9: Material property values used in the numerical analysis of beam WE-5

Property	f_c^2	f_{ct}	$E_c^{1,2}$	G_f^1	G_c^1	f_y	f_t^2	f_{yw}	E_s
Unit	N/mm ²	N/mm ²	N/mm ²	N/mm	N/mm	N/mm ²	N/mm ²	N/mm ²	N/mm ²
Static	26.9	2.72	25480	0.071	46	470	689	248	200000
Dynamic	37.2	354	32374	0.071	46	596	740	447	200000

¹ Quasi-static values for E_c , G_f and G_c estimated (as a function of f_c) from [72], [129] and [197], respectively

² Dynamic values for f_c , E_c and f_t based on [129] and [143].

In the tests [146], the beams were loaded with a uniformly distributed explosion load. Figure 5.34 depicts the load-time function recorded in the test of beam WE-5 (black line). Multi-linear (green) and bi-linear (red) approximations to this load are also shown. Those were applied in the present study in conjunction with an implicit time integration scheme [199], using a time step size of $\Delta t = 1.5 \cdot 10^{-5}$ s. This Δt was sufficiently small to avoid numerical convergence difficulties in the nonlinear analysis and, at the same time, sufficiently large to keep the computational costs within reasonable limits.

Figure 5.34 Multi-linear (green) and bi-linear (red) approximation to the time-dependant evolution of blast loading (black) to beam WE-5.

The numerical and experimental results for the time-dependant mid-span deflection (left) and reaction forces (right) are compared in Figure 5.35. The blue lines reflect the numerical results found with static material properties (Table 5.9) and the bi-linear load function (Figure 5.34). The large differences between these values and the test results (black lines) identify the need to take the strain rate effects on the material properties of the beam into consideration. When those effects are factored in by applying the dynamic property values given in Table 5.9, model prediction improves substantially, as indicated by the red curves. Using the multi-linear load function (Figure 5.34) also improves the results slightly, especially as regards deflection (green curves).

Figure 5.35 Comparison of numerical and experimental results of beam WE-5, tested by Seabold [146]: Deflection at mid-span (left); Support reaction (right).

Figure 5.36 shows beam WE-5 after the blast test. Large diagonal shear cracks can be observed which, according to the test records, appeared at about $t \approx 9$ ms after the impingement of the blast load on the member. However, collapse of the beam was prevented by the stirrups, which shortly after diagonal cracking experimented a marked increase of the strain rates ($t \approx 10$ ms). Their significant contribution to the shear force transfer is further characterized by the initiation of yielding at about 12 ms as schematically indicated in Figure 5.36 (green arrows). The yielding of the stirrups concurred with crushing of the concrete near the critical section (blue arrows).

Figure 5.36 Beam WE5 after blast test [146]

The numerical model of beam WE-5 is able to reproduce the marked increase in the strain rates of the stirrups located around the diagonal cracks, as well as their yield point. On the contrary, crushing of the concrete could not be observed in this region nor could the large diagonal shear cracks be reproduced. Previous efforts related to validation of beam models under static loads showed that the use of a *fixed smeared crack model* (assumes fixed crack directions during the entire computational process) in conjunction with a *variable shear retention factor* [193] might be more appropriate to this end. Unlike the herein employed rotating crack approach (section 5.5.3), where the shear retention factor can be implicitly assumed equal to one [193], a variable shear retention factor can account for diminishing shear stress transfer upon further crack opening. This is considered a more realistic approach, especially for the analysis of shear critical-members [200, 201]. Figure 5.37 shows the principle strain distribution corresponding to two different load stages in the model of Seabold's [146] test beam OE3 (without shear reinforcement), where a variable shear stiffness was introduced. The beam was exposed to an increasing, uniformly distributed static load until collapse. Stage a represents a load level of 56 N/mm where the beam failed in diagonal tension after the shear crack propagated into the upper portion of the member. The collapse (stage b) was retarded by dowel action (modelled by replacing the truss-type reinforcement for beam-type elements, in addition to the consideration of geometrical non-linear behaviour) and finally occurred at a load-level of about 73 N/mm. Figure 5.37 shows that the corresponding principal strain distribution matches the crack pattern of the tested beam reasonably well.

Figure 5.37 Numerically deduced crack patterns (a = failure load, b = collapse) of statically loaded beam OE3 without shear reinforcement and comparison to test result

Since the use of the fixed crack approach along with the assumption of a variable shear stiffness entailed severe numerical problems during the solution procedure of the blast load-exposed beam WE-5 model, the rotating crack approach was finally adopted for this purpose. The results presented above (Figure 5.35) confirm the validity of this approach for analysing the global structural behaviour of RC beams under blast loads. Moreover, these results corroborate the ability of the DIF-based procedure adopted to factor strain rate effects into this analysis.

5.5.5 Numerical analysis procedure

The numerical model thus validated was subsequently adapted to the geometry, material and loading characteristics of all 18 beams selected (section 5.5.2). The sequence followed in the analysis of these members was as follows:

- application of uniformly distributed static preloads, $\tilde{g} + \tilde{q}_l$ (load stage 1, Figure 4.8) in the framework of a force-controlled, non-linear static analysis based on a Newton-Raphson iteration scheme
- application of uniformly distributed blast loads, $\tilde{p}_{EX}(t)$ (load stage 2, Figure 4.8), characterised by a triangular load-pulse (Figure 4.7) with peak value $\tilde{p}_{EX,peak}$ and duration t_p in the framework of a non-linear dynamic analysis based on implicit integration [199, 202] using time steps $\Delta t = 10^{-4}$ to 10^{-3} s.

The static preloading ($\tilde{g} + \tilde{q}_l$) and the explosive pulse duration (t_p) values applied were the same as in the simplified dynamic analysis, in which the beams were found to resist a specific peak load, $\tilde{p}_{EX,peak}$, just within the bounds of their ductility ceiling (5.10). That was the load applied in the numerical analysis.

5.5.6 Comparative study

Flexural behaviour

The present section compares the simplified (SA) to the numerical (NA) analysis findings. By way of example, the structural response of beam n° 1 (Table 5.8), submitted to a static preloading of $\tilde{g} + \tilde{q}_l = 23.6$ kN/m and an explosion load-pulse characterised by a peak-load of $\tilde{p}_{EX,peak} = 24.2$ kN/m and a load

duration of $t_p = 0.2$ s, will be presented in more detail. The flexural response of the beam is graphed in Figure 5.38: on the left, bending moment, M , is plotted against mid-span deflection, δ , and on the right, δ against time. The SA and NA results are very similar, both in qualitative and quantitative terms. One significant difference is observed, however, in the transition from state I (non-cracked) to II (cracked), attributable to the fact that the approach to concrete tensile behaviour was more realistic in the numerical model (Figure 5.38, left). The transition takes place during the response to static loading. Once this load was fully applied, the two models predicted practically identical deflections, $\delta_{stat} \approx 0.021$ m.

The onset of the explosion at $t = 0$ elicits a dynamic response in the beams. The M - δ diagram for the sample beam (Figure 5.38, left) shows that the respective stiffness, $k_{M,dyn}$, is practically the same in the two approaches, with $k_{M,dyn,SA} = 3196$ kNm/m under SA and $k_{M,dyn,NA} = 2971$ kNm/m under NA. The latter was obtained by linear regression analysis of the M - δ pairs in the interval $[\delta_{stat} < \delta < \delta_y]$. Further to Figure 5.39 (left), this finding was representative of the 18 beams analysed. The conclusion that may be drawn is that stiffness, k_{dyn} , which characterises the effective resistance function used in the SDOF analysis (Figure 5.7), in turn directly related to $k_{M,dyn}$ (as per equation (3.32)), was suitably estimated. Consequently, the predictions of yield displacement δ_y by, respectively, the SA and NA, are in good agreement too, as shown in Figure 5.39 (right).

Figure 5.38 Moment-mid-span deflection (left) and mid-span deflection-time (right) curves for a sample beam found with simplified (red) and numerical (green) analysis

Figure 5.39 NA versus SA stiffness $k_{M,II}$ (left) and yield initial deflection δ_y (right) for 18 beams

The maximum bending moment obtained for the numerically analysed sample beam (by integrating stress) slightly exceeded the ultimate moment $M_{R,u}$ deduced with the simplified analysis (Figure 5.38,

left). That would appear to be a logical finding, since the contribution of concrete to tensile resistance is disregarded in the latter. Of greater interest is the similarity in the ductility demanded of the beam in the two types of analysis. According to the plot of the maximum deflections ($\delta_{\max}$) for the 18 beams analysed in Figure 5.40 (left), this finding is generally applicable in the scope of the present study. The mayor differences observed by Rong and Li [171, 172] between the SA- and NA based displacement ductility ratios $\mu_{\delta, \text{dyn}}$ for blast-loaded beams (see section 3.3.6) are not detected here. This can be drawn from Figure 5.40 (right) where the percentage deviation $\zeta_{\mu_{\delta, \text{dyn}}}$ between $\mu_{\delta, \text{dyn}}$ according to SA and NA, given by (3.48), is plotted for the 18 beams analysed (blue dots). Yet it is true that, in line with Rong and Li's findings, the herein obtained $\zeta_{\mu_{\delta, \text{dyn}}}$ tend to rise when the longitudinal reinforcement ratio ρ_l of the beams decreases, they remain considerably below the mean value obtained by the mentioned authors (orange line), previously shown in Figure 3.19. To explain this, it must be recalled that the SDOF analysis conducted by Rong and Li [171, 172] significantly underestimated the numerical yield displacement δ_y , what the authors related to the choice of the member stiffness (section 3.3.6). In the present study, the estimated member stiffness, and as consequence, the member yield displacement, concur fairly well with the numerical results (Figure 5.39). This confirms once more that the degree of precision in the prediction of the structural behaviour by means of simplified dynamic methods, such as SDOF-based models, greatly depends on the choice of the corresponding modeling assumptions and hypothesis rather than on the methodology itself (section 3.3.6).

Figure 5.40 NA versus SA maximum deflection $\delta_{\max}$ for 18 beams (left); Percentage deviation $\zeta_{\mu_{\delta, \text{dyn}}}$ between $\mu_{\delta, \text{dyn}}$ according to SA and NA versus longitudinal reinforcement ratio ρ_l for 18 beams (right)

Dynamic reaction forces

As a measure for the dynamic shear forces developing close to the member supports, the support reaction forces will be analysed in the following. The reaction force $V(t)$ of the sample beam n° 1 exposed to the sequence of gravitational loads and explosion load-pulse ($t_p = 0.2$ s, $\tilde{p}_{\text{EX, peak}} = 24.2$ kN/m) is shown in Figure 5.41 (left) as a function of time. The two analyses (SA and NA) deliver generally similar results. Closer inspection of the figure reveals that, in spite of the introduced numerical damping (HHT-algorithm [202] with $\alpha = -0.3$; for $\alpha = 0$ the HHT-method reduces to the Newmark algorithm), the finite element approach predicts oscillations in the earliest dynamic response. The post-peak reaction force determined with NA was also characterised by oscillations, although of a lower frequency. The simplified model was unable to reproduce such oscillations, observable also in experimental studies (Figure 3.18, Figure 5.35, right). The maximum explosion-induced reaction V_{EX} (excluding reaction force $V_{\text{E, stat}}$ due to static loads) was around 24 % higher under NA than SA (Figure 5.41, left). Figure 5.41 (right) shows that, with one exception, the numerical results ($V_{\text{EX, NA}}$) for the 18 beams were 5 to 30 %

higher than the simplified values ($V_{EX,SA}$). Larger differences between both approaches were found for comparatively smaller non-dimensional time-ratios t_p/T . Similar observations were made by Schachter et al. [170], where the dynamic behaviour of steel beams was analysed (see also section 3.3.6).

Figure 5.41 Reaction force-time curve for a sample beam found with simplified (red) and numerical (green) analysis (left); Ratio $(V_{EX,NA} - V_{EX,SA}) / V_{EX,SA}$ versus ratio t_p/T for 18 beams (right).

As all structures, the beams analysed here are in reality distributed mass systems and, as such, theoretically dispose of an infinite number of degrees of freedom [164]. However, generally only few of the lower response modes are of any significance, and for many practical situations, only the fundamental mode is relevant. In the present study, an equivalent single degree of freedom system, in which only the elementary motion type is possible, could adequately describe the flexural behaviour of the RC beams, as shown before. On the contrary, the results shown in Figure 5.41 suggest that the consideration of the elementary response mode alone is not sufficient for a close representation of the member support reactions and could contribute to a significant underestimation of the member shear forces close to the supports. In the following, this subject will be analysed in more detail.

The time-dependant evolution of the three force components of the dynamic equilibrium equation given by (3.43), are shown in Figure 5.42 (left) as obtained in the SA of the sample beam: the resultant force F of the applied explosion loading $\tilde{p}_{EX}$, the corresponding support reaction V_{EX} and inertia force I . It can be observed that shortly after the onset of the explosion ($t = 0$), the applied force F is mainly resisted by the inertia force I , what is attributable to the fact that the beam has had no time to deform at that early response stage, i.e. the flexural resistance R is not yet developed. However, in the further course of the member response, the inertia I loses increasingly importance, while the force transmitted to the supports (V_{EX}) increases progressively. The peak of V_{EX} approximately coincides with the maximum of the applied force F at $t_{V,max,SA} \approx t_p/2 = 0.1$ s. Only a few milliseconds before, the yield strength of the longitudinal reinforcement is reached at the mid-span cross-section of the beam ($t_{y,SA} = 0.096$ s, Figure 5.38). During the following plastic response stage, V_{EX} experiences a linear decay, until at $t = t_{max,SA}$ the beam experiences its maximum bending response (Figure 5.38), approximately coincident with $t = t_p = 0.2$ s. The subsequently initiated free vibration response phase ($t > t_p$) is characterised by mutually compensating V_{EX} and I forces.

By means of the dynamic equilibrium equation (3.43), the time-dependant inertia force I contributing to the resistance of the *numerically analysed beam* can be deduced from the applied force F and the dynamic support reaction V_{EX} . The result is compared in the right part of Figure 5.42 (green curve) to the SA-based inertia force (red curve). It can be observed that, disregarding the high frequency oscillations, the SA-based result conforms well to the numerical findings during the elastic response of

the member ($t < t_y$). However, at $t = t_{y,NA} = 0.099$ s, when the plastic hinge begins to form around the mid-span cross-section, the numerically deduced inertia force I experiences a sudden, pronounced decay, which is compensated by further increase of the reaction force V_{EX} , until its peak-value is obtained at $t_{V,max,NA} \approx 0.11$ s (Figure 5.41, left). On the contrary, in the SA, which cannot reproduce this decay, the maximum reaction force reaches peak about 10 ms before, at $t_{V,max,SA} \approx t_p/2 = 0.1$ s, coincident with the peak of the applied force F .

Figure 5.42 Breakdown of dynamic equilibrium (3.43) applied to sample beam (left): F (applied force), V_{EX} (support reaction) and I (inertia force); Inertia force-time curve for sample beam found with simplified (red) and numerical (green) analysis (right)

In order to explore the backgrounds of the observed discrepancy between SA- and NA-based inertia and reaction forces, Figure 5.43 compares the accelerations $\ddot{\delta}(x, t)$ along the longitudinal x -axis of the sample beam at different instants t (Figure 5.42, right) during the member response. The graphs corresponding to the SA (red) are at any time proportional to the assumed deflected shapes $\phi(x)$ (Figure 3.10) (red). The numerical results (green) were obtained at each of the nodes of the FE-mesh (Figure 5.31). Unrealistic, high frequency noise in the numerical solution could be damped out [202].

Within the first milliseconds after initiation of the blast, where the member has not yet undergone any notable flexural deformations, the numerical $\ddot{\delta}(x, t)$ plots are subject to rapidly fluctuating oscillations, suggesting significant influence of higher response modes (Figure 5.43-a, $t = 0.012$ s). The integration (3.44) of these continually changing $\ddot{\delta}(x, t)$ shapes reflects in the previously mentioned oscillations of inertia- and reaction forces during the earliest dynamic response the beam (Figure 5.41, left, Figure 5.42, right). On the contrary, the integration of the parabola-type $\ddot{\delta}(x, t)$ -plots laid down in the SA, based on the fundamental response mode, entails a continuous inertia-and reaction force-time relationship, which does not reflect the influence of higher response modes. In any case, the likelihood of a premature shear failure in the first instants after applying the explosive loading on the beam is practically zero since the reaction forces remain clearly below the provided shear resistance. More impulsive load conditions (e.g. a lower rise time of the explosion load-pulse) could increase this likelihood, as pointed out in [148].

The approximation of the numerical- and simplified $\ddot{\delta}(x, t)$ -plots improves in the further course of the member response, suggesting a decreasing influence of the higher response modes. At $t = 0.048$ s, the reaction force contributes significantly to maintenance of the dynamic equilibrium of the beam (Figure 5.42, left), denoting the participation of flexural stiffness to the resistance mechanism. As shown in Figure 5.43-b, the numerical result now reasonably approaches the parabola function assumed in the SA, thus suggesting almost proportional behaviour between the intensity of the inertia force $i(x)$ and the

ordinate of the deflected shape $\phi(x)$ at any point of the beam. Consequently, the resulting inertia forces $I(t)$ according to NA and SA are in very good agreement (Figure 5.42, right).

The conformity between NA and SA ends when the yield strength of the longitudinal tensile reinforcement is being approached. At that time, when the plastic hinge starts to form around the mid-span cross-section, a jump in the beam acceleration can be observed (Figure 5.43-c), also reported by van Wees and Peters [169] and Magnusson [144]. This jump again excites higher vibration modes, which seem to become increasingly important as the plastic response stage proceeds. Figure 5.43-d shows that at $t = t_{Vmax} = 0.11$ s, a large part of the beam is submitted to positive accelerations. The linear $\ddot{\delta}(x)$ -function, based on the rigid body behaviour during the plastic member response (Figure 3.10) assumed in the SA, is clearly not being approached, neither qualitatively nor quantitatively. This circumstance reflects in significantly different resulting inertia forces $I(t)$, as already observed in (Figure 5.42, right). After having experienced a sharp decay at $t = t_y$, the numerically obtained $I(t)$ approaches zero values at $t_{Vmax} = 0.11$ s, i.e. the still applied explosion load is almost entirely counteracted by the reaction forces. On the contrary, in the SA this decay cannot be simulated wherefore $I(t)$ still contributes to the resistance in a significant manner. This discrepancy reasonably explains the difference in the maximum reaction force recorded in both approaches.

Figure 5.43 Acceleration plots $\ddot{\delta}(x, t)$ along longitudinal x-axis of the sample beam ($x=0$: support; $x = 2.5$ m: midspan) at different instants t during the dynamic response; a: $t = 0.012$ s (early elastic response); b: $t = 0.048$ s (developed elastic response); c: $t = 0.096$ ($\approx$ limit of elastic response); d: $t = 0.11$ s (plastic response; V_{max}).

Final observations

From the foregoing analysis it can be concluded that the simplified methods, based on the fundamental mode, might underestimate the support reactions of RC members performing in the post-yield response

stage, i.e. after the plastic hinge at the critical cross-section starts to develop. The formation of the plastic hinge triggers a significant contribution of higher response modes to the distribution of inertia forces along the members and hence to their support reactions. Similar observations were made for blast-exposed steel beams [170]. In the mentioned study, it was found that the prediction capacity of the simplified methods based on the fundamental mode significantly diminished with increasing displacement ductility ratio μ_δ (5.10). This has an important implication for the present study, where tension-induced shear failure (failure of the stirrups) close to the member is associated with lower μ_δ ratios than those assumed in the context of a ductile bending failure, and consequently, with lower sustainable peak pressures (Figure 5.18, left). In other words, shear failure is likely to occur at far lower peak-loads $\tilde{p}_{EX,peak}$ than applied in the present comparative study, chosen to comply with the ductility ceilings associated with flexural failure of the SA-beams (section 5.5.5), rather than their shear capacity. In order to comply with (5.4), which imposes equality between the dynamic reaction forces $V_{EX} (= V_{E,dyn})$ and the corresponding shear resistance $V_{R,dyn}$, the peak-load applied in the comparative study must hence be reduced, in case of the example beam n° 1 in about 50%, from $\tilde{p}_{EX,peak} = 24.2$ kN/m (assumed in the calculations presented above) to $\tilde{p}_{EX,peak} = 12.5$ kN/m. The reaction force $V(t)$ of the example beam under this reduced peak-load is compared in Figure 5.44. It can be observed that both solutions (NA and SA) are very close to each other. The magnitude of the oscillations affecting the post-yield response of the NA is notably smaller than shown before in Figure 5.41 (left). In consequence of this, the maximum support reaction obtained in the NA ($V_{EX,NA} = 38.8$ kN) is only about 7% higher than the SA solution ($V_{EX,SA} = 36.4$ kN), i.e. far smaller than the 24% difference detected in Figure 5.41 (left). The conclusion is that the above-detected discrepancies between SA- and NA-based support reactions, due to the influence of higher response modes, are strongly put into perspective since they are associated with unrealistic high peak-load levels (in relation to the provided shear force resistance $V_{R,s}$ limited by failure of the stirrups). On the contrary, the mentioned discrepancies appear to be principally relevant for the compression-controlled shear failure of the considered RC beams. In any case, it should be kept in mind that this failure mode is generally highly unlikely to occur (section 5.4.2) what will be reflected in the corresponding reliability analysis (Chapter 6).

Figure 5.44 Reaction force-time curve for the sample beam found with simplified (red) and numerical (green) analysis (applied peak-load level complies with tension-controlled shear failure).

5.6 Summary

The dynamic load- and increase factors (DLF_E and DIF_m) by which means the dynamic effects during the structural response, such as the contribution of inertia forces, energy dissipation and strain rate effects,

are being quantified within the LSFs are deduced from a deterministic analysis of the defined structural members, based on mean values for the relevant material properties. Simplified dynamic models and methods, previously described in section 3.3, are used for this purpose. Chapter 5 contains the corresponding developments and results.

The approach for the definition of the DLF_E and DIF_m is exposed in section 5.1. The former are related to the ultimate resistance corresponding to the failure mechanisms analyzed. The latter are based on average material strain rates deduced from the simplified dynamic analysis of the members. For purpose of this analysis, an iterative procedure has been developed that factors in material non-linearity, strain rate effects and the gravitational pre-loading of the members prior to the explosion event (section 5.2, Figure 5.6).

The implementation of this procedure for the different structural member types and failure modes is subject of section 5.3. Thereby, the focus is placed on the development of the dynamic resistance function, based on an $M-\chi$ analysis (Figure 5.7) under an appropriate assumption for a relationship between curvature- and displacement ductility (see also Annex C). The analysis of the columns under a dynamic bending moment-axial force interaction requires a specific solution algorithm that accounts for the axial force dependent formulation of structural resistance under consideration of both the material- and geometrical non-linearities involved (section 5.3.4, Figure 5.13 to Figure 5.16, Annex D).

The results of the dynamic analysis procedure are described in section 5.4. Concerning the beams, the results manifest that the required flexural resistance to face the applied blast load significantly decreases as the member deformation capacity increases (section 5.4.2, Figure 5.18, left). Due to comparatively low reinforcement degrees, this effect is especially important for the beams subjected to explosion loads acting in opposite direction to the gravitational loads (section 5.4.3, Figure 5.21). On the other hand, it seems likely that the beams are not able to resist the dynamic shear forces that arise during the flexural response close to the member supports. The results show that at the time of tension-controlled shear failure, only about 1/3 of the energy dissipation capacity corresponding to the flexural failure mode is taken advantage of (Figure 5.18, right) what reflects in comparatively higher DLF 's (Figure 5.19, left). On the contrary, a compression-controlled shear failure seems highly unlikely to occur. It is found that the peak pressure the members are able to sustain until this type of failure occurs, largely exceeds the corresponding value associated with the tension-controlled shear mode or the flexural mode (Figure 5.19, right).

In relation to these results, it is interesting to recall that the mid-span and the support cross-sections of the beams have been strictly designed, respectively for bending and shear, to resist the nominal explosion pressure specified in the applied code (section 4.7). In the flexural design, a ductile cross-sectional behavior was envisaged by limiting the neutral axis depth x/d . Nevertheless, it seems likely that the beams as designed, will fail in the more brittle shear (tension tie) mode before the full flexural response might be developed. This finding is consistent with the frequently quoted observation that structural members, when exposed to blast loads, might fail in shear, even if they were designed to exhibit a ductile, flexural behavior under quasi-static loading conditions (section 3.2.3). In order to perform in the desired ductile flexural mode, the beams would require either a higher shear resistance or a lower bending resistance and/or stiffness. The latter option could be achieved by considering the beneficial effect of energy dissipation explicitly in the flexural design stage of explosion-exposed members, for instance by introducing dynamic load factors such as derived in the present thesis. In addition, the current definition of the design material properties for accidental situations should be analyzed in detail. The very dispersed and failure-mode dependent ratios between mean- and design resistance (Table 5.4) corrobo-

rate this and suggest that the introduction of failure mode- and load-type specific dynamic increase factors for the materials might contribute to a more rational design approach for certain accidental situations.

In comparison to the beams, the analysis of the columns revealed far less disparity in the energy absorption capacity associated with the different failure modes considered (section 5.4.4). Due to a comparatively high flexural stiffness, accentuated by the influence of compression axial forces, this capacity is generally small. This manifests in similar dynamic load factors for bending and shear failure modes (Figure 5.30), with values larger than unity in many cases.

In addition, the analysis of the columns revealed that the failure domains of the members might strongly differ from those anticipated in the design stage what, ultimately, could contribute to irrational design solutions. In particular, it was shown that the failure point on the interaction diagram based on mean values for the material properties (including strain rate effects) might be situated in a failure domain that completely differs from the design predictions. Only the particular loading characteristics associated with the explosion, which induce a simultaneous increase of bending moment and decrease of axial force (Figure 4.11), make this phenomenon possible. In other words, the observed discrepancy would not appear for the assumption of normal use conditions, i.e. under the influence of gravitational loads only. For such conditions, a constant eccentricity can be assumed in many cases (Figure 2.18) and hence the consideration of more realistic material properties would not imply significant changes of the failure point location with respect to the design situation.

In order to improve the described situation, it would seem advisable to consider more realistic material properties in the design stage of columns involving M-N interaction associated with gas explosions, or other accidental situations, involving extraordinary load path characteristics. In the light of the herein obtained results it could be even rational to envisage partial factors of $\gamma = 1.0$ for both concrete and steel in order to shift the design M-N interaction diagrams towards more realistic boundaries and thereby to avoid the described inconsistencies concerning the failure point location.

For purpose of the dynamic analysis procedure described in section 5.2, and implemented in section 5.3, a number of simplifying assumptions had to be made, particularly in connection with the dynamic resistance function. The mentioned assumptions were validated, to the extent possible, with a finite element model that factors in the many complex features of structural behaviour under explosive loading (section 5.5). A numerical model based on a plane stress assumption was developed (section 5.5.3, Figure 5.31) and its structural performance compared to test results reported in the literature (section 5.5.4, Figure 5.35). That model was subsequently used to explore the behaviour of 18 RC beams previously analysed with the simplified analysis procedure and models. The comparative study (section 5.5.6), carried out on the occasion of a researcher exchange programme at Delft University of Technology, showed that numerical results agree closely with the simplified model predictions, particularly with respect to flexural behaviour (Figure 5.38 to Figure 5.40). On the contrary, the simplified methods might underestimate the support reactions of RC members performing in the post-yield response stage. The formation of the plastic hinge triggers a significant contribution of higher response modes to the distribution of inertia forces along the members and hence to their support reactions (Figure 5.41 to Figure 5.43). In any case, the impact of this finding on the further developments of the present study is negligible for the critical tension tie shear failure mode occurs far before the yield strength of the longitudinal reinforcement of the beams is reached. When performing the analysis with the peak pressure corresponding to the ultimate shear resistance of the beams, numerical and simplified analysis results are in good agreement (Figure 5.44).

Chapter 6 Reliability analysis

In section 4.1, the basic assumptions for the inference of risk-based requirements in the context of the present study were exposed. Acceptable levels for structure-related risks were linked to the risk set out in the structural design codes and standards in force, which depends on the *reliability level implicitly required* by these codes (section 4.1.1). The determination of this level in the context of accidental situations characterized by gas explosions is subject of step 5 of the established analysis procedure (section 4.3, Figure 6.1). Previous developments included the definition of the LSF's for the members and failure modes to be analyzed (section 4.6), the strict member design based on these functions (section 4.7), the stochastic characterization of the basic variables involved (section 4.8) and the dynamic analysis in order to quantify associated dynamic load- and strain rate effects (Chapter 5).

Based on these developments, the present chapter addresses procedural aspects and the results of the First Order Reliability Method (FORM)-based analysis carried out to establish the reliability level implicitly required by the employed codes. Section 6.1 describes the specifically developed reliability procedure for the analysis of column reliability under M-N interaction, what requires the determination of the strain distribution corresponding to the most likely failure point on the LSF (section 4.8.2). Subsequently, section 6.2 presents the results for both beams and columns under the influence of the assumed accidental load arrangements (section 4.5). Finally, section 6.3 investigates into the accumulated failure probabilities of some of these members, taking account of the fact that in addition to the accidental scenarios, their failure might be triggered by persistent load arrangements.

Figure 6.1 Procedural flow-chart

6.1 Analysis of columns under M-N interaction

6.1.1 Introduction

In section 2.5, it was exposed why the reliability estimation of RC members subjected to M-N interaction is in many senses a complex endeavor. It was shown that the reliability level is dependent on the failure point location on the interaction diagram, which, in turn, is a function of the member loading sequence (Figure 2.18). Taking account of the assumed load-path characteristics for the herein studied columns, the limit state corresponding to M-N-induced failure was defined in section 4.6.3 by expression (4.27), based on direction of path n°2 (Figure 4.11), which describes the structural response to the explosion-induced action effects taking account of the static pre-loading due to gravitational loads. The LSF (4.27) is a problem-specific implementation of formulation (2.53), which imposes equality of the length of the action effect- and resistance vectors (zero safety margin), respectively (**E**) and (**R**) (Figure 2.18).

Important to observe is that the LSF (4.27) is dependent on the strain distribution (ϵ) across the critical cross-section, as well as on the - via curvature - related displacements (δ), required for quantification of second order effects. As already announced in section 4.8.2, performing the reliability analysis requires determining the strain distribution corresponding to the most likely-failure point on the LSF (4.27), also known as FOSM-design point (Figure 2.12). This can be achieved by coupling the enhanced FOSM-, or FORM analysis to the deterministic solution algorithm, presented in section 5.3.4 to determine the relevant failure mode and the location of the failure point on the M-N interaction diagram. An iterative analysis procedure that enables this coupling has been developed and applied, in a previous work [99], to a set of columns under gravitational load exposure. A generic description of this procedure is given in section 6.1.2 below. An application example for the herein studied, explosion-exposed members is presented in section 6.1.3.

6.1.2 Procedure

The developed iterative procedure for purpose of the reliability analysis of the columns under M-N exposure is schematically represented in the flow chart shown below (Figure 6.2). The different steps comprising the procedure are described in advance.

Figure 6.2 Iterative reliability analysis procedure applied to columns under M-N interaction

The mean value-based, deterministic analysis procedure of the columns constitutes the point of departure. Member failure mode and the corresponding failure point on the M-N interaction diagram are determined as described in section 5.3.4 under consideration of the specific member loading characteristics and the involved geometrical- and material non-linearities. The strain distribution in the critical cross-section associated with the obtained failure point provides an initial estimate of the strain distribution corresponding to the FORM design point, $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_j$, where $j = 1$.

Along with the previously established probabilistic models for the different geometrical, material and loading properties of the members (section 4.8), strain distribution $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_j$, and the related displacements $(\delta^*, \delta_{stat}^*)_j$, characterize the set of basic variables X_i involved in the LSF (4.27). Based on this input-data, the FORM analysis is being carried out. The resulting FORM design point vector, $(\mathbf{x}^*)_j$, provides the coordinates $x_{i,j}^*$ of the most likely-failure point on the limit state surface (4.27), in terms of each of the involved X_i , as expressed by (6.1) or (6.2).

$$(\mathbf{x}^*)_j^T = (x_1^*, x_2^*, \dots, x_n^*)_j \quad (6.1)$$

$$(\mathbf{x}^*)_j^T = (b^*, d^*, h^*, A_s^*, \dots, f_{c,dyn}^*, f_{y,dyn}^*, \dots, M_G^*, M_Q^*, M_{EX}^*, N_G^*, N_Q^*, N_{EX}^*, \dots, \varepsilon_{s1}^*, \varepsilon_{s2}^*, \delta^*, \delta_{stat}^*)_j \quad (6.2)$$

Based on the design point coordinates $x_{i,j}^*$, the resulting action effects $N_{E,j}^*$ and $M_{E,j}^*$ are computed from (4.22) and (4.23), with input from (4.28) to (4.36). Likewise, the resulting resistances $N_{R,j}^*$ and $M_{R,j}^*$ are determined from equations (4.37) to (4.44). Bending moments $M_{E,j}^*$ and $M_{R,j}^*$ are normalized to the cross-section depth h_j^* . The obtained pairs $(M_E^*/h^*, N_E^*)_j$ and $(M_R^*/h^*, N_R^*)_j$ constitute the final points of the action effect- and resistance vectors, respectively $(\mathbf{E}^*)_j$, and $(\mathbf{R}^*)_j$ (Figure 2.18), corresponding to the most likely failure point on the LSF (4.27). The starting point of these vectors is given by the pair of action effects prior to the explosion event $(M_{E,stat}^*/h^*, N_{E,stat}^*)_j$.

It must be noted that LSF (4.27), as a specific formulation of (2.53), imposes equality of the length of action effect- and resistance vectors, $(\mathbf{E}^*)_j$, and $(\mathbf{R}^*)_j$, *but not of their direction*. Hence, although condition (4.27) is fulfilled, it is possible that the pairs $(M_E^*/h^*, N_E^*)_j$ and $(M_R^*/h^*, N_R^*)_j$ are not aligned with point $(M_{E,stat}^*/h^*, N_{E,stat}^*)_j$, so represented schematically in the left part of Figure 6.3. In this case, although $|\mathbf{E}^*)_j| = |\mathbf{R}^*)_j|$, as imposed by (4.27), the limit state is not yet reached since point $(M_E^*/h^*, N_E^*)_j$ does fall within the boundaries of the interaction diagram (*safe region*). The origin of this apparent inconsistency lies in the fact that (4.27) is a unidimensional simplification of the generic, two-dimensional limit state definition given by (4.24) and (4.25). This generic definition imposes simultaneous fulfilment of the equilibrium conditions between acting and resisting axial forces and bending moments, respectively, and, consequently, requires pairs $(M_E^*/h^*, N_E^*)_j$ and $(M_R^*/h^*, N_R^*)_j$ to be merged in a single point, as represented in the right part of Figure 6.3.

Figure 6.3 Action effect $(\mathbf{E}^*)_j$ - and resistance $(\mathbf{R}^*)_j$ vectors corresponding to design point vector $(\mathbf{x}_i^*)_j$; left: Same length but different direction; right: Same length and same direction (final iteration-step j)

Taking account of these considerations, the obtained pairs $(M_E^*/h^*, N_E^*)_j$ and $(M_R^*/h^*, N_R^*)_j$ are checked in a subsequent step for equality. In the affirmative case, the iterative analysis procedure has ended and the reliability index β related to design point $(\mathbf{x}^*)_j$ constitutes the desired, final result. On the contrary, action effect vector $(\mathbf{E}^*)_j$ is being elongated to obtain interception point $(\overline{M_R^*/h^*}, \overline{N_R^*})_j$ with the interaction diagram, as sketched in Figure 6.3 (left). The strain distribution $(\overline{\varepsilon_{s1}^*}, \overline{\varepsilon_{s2}^*})_j$ corresponding to this updated failure point, as well as the related displacements $(\overline{\delta^*}, \overline{\delta_{stat}^*})_j$, provide input-data to a new FORM analysis leading to a new design point vector, $(\mathbf{x}^*)_j$ (6.1), where $j = 2$. This procedure is being continued until, as shown in the right part of Figure 6.3, pairs $(M_E^*/h^*, N_E^*)_j$ and $(M_R^*/h^*, N_R^*)_j$ approximately coincide within the established tolerance of $\max(\Delta M^*, \Delta N^*) = 1\%$. In general, $j = 2$ to 5 steps suffice to obtain convergence of the results.

6.1.3 Example

In the following, the previously described procedure will be applied to example column E2 introduced in section 5.4.4. Figure 6.4-a reproduces the result of the mean value analysis of the member (Figure 5.25, left). The failure point is situated in domain U4 (Figure 5.22), just above the balance point. The corresponding strain in the compression reinforcement layer, $\varepsilon_{s2} = -3.15\text{‰}$ exceeds the dynamic yield strain $\varepsilon_{y,dyn}$ while the strain in tensile reinforcement layer, $\varepsilon_{s1} = 2.91\text{‰}$, remains slightly below.

Strains $(\varepsilon_{s1}, \varepsilon_{s2})$ and the – via curvature – related mid-span displacements δ and δ_{stat} , are subsequently established as an initial estimation of the FORM design values, $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_{j=1}$ and $(\delta^*, \delta_{stat}^*)_{j=1}$, see Table 6.1. The FORM algorithm is then carried out, based on the LSF (4.27). From the obtained design point coordinates $x_{i,j=1}^*$, the resulting action effects, $N_{E,j=1}^* = 11094\text{ kN}$ and $M_{E,j=1}^* = 4113\text{ kNm}$, as well as the resulting resistances $N_{R,j=1}^* = 12118\text{ kN}$ and $M_{R,j=1}^* = 4816\text{ kNm}$ are determined (Table 6.1). These are shown in Figure 6.4-b (red dots). Since $(M_E^*/h^*, N_E^*)_{j=1}$ is far away from coincidence with $(M_R^*/h^*, N_R^*)_{j=1}$, action effect vector $(\mathbf{E}^*)_{j=1}$, defined by the points $(M_{E,stat}^*/h^*, N_{E,stat}^*)_{j=1}$ and $(M_E^*/h^*, N_E^*)_{j=1}$, is being elongated and intercepted with the interaction diagram, leading to the updated failure point $(\overline{M_R^*/h^*}, \overline{N_R^*})_{j=1}$, shown in Figure 6.4-b. The corresponding strain distribution $(\overline{\varepsilon_{s1}^*}, \overline{\varepsilon_{s2}^*})_{j=1} = (3.93\text{‰}, -2.99\text{‰})$ as well as the related displacements $(\overline{\delta^*}, \overline{\delta_{stat}^*})_{j=1}$, provide input-data to a new FORM analysis based on (4.27). Thereby account must be taken of the fact that the failure domain has changed from U4

to U3, what implies an exchange of formulations (4.40) and (4.41) for the determination of forces F_{s1} and F_{s2} in, respectively, the tensile and compression reinforcing steel layers.

The results of the new FORM analysis are summarized in Table 6.1 ($j = 2$) and Figure 6.4-c. It is illustrated that action effects $(M_E^*/h^*, N_E^*)_{j=2}$ and resistances $(M_R^*/h^*, N_R^*)_{j=2}$, deduced from the design point coordinates $X_{i,j=2}^*$, clearly approached and now almost coincide. However, since the tolerance criterion is still slightly exceeded ($\max(\Delta M^*, \Delta N^*) = 2\% > 1\%$), the failure point and the corresponding strain distribution are once more updated. The findings of the subsequent 3rd iteration step ($j=3$) are shown in Table 6.1 and Figure 6.4-d. Now, coincidence between action effects $(M_E^*/h^*, N_E^*)_{j=3}$ and resistances $(M_R^*/h^*, N_R^*)_{j=3}$ has been achieved under consideration of the established tolerance ($\max(\Delta M^*, \Delta N^*) = 0.6\% < 1\%$) and thus the procedure has ended.

Figure 6.4 Action effects and resistance during the iteration procedure for the reliability analysis of example column E2; a) Mean value analysis (starting point), b) Iteration step $j = 1$, c) $j = 2$, d) $j = 3$

Table 6.1 Summary of results obtained in reliability analysis procedure applied to column E2 (Positive N = compression axial forces)

j	ε_{s1}^* [‰]	ε_{s2}^* [‰]	M_R^* [kNm]	M_E^* [kNm]	N_R^* [kN]	N_E^* [kN]	$\max(\Delta M^*, \Delta N^*)$ [%]	$\bar{M}_R^*$ [kNm]	$\bar{N}_R^*$ [kN]	$\bar{\varepsilon}_{s1}^*$ [‰]	$\bar{\varepsilon}_{s2}^*$ [‰]
1	2.91	-3.15	4816	4113	12188	11094	14.5	4710	9990	3.93	-2.99
2	3.93	-2.99	4741	4644	10112	9985	2.0	4674	9935	4.02	-2.97
3	4.02	-2.97	4719	4688	9930	9890	0.6	4675	9924	4.02	-2.97

Attention should be drawn to the fact that the mean value- and FORM analysis provide comparable results. It might be noted, for instance, that the corresponding interaction diagrams are very similar, almost equal in the tension-controlled failure zone, as shown in Figure 6.5 (left). The failure points are

both located close to the respective balance points of these diagrams. Hence, unsurprisingly, the initial estimation for the corresponding strain distribution, $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_{j=1}$, based on the mean-value analysis, was actually relatively close to the final solution, $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_{j=3}$, apart from the observed change of the failure domain from compression- to tension-controlled failure. This, in turn, explains the relatively moderate decrease the failure probability p_{fEX} (4.6) of the example column experiences during the iteration procedure, from $p_{fEX} \approx 4.0 \cdot 10^{-5}$, after the first iteration step ($j=1$), to $p_{fEX} \approx 1.3 \cdot 10^{-5}$ after step three ($j=3$), as shown in Figure 6.6 (continuous line). As a rule, the closer the initial estimation of the failure point and the associated strain distribution to the final solution, the smaller are the variations in p_{fEX} during the iteration procedure. This becomes evident if, for purpose of illustration only, the initial strain estimation for the FORM-design point, $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_{j=1}$, is adopted from the code-based design procedure following the accidental design rules (section 4.7.2). Figure 6.5 (right) reminds that, as shown earlier (Figure 5.25, right), the code-based design point (M_{Rd}, N_{Rd}) was located in failure domain 5 (Figure 5.22), where the entire cross-section is under compressive stresses. The corresponding design strain in the reinforcing steel, $(\varepsilon_{s1d}, \varepsilon_{s2d}) = (-0.6\text{‰}, -3.0\text{‰})$, is fundamentally different from the FORM-design point solution $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)$, according to which the failure point (M_R^*, N_R^*) is located in domain 3. Consequently when adopting $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_{j=1} = (\varepsilon_{s1d}, \varepsilon_{s2d})$, the variations in p_{fEX} during the reliability analysis procedure are large, far larger than if $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)_{j=1}$ is based on the mean value analysis. The dashed line in Figure 6.6 shows this. The failure probability p_{fEX} decreases from $p_{fEX} \approx 7.2 \cdot 10^{-3}$, after the first iteration step, to $p_{fEX} \approx 1.2 \cdot 10^{-5}$ after the final step six ($j=6$). Necessarily, the result after the final iteration step is approximately the same as obtained before (continuous line). The difference is that, due to the imprecise initial estimation of $(\varepsilon_{s1}^*, \varepsilon_{s2}^*)$, more iteration steps are required to reach convergence.

Figure 6.5 Comparison between interaction diagrams and failure points corresponding to FORM design point (blue curves) and, respectively, mean value analysis (left) and code-based design (right)

Figure 6.6 Evolution of failure probability $p_{f|EX}$ during iteration with initial strain distribution according to mean value analysis (continuous line) and code-based member design (dashed line).

The fact that the mean value analysis of the M-N exposed columns provides failure point estimations that approach the FORM design point solution in a sufficiently exact manner, suggests that the associated strain distributions could be employed for purpose of a direct reliability estimate, i.e. a unique FORM analysis without the need to follow the iteration-based procedure described in section 6.1.2. On the contrary, the erroneous estimation of the failure point location in the code-based design rules out the use of the corresponding design strain distribution for such a purpose, since this could entail completely distorted reliability levels. When the reliability analysis of the explosion-loaded columns should be based on the code-based design strain, the application of the iteration-based procedure (section 6.1.2) seems unavoidable. Previous work showed that this is not the case if only gravitational loads are considered [99]. Under assumption of a constant eccentricity (no second order effects were accounted for in [99]), the code-based design procedure and the FORM analysis delivered very similar design points and associated strain distributions.

6.2 Reliability level under explosion-induced load-arrangements

6.2.1 Introduction

The present subsection addresses the *reliability level implicitly required by the codes* for the representative set of strictly designed structural members (section 4.4) subjected to the gas explosion-induced load arrangements defined in section 4.5.

In section 3.1.2, it was exposed that the occurrence rate of gas explosions $p(EX)$ can be modeled by a Poisson process, depending on the probability of an explosion event per time unit λ . The good correlation among the findings reported in studies conducted in different countries and different periods (Table 3.1) allows for an assumption of $\lambda \approx 10^{-5}$ explosions per year and housing-unit supplied with gas. Factoring this value into equation (3.1), the occurrence probability $p(EX)$ of a gas explosion in a 50-year reference period (T_{ref}) is given by (6.3).

$$p(EX) \approx \lambda \cdot T_{ref} = 5 \cdot 10^{-4} \quad T_{ref} = 50 \text{ years} \quad (6.3)$$

The advanced FOSM-or FORM analysis (section 2.2.3) deliver the implicitly admissible, conditional failure probabilities $p_{f,adm|EX}$ for the members, given the occurrence of a gas explosion within the established

reference period T_{ref} of 50 years. As a function of both, $p_{f,\text{adm}}|\text{EX}$ and occurrence probability $p(\text{EX})$, the implicitly admissible failure probability $p_{f,\text{adm}}$ is then established according to (4.6).

The implicitly admissible failure probability $p_{f,\text{adm}}$ can be converted to required reliability indices β_{req} using the inverse standard normal distribution (2.31). In line with the nomenclature established in section 4.1, the reliability index $\beta_{\text{req,EX}}$ is referred to in advance. Where relevant, a distinction between members designed according to, respectively, the accidental (Figure 4.1, path a_1) and the persistent design situations (Figure 4.1, path b_1) is drawn.

In the following sections 6.2.2 and 6.2.3, the results obtained for, respectively, the beams and the columns will be presented and discussed. Subsequently, in section 6.2.4 the impact of the previously mentioned changes in the employed design rules (section 4.7.4) on those results is evaluated. Some final remarks regarding the notional character of the obtained results are given in section 6.2.5.

6.2.2 Beams

Positive bending failure at mid-span ($M_{(+)}$)

Figure 6.7 (left) shows the reliability indices $\beta_{\text{req,EX}}$ ($T_{\text{ref}} = 50$ years) corresponding to the flexural failure mode $M_{(+)}$ of the 486 beams subjected to load arrangement $p_{\text{EX}(+)}$ (Figure 4.8). The results correspond to members strictly designed according to the *accidental situation* (Figure 4.1, path a_1), irrespective of the fact that this situation might be mandatory for member design or not. On the left, the $\beta_{\text{req,EX}}$ are represented as a function of the accidental load ratio v_{acc} between the explosion- and the total loads (4.62). It is shown that $\beta_{\text{req,EX}}$ decreases as v_{acc} increases what might be mainly attributed to the relatively large uncertainty associated with the explosion pressure p_{EX} . The strong influence of the explosion load on the results, which is moreover reflected in their dependency on the venting coefficient A_v/V , can be deduced from Figure 6.7 (right), which shows the sensitivity coefficients α_i (Figure 2.12) corresponding to each of the basic variables X_i involved in the limit state function (4.14). The figure reveals that, with the exception of a few cases, characterized by very low v_{acc} , the explosion-induced bending moment M_{EX} is the clearly dominant variable for the considered failure mode. The corresponding α_i (blue dots) increase with rising v_{acc} and converge to a value of about 0.98. A similar behaviour is denoted for the model uncertainty coefficient corresponding to the dynamic load effect, $\xi_{\text{ME,dyn}}$, with α_i values of around 0.3 for the majority of the studied members (red dots). While the influence of M_{EX} and $\xi_{\text{ME,dyn}}$ on the failure probability increases with rising v_{acc} , the contrary is observed for all other variables. The higher v_{acc} , the smaller is the influence of geometrical or material parameters, as well as static loads, on the likelihood of an explosion-induced bending failure of the beams. For most of the studied cases, these variables could be treated as deterministic without introducing an appreciable error in the reliability level.

Figure 6.7 Reliability index $\beta_{req,EX}$ (left) and corresponding sensitivity coefficients α_i (right) for failure mode $M_{(+)}$ under load-arrangement $p_{EX(+)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of accidental load ratio, v_{acc}

The dominant influence of the explosion load on the member reliability is manifested as well in the following Figure 6.8. The left part shows, as a function of v_{acc} , the ratio between the resisting bending moment M_R^* , based on components x_i^* of the FORM design point vector $\mathbf{x}^*$ (section 2.2.3), to the code-based design moment M_{Rd} of the beams. It is observed that this ratio increases with rising influence of the explosion load, reaching from about 1.11, for small v_{acc} , to about 1.34, if v_{acc} is very large. On the contrary, an optimum design rule calibration, would suggest M_R^*/M_{Rd} ratios close to unity. Interestingly, this is achieved when replacing design moment M_{Rd} for moment M_{Rm} according to the *mean value analysis* under consideration of strain-rate effects (section 5.4.2). As shown in Figure 6.8 (right), ratios M_R^*/M_{Rm} reasonably approach unity with increasing influence of the explosion load on the member reliability. This is indeed logical since, as seen before in Figure 6.7, the impact of the resistance variables (geometry and material parameters) on the failure probability decreases simultaneously and hence the corresponding design point coordinates x_i^* approach the respective mean values. These results corroborate earlier findings in Chapter 5 that questioned the appropriateness of the current definition of design material parameters in the context of accidental design rules. Indeed, Figure 6.8 (right), suggests that a more realistic representation of these parameters, by means of their respective mean values, including strain rate effects, would seem to provide a more rational solution for accidental design situations involving gas explosion-induced load arrangements.

Figure 6.8 Ratios between FORM design point bending moment M_R^* and, respectively, code-based design value M_{Rd} (left) or mean value M_{Rm} (right), for failure mode $M_{(+)}$ under load-arrangement $p_{EX(+)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of accidental load ratio, v_{acc}

In the left part of Figure 6.9, the before-discussed results corresponding to beams designed for the *accidental situation* (Figure 4.1, path a_1), are compared to reliability indices $\beta_{\text{req,EX}}$ for members designed to the requirements of the *persistent situation* (Figure 4.1, path b_1). It is shown that the elements designed for normal use conditions are characterized by comparatively higher reliability levels if the explosion load is small in relation to the total loads, whereas for large v_{acc} ratios the contrary is being observed. A transition zone between v_{acc} ratios of about 0.35 and 0.45 can be identified inside of which the member reliability level is found to be almost independent of the design situation.

In the right part of Figure 6.9, the reliability indices $\beta_{\text{req,EX}}$ are distinguished depending on the *mandatory design situation* of the members (Figure 4.1, paths a_1 or b_1). It can be observed that for small v_{acc} , at the left of the mentioned transition zone ($v_{\text{acc}} \lesssim 0.35$), the persistent design situations are mandatory for design (path a_1 , 273 members), i.e. the provided resistance, and consequently $\beta_{\text{req,EX}}$, is higher than demanded by the accidental design situations (Figure 6.9, left). On the contrary, such situations govern member layout for larger v_{acc} (path b_1 , 213 members), situated at the right of the transition zone ($v_{\text{acc}} \gtrsim 0.45$). For these members, the provided member resistance in the design for normal loads is comparatively lower and so are the corresponding β . The fact that a smaller resistance likewise entails a higher deformation capacity of the members, what is generally favorable to the reliability level, was not found to be a decisive influence.

Figure 6.9 Reliability index $\beta_{\text{req,EX}}$ for failure mode $M_{(+)}$ under load-arrangement $p_{\text{EX}(+)}$ of beams, as a function of accidental load ratio, v_{acc} ; Left: Comparison between *accidental and persistent design* (Figure 4.1, paths a_1 and b_1); Right: Distinction for *mandatory design situation* (Figure 4.1, paths a_1 or b_1)

The results shown in the right part of Figure 6.9 represent the reliability level, which further to the mandatory design situation according to the applied design codes, would be implicitly required for the flexural failure mode of the studied RC beams under the explosion load scenario $p_{\text{EX}(+)}$. A statistical evaluation of these results delivers a mean value of $\mu_{\beta} \approx 4.3$ which is situated above the nominal target ceiling demanded by current design codes, $\beta_{\text{t,code}} = 3.8$. However, the scatter affecting the results should also be highlighted. Assuming β to be normally distributed, the estimated fraction below $\beta_{\text{t,code}}$ is still about 17%, as schematically shown in Figure 6.8, right.

Shear failure at support (V_c or V_s)

In the following, Figure 6.10 to Figure 6.15 summarize the results corresponding to an explosion-induced shear failure scenario of the beams under load arrangement $p_{\text{EX}(+)}$ (Figure 4.8). The results for tensile (V_s)- and compression (V_c) controlled shear failure are distinguished on, respectively, the left and the right of the mentioned figures.

Figure 6.10 shows the reliability index $\beta_{\text{req,EX}}$ for members strictly designed according to the *accidental situation* (Figure 4.1, path a_1), irrespective of the fact that this situation might be mandatory or not. The corresponding sensitivity coefficients α_i are represented in Figure 6.11 below. It is shown that, as observed before for the flexural failure mode, the explosion pressure-related action effect, shear force V_{EX} , and the corresponding model uncertainty $\xi_{\text{VE,dyn}}$, have greatest effect on the failure probability of the members. The influence of other variables is only considerable for very low accidental load ratios v_{acc} , where the explosion load is small in relation to the total loads. Exceptions to this are the model uncertainty associated with the resistance of the compression strut, $\xi_{\text{VR,c}}$, and the concrete compressive strength f_c involved in the determination of this resistance. As denoted by the corresponding α_i coefficients, shown in Figure 6.11 (right), these variables might influence the probability for compression-controlled shear failure also in cases where explosion loads prevail over the gravitational loads.

Figure 6.10 Reliability index $\beta_{\text{req,EX}}$ ($T_{\text{ref}} = 50$ years) for failure mode V_s (left) and V_c (right) under load-arrangement $p_{\text{EX}(+)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of accidental load ratio v_{acc}

Figure 6.11 Sensitivity coefficients α_i for failure mode V_s (left) and V_c (right) under load-arrangement $p_{\text{EX}(+)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of load ratio v_{acc}

The ratios between the FORM design point resistances V_R^* and the code-based design resistances V_{Rd} are shown in Figure 6.12. As observed before for the bending failure mode, these ratios diverge from unity as the influence of the explosion pressure on the reliability level increases. Especially resistance $V_{\text{R,c}}^*$, associated with failure of compression strut, might largely exceed the code-based design value $V_{\text{Rd,c}}$, up to about 2.2 times. When replacing the code-based design-values V_{Rd} by the corresponding mean values V_{Rm} , including dynamic material properties, this changes drastically. Figure 6.13 shows that, as observed before for bending (Figure 6.8, right), ratios V_R^*/V_{Rm} approach unity for large v_{acc} . As already

stated before, this is related to the dominant influence of the explosion pressure on the reliability level that eclipses the contribution of other variables involved in the problem. The consideration of more realistic dynamic material properties and/or dynamic load effects in the design stage could provide a possible solution approach to the detected inconsistencies.

Figure 6.12 Ratios between FORM design point shear force V_R^* and code-based design value V_{Rd} for failure mode V_s (left) and V_c (right) under load-arrangement $p_{EX(+)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of ratio between explosion- and total loads, v_{acc}

Figure 6.13 Ratios between FORM design point shear force V_R^* and mean value V_{Rm} for failure mode V_s (left) and V_c (right) under load-arrangement $p_{EX(+)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of ratio between explosion- and total loads, v_{acc}

Figure 6.14 shows the reliability level, which further to the *mandatory design situation* according to the applied design codes (Figure 4.1, paths a_1 or b_1), would be implicitly required for the shear failure modes of the studied RC beams under the accidental load scenario $p_{EX(+)}$. In comparison to the bending failure mode (Figure 6.9, right), the relatively high mean values μ_β and the insignificant fractions p below $\beta_{t,code} = 3.8$ associated with an explosion-induced failure of the compression strut (V_c) at the support of the beams ($\mu_\beta = 4.7$, $p = 2\%$) suggest that this failure mode is rather unlikely to occur. On the contrary, the failure of the tension tie (V_s) turns out to be especially critical, corroborated by comparatively low μ_β and high p values ($\mu_\beta = 4.0$, $p = 30\%$). These results support the findings from the mean-value analysis in section 5.4.2, where the proneness of the strictly designed beams to fail in tension-induced shear, on one hand, and their highly unlikely shear-compression failure, on the other, was already anticipated.

Figure 6.14 Reliability index $\beta_{req,EX}$ ($T_{ref} = 50$ years) for failure mode V_s (left) and V_c (right) under load-arrangement $p_{EX(+)}$ of beams designed according to *mandatory design situation* (Figure 4.1, paths a_1 or b_1), as a function of accidental load ratio v_{acc}

Negative bending failure at mid-span ($M_{(-)}$)

The reliability index $\beta_{req,EX}$ ($T_{ref} = 50$ years) corresponding to the negative flexural failure mode $M_{(-)}$ of the 136 beams subjected to load arrangement $p_{EX(-)}$ (Figure 4.8) is shown in Figure 6.15 (left). The results correspond to members strictly designed according to the *accidental situation* (Figure 4.1, path a_1), which, in absence of any persistent influences acting in contrary sense to the gravitational loads, corresponds necessarily to the *mandatory situation* for failure mode $M_{(-)}$. Characterized by a relatively low mean value ($\mu_\beta = 4.0$) and scatter ($v_\beta = 4\%$), the $\beta_{req,EX}$ are represented as a function of the venting coefficient A_v/V and the *inverse value* of accidental load ratio v_{acc} between the explosion load (EX) and the total load (EX-G), on the abscissae. As a tendency, $\beta_{req,EX}$ is observed to decrease with increasing v_{acc}^{-1} , what may be mainly traced back to comparatively larger permanent loads (G): In the member design stage, the higher the contribution of G to the resistance mechanism, the lower is the required amount of reinforcement to resist the explosion induced action effect. A low reinforcing degree, in turn, increases significantly the deformation capacity of the members (section 5.4.3) what ultimately contributes to a higher reliability level. Due to the comparatively lower reinforcing degrees and the associated larger deformation capacity, the influence of energy dissipation on the reliability of the beams is higher than for the beams performing in positive bending, where gravitational loads and explosion load act in the same direction.

Figure 6.15 Reliability index $\beta_{req,EX}$ (left) and corresponding sensitivity coefficients α_i (right) for failure mode $M_{(-)}$ under load-arrangement $p_{EX(-)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of inverse accidental load ratio, v_{acc}^{-1}

The mentioned contribution of the permanent loads G to the resistance mechanism may be observed likewise in Figure 6.15 (right) which shows the sensitivity coefficients α_i for the involved basic variables. It is shown that for small v_{acc}^{-1} , the bending moment due to the permanent loads on the floor system, M_{Gf} , and the associated model uncertainty $\xi_{ME,stat}$, do have certain influence on the member reliability, although not of significant character. With increasing v_{acc}^{-1} , this influence vanishes while other resistance variables, such as the yield strength of the reinforcement, f_y , or the model uncertainty for the resisting bending moment, ξ_{MR} , slightly gain importance. In any case, the influence on the reliability level of all these variables is almost negligible in comparison to the contribution of the accidental load effect, M_{EX} , and, to a minor extent, the associated model uncertainty $\xi_{ME,dyn}$. Figure 6.15 (right) shows that these variables clearly dominate the failure probability, what also explains the significant benefit of higher venting coefficients A_v/V observed in Figure 6.15 (left). In compliance with the observations made before for the beams under load arrangement $p_{EX(+)}$, Figure 6.16 shows that the dominant influence of the explosion-related action effect would call for a member design under assumption of more realistic values for certain resistance variables. Contrary to the code-design based bending moments, M_{Rd} , the mean values M_{Rm} , which include strain rate effects, provide a good estimate of moment M_R^* associated with the FORM design point.

Figure 6.16 Ratios between FORM design point bending moment M_R^* and, respectively, code-based design value M_{Rd} (left) or mean value M_{Rm} (right), for failure mode $M_{(-)}$ under load-arrangement $p_{EX(-)}$ of beams designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of inverse accidental load ratio, v_{acc}^{-1}

6.2.3 Columns

Failure due to bending-axial force interaction at the intermediate cross-section (M-N)

Figure 6.17 (left) shows the reliability index $\beta_{req,EX}$ ($T_{ref} = 50$ years) corresponding to failure of the intermediate cross-section of the columns due to the interaction of bending moments and axial forces (M-N) under load arrangement $N_{EX(-)}$ (Figure 4.9). A huge scatter characterizes the results, which correspond to members strictly designed according to the *accidental situation* (Figure 4.1, path a_1), irrespective of the fact that this situation might be mandatory for design or not. The reliability indices β are observed to depend strongly on the normalized design eccentricity, $(e/h)_{d,acc}$ (4.63). Columns subjected to small $(e/h)_{d,acc}$, where design axial compression forces prevail, are characterized by high β . On the contrary, large $(e/h)_{d,acc}$, indicative of dominant design bending moments, are associated with small β . The minimum value of about 3.6 approximately coincides with the minimum values found for the beams under the influence of pure bending (Figure 6.9, Figure 6.15), what is a reasonable finding.

The dependency of the reliability level on the normalized eccentricity $(e/h)_{d,acc}$ is related to the potential safety margin the members dispose in the M-N space to oppose the explosion-induced action effects.

This margin is represented by the length of the resistance vector $\mathbf{R}^*$ (see Figure 6.3), equal to the distance between the pair of static action effects, acting prior to the explosion event ($M_{E,stat}^*/h^*$, $N_{E,stat}^*$), and the failure point on the M-N interaction diagram (M_R^*/h^* , N_R^*). As a rule, the smaller $(e/h)_{d,acc}$ the larger is this potential safety margin and, hence, the higher is the required explosion pressure p_{EX}^* to induce member failure. This, in turn, has a direct consequence on the failure probability. This is visible in Figure 6.18 (left), which relates the reliability index $\beta_{req,EX}$ corresponding to example columns E1 to E5, introduced in section 5.4.4, to the ratio between the FORM design point coordinate for the explosion pressure, p_{EX}^* , and its mean value μ_{pEX} (Figure 4.21). Ratio p_{EX}^*/μ_{pEX} is introduced here as a measure for the likelihood of the occurrence of an explosion event where the failure pressure p_{EX}^* is generated. The higher p_{EX}^*/μ_{pEX} the smaller is this likelihood, and as Figure 6.18 (left) shows, the higher is the reliability level.

Figure 6.17 (right) confirms the strong dependency of the reliability level on the explosion pressure magnitude. The sensitivity coefficients α_i associated with variable p_{EX} are larger than 0.9 in all cases, denoting a dominant influence on the member failure probability. All other variables involved are only scantily- or not at all relevant. Represented examples are the concrete compressive strength f_c or the axial force due to permanent loads of the building floor systems, N_{Gf} . It should be noted that due to the load-path dependent formulation of the LSF (4.7), the distinction of load and resistance variables is not straightforward what explains that α_{f_c} and $\alpha_{N_{Gf}}$ can adopt both positive and negative values.

Figure 6.17 Reliability index $\beta_{req,EX}$ ($T_{ref} = 50$ years) (left) and corresponding sensitivity coefficients α_i (right) for failure due to M-N interaction, under load-arrangement $N_{EX(-)}$, of columns designed according to *accidental design situation* (Figure 4.1, path a_1), as a function of normalized design eccentricity $(e/h)_{d,acc}$

A couple of examples will be shown in advance to provide a better insight into the above-described findings. At first, example column E5, previously introduced in section 5.4.4 (Figure 5.28) will be referred to. Figure 6.18 (right) shows the result obtained after the final iteration step of the reliability analysis procedure introduced in section 6.1. Failure occurs due to buckling in the tensile force-controlled domain B_T (Figure 5.22) under a normalized eccentricity of $(e/h)^* \approx 0.9$ what almost matches the prediction in member design, $(e/h)_d \approx 0.8$ (Figure 5.28, right). Given the prior loading due to the gravitational loads and considering the, in this case, significant second-order effects, an explosion-induced bending moment $M_{E,dyn}^*$ of about 69 kNm and a simultaneous axial force component $N_{E,dyn}^*$ of only 22 kN would cause such a failure. An explosion pressure p_{EX}^* of less than 7 kN/m² would be sufficient to induce these action effects. This value still exceeds the assumed mean value μ_{pEX} around two times, as can be drawn from Figure 6.18 (left). However, taking into account the relatively large coefficient of variation associated with the explosion pressure ($CoV_{pEX} = 0.7$), the likelihood for this to happen in a

realistic gas explosion scenario is relatively high what manifests in an accordingly high admissible member failure probability given the occurrence of the explosion ($p_{f,adm|EX} \approx 3.7 \cdot 10^{-2}$). Finally, factoring in the occurrence probability $p(EX)$ (6.3), a required reliability index associated with a 50 year reference period of $\beta_{req,EX} \approx 4.1$ is obtained.

A different conclusion is obtained for example column E2, previously described in sections 5.4.4 (Figure 5.25) and 6.1.3 (Figure 6.4, Figure 6.5). In spite of the small design eccentricity, $(e/h)_{d,acc} \approx 0.1$, which locates the design failure point of this column in the compression-controlled domain 5, the reliability analysis concludes with a prediction of a tension-controlled failure close to the balance point, with a corresponding normalized eccentricity of $(e/h)^* \approx 0.7$. Taking as a starting point the initial action effects due to gravitational loads, $(M_{E,stat}^*/h^*, N_{E,stat}^*)$, such a failure would require an explosion-induced increase of the bending moment of $M_{E,dyn}^* \approx 4100$ kNm, under a simultaneous decrease of the compression axial force of about $N_{E,dyn}^* \approx 7000$ kN. Therefore, the corresponding peak pressure p_{EX}^* acting on the closures of the compartment where the explosion occurs, would have to attain an extremely high value, about 10 times higher than the assumed mean value μ_{pEX} , as shown in Figure 6.18, left. Such a value is very unlikely to occur under realistic circumstances and, consequently, the probability of failure given the occurrence of the explosion is accordingly small ($p_{f,adm|EX} \approx 1.3 \cdot 10^{-5}$). When, in addition, the occurrence probability $p(EX)$ is factored in (6.3), a practically zero failure probability is obtained. The corresponding reliability index $\beta_{req,EX}$ is 5.7 ($T_{ref} = 50$ years), hence considerably higher than in example E5.

Figure 6.18 Reliability index $\beta_{req,EX}$ (left) for example columns E1 to E5 as a function of ratio between FORM design point coordinate for explosion pressure p_{EX}^* and mean value μ_{pEX} assumed in FORM analysis (left); Result of reliability analysis for example column E5 (right)

Example column E2 is representative for a number of members whose failure mode under the assumed load arrangement $N_{EX(-)}$ could not be anticipated in the member design stage. The fundamentally different normalized eccentricities (e/h) corresponding to, respectively, the code-based design point, $(e/h)_{d,acc} \approx 0.1$, and the FORM design point, $(e/h)^* \approx 0.7$, underline this (Figure 6.5). Figure 6.19 shows the relationship between $(e/h)^*$ and $(e/h)_{d,acc}$ for all the 28512 columns designed according to the accidental situation. It can be observed that, similar to column E2, a large number of members, circled in the Figure, are characterized by small $(e/h)_{d,acc}$ and large (positive or negative) $(e/h)^*$. Although many of these elements are not of practical relevance, since the persistent design situation would demand a higher resistance than the accidental, these results affirm the potential for improvement of the current design rules for columns exposed to accidental events, such as gas explosions. In addition to the high uncertainties associated with such events, the explicit consideration of the M-N trajectory is thereby of crucial importance. While current rules might be appropriate for design of persistent situations, where

normal, mainly gravitational actions prevail, they might deliver irrational results if applied to extraordinary sequences of bending moments and axial forces. That was previously detected in section 5.4.4 and is confirmed by the reliability analysis. The right part of Figure 6.19, where $(e/h)_{d,acc}$ has been replaced by the normalized eccentricity $(e/h)_m$ found in the mean value analysis (section 5.4.4), suggests that an improvement of this situation might envisage a mean-value based design, under explicit consideration of strain-rate effects. It is shown that the mayor inconsistencies regarding the member failure mode, detected before in the $(e/h)_{d,acc} - (e/h)^*$ representation (Figure 6.19, left) have disappeared. It should be noted that the asymptotic branches which can be detected in Figure 6.19 (right) denote a moderate deviation in the failure point location, while the failure mode is well represented by the mean value analysis. For instance, normalized eccentricities of $(e/h)_m \approx 10$ and $(e/h)^* \approx -50$ both denote a cross-section failure under prevailing bending moments with almost negligible axial force contribution.

Figure 6.19 Normalized eccentricity corresponding to FORM design point $(e/h)^*$ versus normalized eccentricity corresponding to code-based design $(e/h)_{d,acc}$ (left) and mean-value analysis $(e/h)_m$ (right)

Figure 6.20 shows the reliability level $\beta_{req,EX}$ ($T_{ref} = 50$ years), which further to the *mandatory design situation* (Figure 4.1, paths a_1 or b_1) according to the applied design codes, would be implicitly required for failure due to M-N interaction of the studied RC columns under the accidental load scenario $N_{EX(-)}$. As previously determined in section 4.7.3, in more than 80 % of the 28512 analysed columns, this situation is governed by normal use conditions (persistent design situations) where gravitational loads prevail. Such members are associated with small normalized eccentricities $(e/h)_{d,acc}$ and accordingly large $\beta_{req,EX}$ (clear dots). On the contrary, for large positive or negative $(e/h)_{d,acc}$, where bending moments prevail, member design is most likely to be performed on the grounds of the accidental design situation. Such members are associated with comparatively smaller $\beta_{req,EX}$ (dark dots), in many cases smaller than the target ceiling, $\beta_{t,code} = 3.8$. A zone can be identified where the transition between both design situations takes place ($0.3 \lesssim (e/h)_{d,acc} \lesssim 0.8$), situated in the vertex of the asymptotic $\beta - (e/h)_{d,acc}$ scatter plot.

Figure 6.20 Reliability index $\beta_{\text{req,EX}}$ ($T_{\text{ref}} = 50$ years) for failure due to M-N interaction, under load-arrangement $N_{\text{EX}(-)}$, of columns designed according to *mandatory design situation* (Figure 4.1, paths a_1 or b_1), as a function of normalized design eccentricity $(e/h)_{\text{d,acc}}$

Failure due to shear-axial force interaction at the support cross-section (V - N)

The reliability indices $\beta_{\text{req,EX}}$ for column failure due to shear-axial force interaction induced by accidental load arrangement $N_{\text{EX}(-)}$ are shown in Figure 6.21. The corresponding sensitivity coefficients α_i for some selected basic variables are represented below in Figure 6.22. Results for tensile (V_s)- and compression (V_c) controlled shear failure are distinguished on, respectively, the left and the right. It should be kept in mind that no distinction is made here for persistent and accidental design situations of the columns, since, according to the assumptions met, the persistent load scenario involves exclusively gravitational actions, which, in absence of any lateral influence, do not contribute to the probability of column shear failure.

The $\beta_{\text{req,EX}}$ corresponding to V_s - N failure of the columns (Figure 6.21, left) are plotted as a function of the ratio between the design shear force $V_{\text{su,d}}$ sustained by the transverse reinforcement A_{sw}/s and the total design shear resistance, $V_{\text{Rd,s}}$, based on (4.48). The higher the contribution of V_{su} to the resistance mechanism the lower is the influence of component V_{cu} due to concrete in compression. The lower V_{cu} the less will the more uncertain axial forces be involved in the resistance of the tensile tie, what results in higher $\beta_{\text{req,EX}}$ values (ratios $V_{\text{su,d}}/V_{\text{Rd,s}} > 1$ indicate the influence of tensile axial forces N_{Ed} on the design shear resistance $V_{\text{Rd,s}}$). Figure 6.22 (left) confirms this. While coefficients α_{NGf} , representing the permanent loads of the floor system, approach zero with increasing $V_{\text{su,d}}/V_{\text{Rd,s}}$, α_{fyw} , which show the influence of the yield strength of the transverse reinforcement, increase. In any case, the influence of these variables on the reliability level is small in comparison to the dependency of explosion pressure p_{EX} , which determines both shear force V_{EX} and axial force N_{EX} acting on the columns. Figure 6.22 (left) shows that variable p_{EX} is dominant in all cases. Moreover, the model uncertainty associated with the dynamic shear force $V_{\text{E,dyn}}$ is relevant to member reliability.

The dominant influence of the explosion pressure and the associated action effects on the reliability level of the columns under V_s - N interaction is likewise manifested by the strong dependency of the venting coefficient A_v/V (Figure 6.21, left). Unsurprisingly, the results underline the beneficial effect of venting providing pressure relief during the explosion event. On average, the failure of the tension tie of the columns turns out to be comparatively critical, corroborated by a low mean μ_β and a high fraction p below the code target value, $\beta_{\text{t,code}} = 3.8$ ($\mu_\beta = 3.7$, $p = 70\%$).

Figure 6.21 Reliability index $\beta_{req,EX}$ ($T_{ref} = 50$ years) for failure mode V_s-N (left) and V_c-N (right) of columns under accidental load-arrangement $N_{EX(-)}$ (Figure 4.1, path a_1), as a function of respectively, ratio between design shear force $V_{su,d}$ and total design shear resistance $V_{Rd,s}$, and shear resistance reduction factor K

Figure 6.22 Sensitivity coefficients α for selected variables corresponding to failure mode V_s-N (left) and V_c-N (right) of columns under load-arrangement $N_{EX(-)}$ as a function of respectively, ratio $V_{su,d}/V_{Rd,s}$ and factor K

The influence of the axial force level in the columns on the resistance of the compression strut, V_{Rc} is accounted for by reduction coefficient K (4.47). Figure 6.22 (right) shows, as a function of K , the sensitivity coefficients α_i for selected basic variables involved. Small K denote a significant influence of the axial compression forces on the failure probability, as manifested by coefficients α_{NGf} , representing the permanent loads of the floor system. It should be observed that axial compression forces appear here as *action variables* due to their unfavorable contribution to member reliability. The dominating *resistance variable* for small K is the concrete compressive strength f_c , with α_{f_c} coefficients close to 0.9.

With reducing influence of the compression axial force level on the resistance of the compression strut, i.e. increasing K , the explosion pressure gains increasingly in importance. Positive α_{pEX} coefficients represent the shear force V_{EX} due to the horizontal explosion load, while negative α_{pEX} are indicative of the here favorable tensile axial force component N_{EX} due to the vertical pressure contribution on floors and roof (Figure 4.6, Figure 4.9). For large K , the shear-inducing, horizontal explosion load becomes the dominant variable. In addition, model uncertainties for both action effects ($\xi_{VE,dyn}$) and resistance (ξ_{VRc}) have a significant influence on the failure probability.

The complex dependencies of the reliability level on the different load- and resistance variables reflect in the reliability indices $\beta_{req,EX}$ shown in Figure 6.21 (right). For very small reduction coefficients K , $\beta_{req,EX}$

values of around 4.5 are attained, irrespective of the venting coefficient A_v/V considered, what is attributable to the almost zero influence of the explosion pressure. For larger K , this influence increases and, consequently, the beneficial effect of venting is manifested in the reliability indices. Nevertheless, due to the considerable influence of variables different from the explosion pressure and the associated dynamic load effects, the dependency of $\beta_{\text{req,EX}}$ on A_v/V is far less pronounced as observed before for the tension-controlled shear failure (Figure 6.21, left). Maximum $\beta_{\text{req,EX}}$ values correspond to reduction factors K around 0.3, whereas minimum reliability indices are attained when the influence of the uncertain explosion pressure is maximum, close to $K = 1.0$. On average, a relatively high mean value and a p -fraction close to zero ($\mu_\beta = 4.5$, $p \approx 0$) suggest that failure of the compression strut due to shear-axial force interaction in the columns is comparatively unlikely to occur.

6.2.4 Influence of changes in design code-rules

The present study is based on a consistent set of national codes which by now have been substituted by updated versions. The most relevant differences (concerning the present study) between the herein applied codes rules and those contained in the standards in force (national codes or Eurocodes) were described before in section 4.7.4. These concern the partial factor γ_c for the concrete compressive strength in accidental design situations, coefficient K for modification of the design shear resistance $V_{R,c}$ due to the influence of compression axial forces and the nominal eccentricity e_{tot} for M-N design of members under influence of second-order effects. The following observations are made with regard to these differences and their possible impact on the previously described results.

Partial safety factor for concrete γ_c

In the present study, a partial safety factor for the concrete compressive strength of $\gamma_c = 1.3$ was adopted from the Spanish Code [83] for the design of RC members in accidental design situations. The current Eurocode [84], on the contrary, prescribes a slightly lower value of $\gamma_c = 1.2$. In general terms, a smaller partial safety factor implies a less conservative design in the sense that it provides the member with a lower resistance, and, consequently, with a comparatively lower reliability level when exposed to a specific load arrangements (assuming that the same probabilistic models are employed to represent the involved basic variables). Most notable this will be for the compression-controlled shear resistance, for which the concrete compressive strength is paramount. However, it was observed before that the failure of the compression strut is comparatively unlikely, as denoted by relatively small failure probabilities (Figure 6.14, Figure 6.21), about 1-2 orders of magnitude smaller than those associated with bending or tension-induced shear failure. Hence, the contribution of the compression shear failure mode to the total, accumulated failure probability of a structural member, P_f (4.7), under consideration of all n_k failure modes the member might be subject to, is comparatively small, and so is its contribution to the structure-related risks. The expected increase of the member failure probability due to the slight reduction of γ_c from 1.3 to 1.2 will not alter this conclusion. The impact of this reduction on the principal results of the present study is hence small.

Reduction factor K

In Figure 4.19 it was shown that the formulation of coefficient K (4.47), to be factored into the shear resistance $V_{R,c}$, in order to account for the stress state in the compression chord, has experienced an increase from the herein-used standard EHE-98 [83] to the equivalent standards in force [72, 84]. This increase will affect the reliability level associated with a failure of the compression strut of the columns. In Figure 6.21 (right) it was shown that, except for very small coefficients K , the larger K the lower the reliability level. Hence, for the majority of the columns analysed here, a strict design according to the

codes in force would probably imply lower reliability levels than obtained in the present study. In this sense, the conclusion is similar as formulated before concerning the reduction of γ_c . The fact that the compression-controlled shear failure of the columns is comparatively unlikely implies that its contribution to the structure-related risks is relatively small. Thus, the increase of coefficient K has only a minor impact on the principal results of the present study.

Design eccentricity for design of columns

In the present study, the design of the RC columns for M-N interaction was based on an approximate design procedure, where both first- and second order effects are accounted for by means of eccentricity e_{tot} , inferred from a nominal curvature (Figure 4.13). In the current version of the Spanish design code for structural concrete, EHE-08 [72], this procedure was withdrawn and another, approximate design procedure, based on a fictitious eccentricity, provided in its place. Also the Eurocode [84] contains different approximate design methods, based on nominal curvature- or nominal stiffness. The general aim of such simplified methods is to provide closed-form solutions to designers which can be applied in the context of an ordinary cross-sectional design according to the UMLS (section 4.6.3) under use of the corresponding strain limits provided by the codes (Figure 4.13). Necessarily, simplified design approaches provide generally more or less conservative results. However, it is of minor importance to the present study if the design procedures contained in the current codes [72, 84] prescribe a slightly higher or lower degree of conservatism in comparison to the method used herein. The important point is that all these methods, if used in conjunction with the code-based design values for action effects and resistance in accidental situations characterised by M-N paths similar to those investigated here, might lead to an erroneous estimation of the failure point location, and thus to irrational design solutions. As stated before, the use of more realistic values for the material properties could be a possible solution to this problem.

6.2.5 Final remarks

It should be recalled that the reliability indices described in sections 6.2.2 and 6.2.3 must be understood as *notional values*. For several reasons, they do not reflect the actual member failure probabilities even though the probabilistic models used represent the physical properties of the structural design variables modelled. Among these reasons is the fact that the failure probabilities do not account for the occurrence of *human error*, which is often in the origin of structural failure [3, 4, 7]. In the present context, human error might be related, for instance, to negligence or ignorance of the explosion hazard, leading to an omission of risk-reducing measures (section 4.1.2). The results described before show that such an omission could entail member reliabilities that fall significantly below the level implicitly required by the codes for the accidental design situation.

Moreover, it must be recognized that the present study focusses on strictly designed cross-sections and members and hence does not reflect failure frequencies of *conservatively designed members in practice*. This is especially relevant for the shear failure modes. As pointed out in section 4.7.3, the strict design of the compression strut at the support cross-sections of both beams and columns requires in many cases only very small cross-section widths b_w , far smaller than required for purpose of constructive feasibility. Hence, from this point of view, actual failure probabilities might be far lower than the notional values obtained here. The same is true for the failure of the tension tie of the columns. Regarding this failure mode it must be kept in mind that, in cases characterized by high axial compression forces, a strict design is only possible if the contribution of the concrete to the resistance mechanism (V_{cu}) is being reduced by an increase of the longitudinal reinforcement A_s (section 4.7.2). However, for some members this leads to unrealistically high and practically unfeasible reinforcing degrees. For realistic reinforcing

degrees, on the contrary, compression axial forces would indeed contribute to the resistance of the tensile tie and thus cross-section design would be conservative with the corresponding consequences on the reliability level.

Regardless of their notional character, the failure probabilities obtained are useful when interpreted and employed in the context of a comparative study, always keeping in mind the assumptions adopted for their determination.

6.3 Accumulated failure probabilities

6.3.1 Introduction

In section 4.2.3, the accumulated failure probability of a structural member P_f was defined by (4.7) as the sum over the n_j individual failure probabilities associated with each of the n_l load arrangements and n_k failure modes the member might be subjected to. Here, a single failure mode ($n_k=1$) and two relevant load arrangements ($n_l=2$), a persistent load arrangement and an explosion-induced load arrangement, will be assumed. Hence, based on (4.7), the implicitly admissible, accumulated probability of failure $P_{f,adm}$ is expressed per (6.4) as the sum of contributions $p_{f,adm,PER}$ and $p_{f,adm,EX}$ in consequence of the persistent and the accidental (explosion) load arrangement, respectively. It is assumed that other potential accidental events, such as earthquakes, fire or impact, are not relevant to the reliability level corresponding to the member and member failure mode in question.

$$P_{f,adm} = p_{f,adm,PER} + p_{f,adm,EX} \quad (6.4)$$

In the context of a strict code-based design procedure ($E_d=R_d$), the fact that two different load arrangements are considered, implies a check for the mandatory design situation (Figure 4.1), which according to the employed design rules, demands the higher member resistance. In other words, only for one out of both considered load-arrangements, the resistance might be strictly adjusted to the corresponding load effects, whereas for the other the design will be unavoidably conservative. Consequently, it would be expected that the failure probability associated with the non-mandatory design situation would be significantly smaller than compared to the probability of failure due to the load arrangement governing member design. In the present context it was argued in section 4.1.2, that if member design is based on the accidental design rules (Figure 4.1, path a), the provided resistance will probably largely exceed the action effects prevailing under normal building use conditions. For these situations, it seems reasonable to assume, that the associated failure probability (path a_2) is negligibly small in relation to the likelihood of explosion-induced member failure (path a_1).

The contrary situation does not seem to be evident, however. As stated in section 4.1.2, if the persistent situation is mandatory for member design (Figure 4.1, path b), it cannot be presupposed that load arrangements associated with a potential accidental explosion event, will be insignificant to member reliability. In spite of the fact that the design code rules might demand a higher member resistance under normal loading conditions, the large uncertainties associated with the explosion load and the unknown consequences of associated dynamic effects on the reliability level do not allow for such a premature conclusion. It was concluded that further analysis would be required to this end.

This subject will be addressed in the following subsection 6.3.2. Precisely it will be analyzed to which extent component $p_{f,adm,EX}$ contributes to the accumulated failure probability, $P_{f,adm}$. The member failure probabilities under persistent load arrangements, $p_{f,adm,PER}$, required for this analysis, have been adopted, where available, from prior studies [78, 99]. Section 6.4 closes with some concluding remarks on the impact of the obtained results on the further development of the study.

6.3.2 Analysis

Figure 6.23 (left) shows the ratio between the admissible member failure probability $p_{f,adm,EX}$ associated with positive bending failure of the 486 studied RC beams induced by explosion load arrangement $p_{EX(+)}$ (Figure 4.8) and the accumulated failure probability $P_{f,adm}$ (6.3) corresponding to this failure mode. The results are presented as a function of accidental load ratio v_{acc} and distinguished according to the mandatory design situation, which is governed in 273 cases by the persistent- and in 213 cases by the accidental load arrangements. The Figure confirms that if the accidental situation is mandatory (dark dots), the explosion load arrangement clearly dominates the member failure probability, i.e., persistent load arrangements under normal use conditions do, as expected, not contribute significantly to $P_{f,adm}$. On the contrary, if design is based on the persistent situation (clear dots), what is the case for smaller v_{acc} , a non-negligible fraction of the corresponding 273 cases is characterized by an important contribution of the explosion-related failure probability $p_{f,adm,EX}$. Ratios $p_{f,adm,EX}/P_{f,adm}$ higher than 0.9 show that in many cases $p_{f,adm,EX}$ corresponds actually to the dominant component, despite the fact that the corresponding load arrangement was not found to be decisive in the member design stage. As mentioned before, this is primary attributable to the large uncertainties associated with the explosion load.

In any case, the described finding is relativized when comparing the accumulated failure probability $P_{f,adm}$ to the value $p_{f,t,code} = 7.2 \cdot 10^{-5}$, associated with the nominal target reliability index established in the employed design codes, $\beta_{t,code} = 3.8$ ($T_{ref} = 50$ years). This comparison is shown in Figure 6.23 (right). It can be observed, that, with a few exceptions, $P_{f,adm}$ remains below $p_{f,t,code}$ if the design is based on the persistent design situation. Only if the accidental situation is mandatory, the nominal target value may be exceeded. However, this is attributable, almost exclusively, to the contribution of the explosion scenario since as seen in Figure 6.23 (left), in these cases the persistent load scenario does not contribute significantly to $P_{f,adm}$.

Similar conclusions are expected for the shear failure modes of the beams, which might likewise be triggered by accidental or persistent load arrangements. On the contrary, it is recalled that negative bending failure of the beams is exclusively associated with explosion-induced load arrangement $p_{EX(-)}$.

Figure 6.23 Ratio of failure probability $p_{f,adm,acc}$ due to accidental load scenarios and accumulated failure probability $P_{f,adm}$ (left) and accumulated failure probability $P_{f,adm}$ (right); Positive bending failure of beams

Another failure mode that, according to the assumptions met in the present study, can be potentially triggered by both accidental and persistent load arrangements is the failure of the intermediate cross-section of the columns due to M-N interaction. For this failure mode, Figure 6.24 (left) shows by analogy to Figure 6.23 (left), ratio $p_{f,adm,EX}/P_{f,adm}$ between the member failure probability of 9504 RC columns

subjected to explosion load arrangement $N_{EX(-)}$ and the accumulated failure probability for the M-N-induced failure mode. For sake of simplicity, only 1/3 of the 28512 studied columns are represented, corresponding to members situated in buildings characterized by an explosion venting element resistance pressure of $p_{stat} = 5 \text{ kN/m}^2$. The results are presented as a function of the normalized accidental design eccentricity $(e/h)_{acc,d}$ and distinguished according to the mandatory design situation, characterized in 7951 cases by the persistent- and in 1553 cases by the accidental load arrangements. The figure confirms the findings for the beams. As expected, the persistent loads do not contribute significantly to $P_{f,adm}$ when the accidental situation is mandatory (dark dots). In the contrary case, if design of the members is based on the persistent situation (clear dots), the probability due to the explosion-induced load arrangement $p_{f,adm,EX}$ contributes significantly to $P_{f,adm}$ for a large number of cases. In many of these members, $p_{f,adm,EX}$ exceeds $p_{f,adm,PER}$ by several orders of magnitude. Nevertheless, also here, the target ceiling $p_{ft,code}$ is, with a few exceptions, not exceeded for such members, what relativizes the importance of this finding (Figure 6.24, right).

Figure 6.24 Ratio of failure probability $p_{f,adm,acc}$ due to accidental load scenarios and accumulated failure probability $P_{f,adm}$ (left) and accumulated failure probability $P_{f,adm}$ (right); Failure due to M-N interaction of columns (only members characterized by $p_{stat} = 5 \text{ kN/m}^2$).

6.3.3 Concluding remarks

The findings from the preceding subsection are relevant to the further developments of the study. Precisely, such developments must account for the fact that a particular structural member may not only fail in consequence of different failure modes, but that certain of these modes might be triggered by different load arrangements, under similar failure probabilities. In particular, this might be the case when the normal load conditions determine the design of the member although it is potentially exposed to the hazard *gas explosion* and the associated action effects. For such members, the risk analysis (Chapter 8) should consider both persistent and accidental load arrangements. Moreover, the results confirm that if the accidental design situation is mandatory, the contribution of persistent loads to the failure probability might be neglected in the risk computation without introducing a significant error.

6.4 Summary

Based on the defined LSF's for the members and failure modes to be analyzed (section 4.6), the strict member design based on these LSF's (section 4.7), the characterization of the basic variables involved (section 4.8), including a quantification of associated dynamic effects (Chapter 5), Chapter 6 addresses procedural aspects and the results of the FORM analysis carried out to establish the reliability level implicitly required by the employed design codes.

Section 6.1 describes the specifically developed reliability procedure for the analysis of column reliability under M-N interaction. The objective of this procedure is the determination of the strain distribution corresponding to the most likely-failure point on the limit state surface, also known as FORM-design point (Figure 2.12). This can be achieved by coupling the FORM analysis to the deterministic solution algorithm, developed in section 5.3.4 to determine the failure mode and the associated failure point location on the M-N interaction diagram. An iterative analysis procedure that enables this coupling has been developed (Figure 6.2, Figure 6.3) and applied, in a previous work [99], to a set of columns under gravitational load exposure. After a generic description of this procedure (section 6.1.2), its implementation for the herein considered explosion-exposed members is shown by means of an example (section 6.1.3, Figure 6.4). It is observed that the mean value analysis of the M-N exposed columns provides failure point estimations that approach the FORM design point solution in a sufficiently exact manner. This suggests that the associated strain distribution could be employed for purpose of a direct reliability estimate, i.e. a unique FORM analysis without the need to follow the iteration-based procedure developed. On the contrary, the erroneous estimation of the failure point location in the code-based design (section 5.4), rules out the use of the corresponding design strain distribution for such a purpose, since this could entail completely distorted reliability levels. Previous work confirms that this a load-path specific problem. Under the assumption of a constant eccentricity, the code-based design procedure and the FORM analysis delivered very similar design points and associated strain distributions [99].

The FORM analysis delivers the admissible, conditional failure probabilities $p_{f,adm|EX}$ for the defined structural members, given the occurrence of a gas explosion within the established reference period T_{ref} (50 years). Taking account of the corresponding occurrence probability $p(EX)$, estimated according to the reported findings in section 3.1.2, the implicitly admissible failure probabilities $p_{f,adm}$ are then established according to (4.6). The probabilities $p_{f,adm}$, which represent the occurrence probabilities p_j for a specific hazard scenario j (section 4.2), can finally be converted to the implicitly required reliability indices β_{req} using the inverse standard normal distribution (section 6.2.1)

The results obtained are presented and discussed in sections 6.2.2 and 6.2.3. The general need for accidental design provisions associated with gas explosions is confirmed. It is shown that in many practically relevant situations, the accidental design rules might demand a higher member resistance than provided in an ordinary design for persistent situations. Ignoring this, the reliability level achieved falls below the level implicitly required by the design codes for the accidental situation. Moreover, the analysis reveals that this level is generally subject to a high scatter. Important differences are observed when comparing the obtained reliability indices for different members and, especially, for different member failure modes. In many cases, the nominal target ceiling ($\beta_{t,code} = 3.8$) is not met. The tension tie shear failure mode of the studied members turns out to be especially critical, while the failure of the compression strut is comparatively unlikely (Figure 6.14, Figure 6.21). These results corroborate the findings of the previous deterministic analysis (Chapter 5).

A closer analysis of the sensitivity coefficients α_i associated with the FORM design point vector $\mathbf{x}^*$ reveals that, almost irrespective of the member type and failure mode, the uncertain explosion-induced action effects dominates the failure probabilities of members whose design is governed by the accidental situation. The influence of geometrical- or material variables, as well as static loads, on the likelihood of an explosion-induced collapse of such members is comparatively small, wherefore they could be treated as deterministic quantities in the reliability calculations.

An analysis of the ratio between the resistance R^* of the beams, computed from the individual components x_i^* of vector $\mathbf{x}^*$, to the mean resistance R_m found in the deterministic analysis, corroborates this

finding. It is shown that ratios R^*/R_m approach unity with increasing influence of the explosion load on the member reliability (Figure 6.8, right, Figure 6.13, Figure 6.16, right). This is attributable to the simultaneous decrease of the impact of the resistance variables (geometry and material parameters) on the member failure probability, what implies that the corresponding design point coordinates x_i^* approach the respective mean values. On the contrary, the ratios between R^* and the code-based design resistance R_d are found to deviate from unity with rising influence of the explosion load (Figure 6.8, left, Figure 6.12, Figure 6.16, left). In accordance with the findings from Chapter 5, these observations question the appropriateness of the current definition of design material parameters in the context of accidental design rules. The results confirm that a more realistic representation of these parameters, including strain rate effects, seems to provide a more rational solution for accidental design situations involving gas explosion-induced load arrangements. The same conclusion is obtained for column failure induced by M-N interaction, drawn from a comparison of the normalized eccentricities $(e/h)^*$ to, respectively, the mean values $(e/h)_m$ and the code-based design values $(e/h)_d$ (Figure 6.19). The comparison confirms that the irrationalities associated with the failure point location, detected in Chapter 5, could probably be circumvented in most cases by shifting the design values for the material parameters towards more realistic limits.

Finally, section 6.3 investigates into the accumulated failure probabilities of structural members, taking account of the fact that, in addition to the accidental load combination, their failure might be triggered by persistent load arrangements. The analysis shows that, depending on the particular conditions prevailing in each case, persistent- or accidental load arrangements might be dominant for a certain member failure scenario, while in other cases they might equally contribute to the accumulated failure probability (section 6.3.2). In particular, this latter situation might occur when the normal load conditions are mandatory for design of the member. For such members, a dominant load arrangement cannot be established a priori, wherefore the risk analysis (Chapter 8) should consider both possibilities (persistent- and accidental load arrangements). On the contrary, if the accidental design situation is mandatory, the contribution of persistent loads to the failure probability might be neglected in the risk computation without introducing a significant error. In this case, the provided member resistance largely exceeds the action effects associated with the normal use conditions of the buildings.

Chapter 7 Consequence models

The estimation of gas explosion-induced structural risks to persons requires the disponibility of an appropriate consequence model. The derivation of such a model is subject of *step 6* of the analysis procedure (section 4.3, Figure 7.1). A basic requirement to the model is that it can be properly linked to the established mathematical framework for risk quantification (section 4.2), what will be briefly addressed in section 7.1. In the following, the model development based on statistical treatment of compiled data on explosion-induced collapse incidents will be described. Section 7.2 characterizes the general features of the established database. Descriptive statistics of the recorded incidents are subsequently provided in section 7.3, followed by classification of data according to its relevance to the derivation of the consequence model. In section 7.4, the most relevant data fields are complemented by means of information provided by the National Statistics Institute (INE) [112]. Finally, section 7.5, addresses the multiple linear regression analysis performed in order to obtain the desired model, including a discussion of the obtained results and a comparison to prior models.

Figure 7.1 Procedural flow-chart

7.1 Context

Gas explosions in buildings might entail a variety of undesired consequences, among them harm to persons and monetary losses. The former include loss of live and injury and may be traced back either to the direct effects of the explosion on a person, for instance due to the direct exposure to the pressure wave, or to its effects on the building structure with the corresponding probability of structural collapse and the associated risks to personal integrity.

In keeping with the general scope of the study (section 1.2), the focus of the consequence analysis is placed on the latter type of personnel consequences, attributable to structural collapse scenarios. In particular, the analysis aims at quantifying loss of live as a function of a parameter that characterizes the damage extensions associated with such scenarios. Previous studies suggested the building net-

room area affected by the collapse (A_{col}) to be appropriate for this purpose [9, 10, 13]. Based on the established hypothesis for description of a structural system collapse, summarized in section 4.5.4, this area can be defined in terms of the geometrical building parameters introduced in section 4.4.2 (Figure 4.4) and the location of the member whose failure triggers the system collapse. The following expressions apply for determination of area $A_{col,j}$ due to a specific beam- (7.1) or column- (7.2) failure scenario j , according to its definition within the mathematical framework for risk quantification established in section 4.2.3:

$$A_{col,j} = C_b \cdot l^2 \quad (7.1)$$

$$A_{col,j} = C_c \cdot l^2 \cdot (n_s + 1 - n_{s,f}) \quad (7.2)$$

Where:

- C_b : Constant accounting for location of collapse-triggering beam: $C_b = 0.5$ for roof beams situated on the building perimeter; $C_b = 1.0$ for roof beams situated inside the building perimeter or floor beams on the building perimeter; $C_b = 2$ for floor beams situated inside the building perimeter.
- C_c : Constant accounting for location of collapse-triggering column; $C_c = 0.5$ for columns situated on the corners of the building perimeter; $C_c = 1.0$ for columns situated on the building perimeter except the corners; $C_c = 2$ for columns situated inside the building perimeter
- l : Distance between axes of columns in longitudinal sense of the building (Figure 4.4)
- n_s : Number of storeys excluding ground floor and roof (Figure 4.4)
- $n_{s,f}$: Number of storey where column failure occurs

By means of equations (7.1) and (7.2) the number of fatalities N_j due to a specific member failure scenario j can be readily estimated provided a consequence model is available which relates N_j to the corresponding area affected by the collapse, $A_{col,j}$. The tasks performed in connection with the development of such a model are summarized in the following sections.

7.2 Data compilation

7.2.1 Introduction

In the framework of a press survey, the online archives of the main Spanish national newspapers were systematically searched for information on explosion-induced structural collapse events in Spain and other western countries. Occasionally, international and local online media sources were also consulted in case information on specific incidents was to be amplified. The data collection included all type of gas explosions in buildings, as well as explosions involving other explosive agents, as for instance, dust or pyrotechnical material. Bomb explosions were excluded from the survey.

A database was established to compile the information. The format used for data collection is shown in Annex H.1. Four different data domains can be distinguished:

- the affected building (section 7.2.2)
- the incident (section 7.2.3)
- the incident consequences (section 7.2.4)

- additional data (section 7.2.5)

7.2.2 Building data

The compiled data corresponding to the affected building comprises the following data fields:

- Use type (residence, hotel, office, shop, etc.)
- Use category (according to [11])
- Location (direction, country)
- Year of construction
- Dimensions (net room area A , n° of storeys n_s , number of housing-units, n_{hu})
- Structural attributes (structural system, construction materials)

Especially interesting for purpose of the study are the building dimensions, characterized here by building net room area A , n° of storeys n_s or n° of housing-units n_{hu} . While n_s or n_{hu} are often indicated in the consulted sources, information on the building net room area A is normally not provided. In some cases, A can be roughly estimated from photographs or videos added to the press articles.

Other relevant information for estimating the consequences of structural collapse is related to the structural attributes of the building, such as the structural system or the construction materials. However, such information is generally scarcely reported in the press. In some cases, photographs or videos provide some insight into the structural characteristics of the building.

7.2.3 Incident data

The characteristics of the explosion incident and its effects on the building are recorded by means of the data fields listed below:

- Date of the incident
- Involved inflammable substance (natural gas, propane, butane, etc.)
- Cause (triggering event) and cause category (Cat. C):
 - o Cat. C1: Intentionally caused
 - o Cat. C2: Accidentally triggered without human intervention
 - o Cat. C3: Accidentally triggered through human intervention
- Qualitative damage description and damage category (Cat. D):
 - o Cat. D1: Light damage (broken windows, doors, furniture, light partition walls, etc.)
 - o Cat. D2: Medium damage (failure of non-loadbearing walls, claddings, etc.)
 - o Cat. D3: Severe damage (collapse of load-bearing structural members)
- Quantitative damage description by means of the following parameters:

- Building area generally affected by the blast wave and building area affected by the collapse, respectively, A_{bw} and A_{col}
- Number of storeys generally affected by the blast wave and number of storeys affected by the collapse, respectively, $n_{s,bw}$ and $n_{s,col}$
- Number of housing-units generally affected by the blast wave and number of housing-units affected by the collapse, respectively, $n_{hu,bw}$ and $n_{hu,col}$
- Occupancy at the time of the incident
 - Building occupancy, Ocu
 - Occupancy of the area affected by the collapse, Ocu_{col}

Three different categories were established to classify the cause of the incident. Intentionally caused gas explosions (Cat. C1), in many cases related to suicide, are distinguished from accidental incidents. The latter are further distinguished into events triggered without direct human intervention (Cat. C2), such as unforeseen technical defects, and those that involve some kind of direct human error (Cat. C3), as for instance, erroneous maintenance- or repair interventions in gas conducts or appliances.

The present study focusses on explosion incidents involving structural collapse. Most incidents involve minor structural damage, however (section 3.1.2). Three categories are established in the present study in order to classify explosion-induced damage to the building. Light damage (Cat. D1), such as broken furniture or failure of doors and windows (Figure 7.2, left), is distinguished from medium (Cat. D2) or severe (Cat. D3) damage to building components. Medium damage is associated with failure of typical non-loadbearing elements, such as interior brick walls or masonry façade panels (Figure 7.2, middle). Of primary interest here are incidents assigned to Cat. D3, characterized by collapse of loadbearing structural members, such as walls, columns, beams or slabs (Figure 7.2, right, Figure 1.2).

Figure 7.2 Examples for incidents involving light (left), medium (middle) and severe (right) damage

It should be noted that extremely severe collapse scenarios, often associated with a high number of fatalities, are amply documented in the media. An example is the collapse of a residential building in Palencia (Spain) in 2007, involving 9 fatalities and more than 30 injuries (Figure 1.2, right). The case occupied the media for several days, what contributed to a large amount of available information, including exceptionally detailed descriptions of the damage extensions (Figure 7.3). On the contrary, less media-attractive incidents involving collapse of far smaller extensions, and only few or no fatalities, are rather scarcely addressed in the media, lacking detailed damage descriptions in many cases. Moreover, such descriptions are generally subjective, subject to the perception of damage by the informing journalist. For instance, a building described in a press release as “completely destroyed” does not necessarily indicates a total building collapse, but might refer to a destroyed flat without failure of any load-

bearing structural members, such as shown in Figure 7.2 (middle). Hence, in order to reliably identify a structural collapse scenario and to perform a trustworthy, quantitative estimate of its extensions, the available information must be cautiously analyzed and evaluated, requiring in many cases a tedious comparative study of several data sources, including local media if appropriate.

Figure 7.3 Detailed description of damage extensions after collapse of a residential building in Palencia [203]

The damage extensions of an incident are quantified in terms of the affected building area, the n° of storeys or the n° of housing-units. A distinction is made for building portions affected by the blast wave in general, and collapsed building parts in particular (damage category D3). Since, as stated before, detailed information on damage extensions is scarce in many cases, though required for the present study, some estimates are necessary. This applies especially to damage parameter A_{col} (section 7.1), which is reliably indicated only in exceptional cases (e.g. Figure 7.3). In case A_{col} can neither be estimated from available images or videos attached to the press articles, it might be deduced from equation (7.3) as a function of the total building area A and the ratio between the number of collapsed- and total storeys $n_{s,col}/n_s$ (or housing-units, $n_{hu,col}/n_{hu}$), in case these parameters are available.

$$\frac{A_{col}}{A} = \frac{n_{s,col}}{n_s} = \frac{n_{hu,col}}{n_{hu}} \quad (7.3)$$

The n° of persons present at the time of the incident, in the building in general, and on the area affected by the collapse (A_{col}) in particular, is referred to as building- and collapse area occupancy, respectively, Ocu and Ocu_{col} . While the total building occupancy (Ocu) can often be extracted or estimated from the available information, precise data on Ocu_{col} is normally lacking. As a rough estimation, Ocu_{col} can be inferred from (7.4) as a function of the total building occupancy Ocu and ratio A_{col}/A , substitutable likewise by ratio $n_{s,col}/n_s$ or $n_{hu,col}/n_{hu}$ according to (7.3).

$$\frac{Ocu_{col}}{Ocu} = \frac{A_{col}}{A} \quad (7.4)$$

7.2.4 Consequence data

In keeping with the scope of the study, the data compilation on the incident consequences focuses on damage to persons. However, for it might be of interest for future studies, data on economic consequences is also recorded. The corresponding data fields are listed below:

- Number of fatalities (N) and fatalities attributed to structural collapse (N_{col})
- Number of severe injuries (I_{sev}) and severe injuries attributed to structural collapse ($I_{sev,col}$)
- Number of slight injuries (I_{sli}) and slight injuries attributed to structural collapse ($I_{sli,col}$)
- Number of total injuries (I) and total injuries attributed to structural collapse (I_{col})
- Number of disappeared persons (D)
- Direct economic consequences
- Indirect economic consequences

A distinction is made for consequences to persons in general, and those that might be traced back to a structural collapse scenario, of primary interest for the present study. This distinction is necessary to account for deaths or injuries attributable to the direct exposure to the explosion effects, such as the released heat or the pressure wave. Moreover, explosions frequently trigger fire, what entails additional risk to persons, e.g. due to suffocation. In many cases, the mentioned distinction is not straight forward, however. An extremely detailed and cautious analysis of the available information is necessary to judge whether injury or death of a person can be attributed to structural collapse or not. Even then, some simplifying hypothesis are required in most cases. As a rule, victims are traced back to structural collapse if there are reliable indications that the person was located on the area affected by the collapse (A_{col}) at the time this collapse occurred and not directly exposed to the explosion effects.

Direct economic consequences refer to costs that are directly associated with the explosion-induced damage to the building, e.g. repair- or demolition costs. Other monetary consequences are designated as indirect costs, including, e.g. costs associated with emergency services or the temporary accommodation of persons after a building evacuation.

7.2.5 Additional data

In addition to the information described above, the database contains the following fields:

- Data source(s)
- Comments
- Relevance

The data field *Comments* contains the hypothesis and assumptions that might have been adopted to estimate the different building-, incident or consequence-related parameters. Moreover, doubtful information, for instance due to contradictory indications in the same or different data sources, is highlighted here. Finally, any interesting complementary information, not included in the previous data fields, is compiled.

The last data field refers to the relevance of a specific incident regarding the inference of a consequence model. In particular, an incident is considered potentially relevant to this purpose if the following conditions are fulfilled:

- The building is classified under use category A or B according to [82]

- The incident is classified under damage category D3 (collapse due to failure of loadbearing structural member)
- The number of persons on the area affected by the collapse (Ocu_{col}) is different from zero
- The number of fatalities due to the collapse (N_{col}) is known
- The compiled information seems reliable

7.3 Data evaluation

7.3.1 Introduction

The systematic data collection according to the descriptions in section 7.2 concluded with a total of 233 recorded explosion incidents. Descriptive statistics are used in first place to provide an overview on the compiled data (section 7.3.2). In the following, the incidents that are potentially relevant to the specific purpose of deriving a consequence model are screened out and statistically described in terms of selected parameters (section 7.3.3).

7.3.2 Overview

Figure 7.4 (left) shows a distribution of the 233 compiled incidents according to the country where these incidents occurred. The figure reveals that the vast majority of the incidents, about 77%, occurred in Spain what is due to the fact that mainly national data sources were analyzed in the press survey (section 7.2.1). Moreover, the database contains incidents in Germany (7%), the U.S. (4%), France (3%), Italy (3%) and other western countries (6%).

In the right part of Figure 7.4, it can be observed that, unsurprisingly, most of the compiled incidents occurred in residential buildings (82%). In addition, the database includes a number of explosions in shopping areas (8%), areas for storage and industrial use (7%), office areas (2%) and areas where people may congregate (1%). It should be noted that a large part of the explosions in shopping areas is attributable to accidents in restaurant kitchens.

Figure 7.4 Distribution of compiled incidents per country (left) and per building use category (right)

Figure 7.5 (left) illustrates the distribution of compiled incidents per time-period during which the incidents occurred. The time-periods are represented by four decades from 1960 to 2009 and a half-decade from 2010 to 2014. It can be observed that the more recent the considered period the higher is the

number of corresponding incidents. Rather than an increasing incident occurrence rate, this result manifests the increasing availability of information in the recent years. Especially remarkable is the noticeable increase from the end of the last to the beginning of the current century, coincident with the “flash-over” of the internet era.

On the right, Figure 7.5 distinguishes the incidents according to the inflammable agents involved. It is revealed that many of the compiled incidents are related to cylinder gas (>33%), such as butane and propane. Natural gas was in the origin of at least 11% of the incidents, another 18% correspond to other substances, such as town gas, dust or pyrotechnical material. In most cases, the explosive agent is unknown or unspecified (38%) - very frequently, gas is identified in the consulted sources as the accident-causing element without further specification of the gas type, however.

Figure 7.5 Distribution of compiled incidents per time-period (left) and per gas-type involved (right)

The distribution of the principle causes of the recorded explosion incidents can be drawn from Figure 7.6 (left). More than 40% of the incidents are accidentally triggered, in most cases without an apparent human intervention (>33%), as represented by cause category C2 (section 7.2.3). Intentionally caused explosions (Cat. C1) constitute more than 12% of the compiled events. In most incidents, the available information is too scarce to draw reliable conclusions on the explosion causes (46%).

Figure 7.6 (right) shows that more than 55% of the gathered incidents correspond to the severe damage category D3 (section 7.2.3), involving structural failure. Less important for the present study are incidents recorded under Cat. D2 (medium damage, >27%) or D1 (light damage, >7%). In about 11% of the recorded cases, the damage category could not be reliably established.

Figure 7.6 Distribution of compiled incidents per incident cause category (left) and per damage category (right)

Figure 7.7 contains information on the consequences to persons induced by explosion events in buildings. On the left, it can be observed that nearly 60% of the compiled incidents involve fatalities. Around 7% entailed more than 10 fatalities and still 3% more than 20 fatalities. The mean value is $\mu_N \approx 5$ fatalities per explosion incident, the corresponding coefficient of variation, $\text{CoV}_N = 645\%$, denotes large associated scatter. However, both values considerably diminish if the explosion-induced collapse of a commercial center in Korea [204] with around 500 fatalities is disregarded in the statistical analysis - in that case, $\mu_N \approx 3$ and $\text{CoV}_N = 241\%$.

The right part of Figure 7.7 displays the distribution of injuries. It is shown that explosion incidents without injuries are seldom ($\approx 13\%$). Moreover, it is stressed that around 12 % of the recorded explosions entailed 20 or more injuries, and nearly 3% more than 40. Consequently, mean value and associated scatter are relatively high, with $\mu_I \approx 13$ injuries and $\text{CoV}_I = 481\%$, respectively. Disregarding the 937 injuries in the above-mentioned incident in Korea [204], these values reduce to $\mu_I \approx 9$ and $\text{CoV}_I = 177\%$.

Figure 7.7 Distribution of compiled incidents per number of fatalities (left) and injuries (right)

7.3.3 Relevant incidents

Screening

After the generic data description presented in the previous section, the 233 compiled explosion events are analyzed in more detail regarding their relevance to the establishment of a consequence model. In line with the requirements defined in section 7.2.5, the model should be applicable to incidents in residential- or office buildings, corresponding to consequence class CC2 [11], which account for about 84 % of the 233 compiled cases (Figure 7.4), i.e. 196 incidents. Out of these, 112 are classified as severe incidents (damage category D3, section 7.2.3), involving collapse of loadbearing structural members and hence relevant to the purpose of the present investigation.

Further to the criteria established in section 7.2.5, it is required for a specific incident to be classified as potentially relevant to the study, that the occupancy of the area affected by the collapse ($O_{cu_{col}}$) is different from zero at the time the collapse occurs. However, in four cases out of the remaining 112 this is not the case, among them delayed collapse incidents that occur due to explosion-induced fire scenarios, after people have already been evacuated from the building. Finally, one out of the remaining 108 cases has to be ruled out due to considerable uncertainties associated with the available information.

In summary, a total of 107 incidents are identified as potentially relevant to the deduction of a consequence model.

Structural attributes

Figure 7.8 (left) shows a distribution of the 107 relevant incidents according to the construction materials of the collapsed structure. It can be observed that in almost 3 out of 4 of these incidents no information at all is available on this matter. In at least 15% of the collapse-incidents, failure of reinforced concrete members was involved, although it could not always be clearly established whether this failure had triggered the collapse or if, on the contrary, it was a consequence of it. An example, although not included in the database, for it occurred recently in 2016, is the collapse of a residential building in Tenerife. Figure 7.8 (right), shows the rubble of the collapsed building. Remaining parts of failed reinforced concrete columns can be observed, although no information is available which could answer the question if this failure is attributable to the explosion effects, or if it might have occurred during the progressive collapse of the building, e.g. in consequence of an uncontrolled load distribution due to collapse of the upper storeys and/or impact forces.

In other cases, it was observed that reinforced concrete slabs or beams collapsed after losing their support in form of a loadbearing brick-wall, as possibly happened in the Palencia-incident (Figure 1.2, Figure 7.3). The analysis of the 107 potentially relevant incidents revealed that failed brickwork or masonry loadbearing elements were involved in at least 10%. Moreover, in another 4%, failure of structural timber members was observed.

In general, the uncertainties associated with the involved structural systems and construction materials in the collapse scenarios are extremely high due to the scarce and imprecise available information. For this reason, such information cannot be accounted for the derivation of a consequence model within the present study.

Figure 7.8 Distribution of 107 potentially relevant incidents per construction material (left); Collapsed building in Tenerife [205] (right)

Selected parameters

Compiled data corresponding to selected parameters characterizing the 107 potentially relevant collapse incidents is provided in Annex H.2. The mentioned parameters are related to the building dimensions (area, A , n° of storeys, n_s and housing-units, n_{hu}), the extensions of the collapse (area affected by the collapse, A_{col} , collapsed n° of storeys, $n_{s,col}$ and housing-units, $n_{hu,col}$), the number of persons present on A and A_{col} at the time of collapse (Ocu , Ocu_{col}) as well as the corresponding number of fatalities (N_{col}). Table 7.1 provides summarizing descriptive statistics.

Attention should be drawn to the relatively large n° of records corresponding to the total- and collapsed n° of storeys, respectively, n_s and $n_{s,col}$, available in around 80% of the 107 incidents. On the contrary, data on the areas is rather scarce. For instance, the area affected by the collapse (A_{col}), fundamental for the development of the consequence model (section 7.1), is available in only 21% of the records. This in spite of the attempts made to estimate A_{col} from equation (7.3), in connection with information on the number of collapsed storeys $n_{s,col}$ or housing-units, $n_{hu,col}$. Since, as Table 7.1 shows, data on the total building area A is likewise scarce (15%), such estimations were practically feasible only in few cases.

The occupancy of the area affected by the collapse (Ocu_{col}), on the contrary, could be relatively often deduced from the available data sources (54%), in many cases with help of relationship (7.4), in conjunction with data on the total building occupancy (Ocu) and the ratio between the collapsed- and the total n° of storeys ($n_{s,col}/n_s$) or housing-units ($n_{hu,col}/n_{hu}$). It should be noted that the statistics provided in Table 7.1 do not include data on minimum or maximum occupations of the building (Ocu), or of the area affected by the collapse (Ocu_{col}). However, such data, which in many cases could be deduced from the analyzed press releases, is useful for the validation of statistical, complementary data, as will be specified in section 7.4.3.

Table 7.1: Statistical evaluation of different parameters corresponding to 107 potentially relevant incidents

Parameter	A [m ²]	n_s	n_{hu}	A_{col} [m ²]	$n_{s,col}$	$n_{hu,col}$	Ocu *	Ocu_{col} *	N_{col}
N ^o of records	16	91	20	23	81	25	48	58	107
% of total	15	85	19	21	76	23	45	54	100
μ	434	3.3	12.5	268	2.4	3.8	17	11	3
σ	433	2.8	5.9	396	1.7	5.0	26	18	5
CoV [%]	100	85	47	147	72	131	148	167	163
Max	1864	22	24	1864	11	16	136	88	27
Min	54	0	6	15	1	1	1	1	0

* Excluding records of minimum or maximum occupancies

Table 7.2 reveals that the mean building area affected by the collapse of the 107 incidents is situated in $A_{col} = 268 \text{ m}^2$. Moreover, average explosion-induced damage to the buildings is characterized by a mean of $n_{s,col} = 2.4$ collapsed storeys and $n_{s,hu} = 3.8$ collapsed housing units. While these mean values might seem surprisingly high, the corresponding large coefficients of variation (CoV), between 72 and 147%, are indicative of a huge associated scatter. This applies as well to the occupancy of the area affected by the collapse, Ocu_{col} and the number of fatalities N_{col} . In this case, CoV's of around 160-170% denote a low concentration of the data around the arithmetic means of, respectively, $Ocu_{col} = 11$ persons and $N_{col} = 3$ fatalities.

7.4 Data complementation

7.4.1 Introduction

The previous sub-section shows that information on the fundamental damage parameter A_{col} is lacking in almost 80% of the potentially relevant collapse incidents. However, in many of those cases the extension of the collapse could be quantified by means of secondary damage parameters, i.e., the number of collapsed housing-units $n_{hu,col}$ or storeys, $n_{s,col}$, available in, respectively, 23 and 76% of the 107 incidents (Table 7.1). In the present section, information provided by the National Statistics Institute (INE) [112] will be incorporated into the database that allows for an estimation of A_{col} based on those secondary

damage parameters (section 7.4.2). Moreover, INE provides data for estimation of the occupancy of this area (O_{col}) in case it could not be previously deduced from the original data sources (section 7.4.3).

The information adopted from INE stems from the housing- and population census carried out in 2001. It is supposed to provide good average estimates for the evaluation period covered in the present survey (Figure 7.5). Although in a strict sense the mentioned census data applies to the Spanish territory only, it is considered representative for the other countries included in the study as well (Figure 7.4).

7.4.2 Area affected by the collapse

Table H.2 [112] in Annex H.3 provides information on the number of housing units n_{hu} in Spain as a function of their net-room area A_{hu} , distinguished into 10 different categories, and the number of storeys n_s corresponding to the building to which these units belong. Based on this information, the average net-room area $A_{hu,av}$ of a Spanish housing unit can be established as a function of n_s , as summarized in Table 7.2 below.

Table H.3 [112] in Annex H.3 presents the number of buildings in Spain comprising a certain number of housing units and storeys. This data allows for establishment of the average number of housing units $n_{hu,av}$ per building comprising a specific number of storeys n_s . The subsequent multiplication of $A_{hu,av}$ and $n_{hu,av}$ delivers the average net room area A_{av} of a Spanish residential building as a function of n_s . In order to take account of areas attributable to collective building use, not implied in the housing unit net room areas, A_{av} is enhanced by 10% after performing the mentioned multiplication. Finally, A_{av} can be divided by n_s in order to obtain the average area per storey, $A_{s,av}$. The deduced areas $A_{s,av}$ and A_{av} are summarized in Table 7.2 as a function of n_s . The last column of this table represents the weighted average for the net room area of a Spanish residential housing unit ($A_{hu,av}=90 \text{ m}^2$), building storey ($A_{s,av}=122 \text{ m}^2$) and building ($A_{hu,av}=249 \text{ m}^2$).

Table 7.2: Average net-room area per housing unit ($A_{hu,av}$), per building storey ($A_{s,av}$) and per building (A_{av}) as a function of the number of building storeys n_s

n_s	1	2	3	4	5	6	7	8	9	≥ 10	Weighted Average
$A_{hu,av}$ [m ² /hu]	97	103	95	82	78	80	85	87	88	87	90
$A_{s,av}$ [m ² /s]	116	70	90	169	213	247	271	297	266	240	122
A_{av} [m ² /b]	116	139	269	680	1064	1483	1896	2378	2401	3611	249

The data given in Table 7.2 is subsequently employed to estimate the net room areas A corresponding to the buildings of the 107 potentially relevant incidents (section 7.3.3), in case this data is lacking. Therefore, equations (7.5) to (7.7) are applied depending on the knowledge of the number of housing units (n_{hu}) and storeys (n_s).

$$A = n_{hu} \cdot A_{hu,av} \quad \text{if } n_{hu} \text{ and } n_s \text{ known} \quad (7.5)$$

$$A = n_{hu} \cdot 90 \quad \text{if } n_{hu} \text{ known and } n_s \text{ unknown} \quad (7.6)$$

$$A = n_s \cdot A_{s,av} \quad \text{if } n_{hu} \text{ unknown and } n_s \text{ known} \quad (7.7)$$

Based on the estimated building net room area A , the area affected by the collapse, A_{col} , can be finally deduced from (7.3) as a function of ratio $n_{hu,col}/n_{hu}$ or $n_{s,col}/n_s$. In the rare case neither n_s nor n_{hu} is available, A_{col} can still be estimated from (7.6) or (7.7), by substituting, respectively, n_{hu} for $n_{hu,col}$, or n_s for $n_{s,col}$, in the latter case in association with the weighted average net room area per storey, $A_{s,av} = 122 \text{ m}^2$ (Table 7.2).

7.4.3 Occupancy of the area affected by the collapse

Table H.4 [112] included in Annex H.3 provides useful statistical information on the building occupancy. It gives the total number of persons in Spain per category of housing unit net room area (A_{hu}). Combining this data with the information on the number of housing units within each of these categories (Table H.2), delivers the average occupancy per Spanish housing unit, summarized in Table 7.3. The last column indicates that on an average around 3 persons share a housing unit. Taking account that the Spanish average housing unit disposes of about 90 m^2 (Table 7.2), it may be concluded that the available mean net room area per building user is around 30 m^2 .

Table 7.3: Average occupancy $Ocu_{hu,av}$ per housing unit as a function of the housing unit net room area A_{hu}

$A_{hu} [\text{m}^2]$	≤ 30	>31 ≤ 45	>46 ≤ 60	>61 ≤ 75	>76 ≤ 90	>91 ≤ 105	>106 ≤ 120	>121 ≤ 150	>151 ≤ 180	> 180	Weighted Average
$Ocu_{hu,av}$ [persons/hu]	1.9	2.1	2.4	2.7	2.9	3.0	3.1	3.2	3.3	3.4	2.9

Based on the data compiled in Table 7.3, a statistical estimation of the total building occupancy Ocu at the time of the incident can be realized by means of equations (7.8) and (7.9) under the following specified conditions:

$$Ocu = n_{hu} \cdot Ocu_{hu,av} \quad \text{if } n_{hu} \text{ and } A_{hu} \text{ known} \quad (7.8)$$

$$Ocu = n_{hu} \cdot 2.9 \quad \text{if } n_{hu} \text{ known and } A_{hu} \text{ unknown} \quad (7.9)$$

In case the number of housing units is not known, Ocu can be estimated from (7.10), which makes use of the previously mentioned available average building net room area per building user, of about 30 m^2 :

$$Ocu = \frac{A}{30} \quad \text{if } n_{hu} \text{ unknown (building net-room area } A \text{ in } \text{m}^2) \quad (7.10)$$

In going from equations (7.8) to (7.10), the statistical uncertainty associated with the estimation of Ocu increases. Additional uncertainty arises from the fact that these equations provide predictions of the theoretical building occupancy, i.e., based on the assumption that all persons living in a building are actually present at the time the incident occurs. This, however, will depend on a number of factors, such as the time span (morning, afternoon, or nighttime) or the day of the week (labor or week-end) during which the incidents occurs. Since no statistical data is available which would allow for a more precise estimation of Ocu under consideration of such effects, full building occupancy is assumed for sake of the data complementation.

Finally, the occupancy Ocu_{col} of the area affected by the collapse A_{col} can be estimated by means of relation (7.4) from the total building occupancy Ocu and the ratio between A_{col} and the total building net room area A . In the exceptional case A cannot be previously established, Ocu_{col} can be obtained from

(7.10) by substituting A for A_{col} . In any case, the estimations for Ocu and Ocu_{col} should comply with available information on minimum or maximum occupancies. Lower bound values can be derived, for instance, from the sum of fatalities (N) and injuries (I), both of which are normally indicated in the data sources. Moreover, in some cases, the press releases contain information on the theoretical building occupancy, what can be employed as an upper bound value to the occupancy at the time of the incident.

7.4.4 Results

The results of the statistical data complementation described in the previous sub-sections are given in Annex H.4. In Table 7.4, which provides a summary, it is shown that in comparison to the original data-sample (Table 7.1) the n^o of records has largely increased. The area affected by the collapse A_{col} and the corresponding occupancy Ocu_{col} are now available in, respectively, 91 and 95 of the 107 incidents. Important to mention is that the order of magnitude of both average values and scatter is not altered. The mean values (μ) of variables Ocu_{col} and A_{col} have experienced an increase of about 5 and 30%, respectively, while the corresponding coefficients of variation (CoV) decreased in about 20% with respect to the original data.

Table 7.4: Statistical evaluation of complemented data corresponding to 107 potentially relevant incidents

Parameter	A [m^2]	A_{col} [m^2]	Ocu	Ocu_{col}
N ^o of records	104	91	101	95
% of total	97	85	94	89
μ	650	345	21	11
σ	566	395	25	15
CoV [%]	87	114	118	134
Max	2401	2401	150	88
Min	54	15	1	1

In summary, 91 explosion-induced collapse incidents, for which the fundamental damage parameter A_{col} could be established, are available for purpose of a multiple linear regression analysis aimed at deriving a consequence model for estimation of structure-related risks to persons. This analysis is described in section 7.5 below.

7.5 Multiple linear regression analysis

7.5.1 Introduction

On the empirical basis of the compiled, evaluated and complemented 91 relevant incidents identified in the foregoing section, the objective pursued in the present section is to derive a mathematical model that, with certain accuracy, allows for an estimation of the number of fatalities N_{col} due to a specific collapse scenario. The extensions of this scenario should be described by the area affected by the collapse A_{col} , by which means the link to the mathematical framework for quantitative evaluation of risks is established (section 7.1). Moreover, it should be analyzed if, besides area A_{col} , the occupancy of this area, Ocu_{col} , has a significant statistical influence on the estimation of N_{col} . The data plots shown in Figure 7.9 (left: N_{col} - A_{col} ; right: N_{col} - Ocu_{col}) and the corresponding tendency lines seem to affirm this hypothesis.

Figure 7.9 N_{col} - A_{col} (left) and N_{col} - Ocu_{col} plots (right) including tendency lines for 91 relevant incidents

The mathematical relationship between N_{col} and the explanatory variables A_{col} and Ocu_{col} , is deduced by means of a multiple linear regression model. Section 7.5.2 describes the general characteristics of such a model and its associated statistical criteria and parameters. In the following, section 7.5.3 presents the most significant models deduced. Finally, the prediction accuracy of these models is discussed in section 7.5.4, where, in addition, a comparison to prior consequence models is presented.

7.5.2 Regression model and associated criteria

Model description

The general equation for a multiple linear regression model can be written as per (7.11) below [206].

$$Y = \alpha + \beta_1 X_1 + \beta_2 X_2 + \cdots \beta_k X_k + \epsilon \quad (7.11)$$

In general terms, this formulation expressed that the dependent variable Y can be described as a function of k explanatory variables and an error- or disturbance term ϵ that encompasses other unspecified, omitted factors. The model parameters β_i gauge the effect of the corresponding explanatory variable X_i on the dependent variable Y , holding all other explanatory variables constant [206]. In addition, a constant model parameter α (intercept) characterizes the linear regression model.

Model evaluation

The overall statistical significance of the model can be checked by means of the F-test, based on the F distribution by Snedecor-Fischer. The objective of this test is to check hypothesis H_0 which states that all of the model parameters β_i equal zero (7.12). In particular, H_0 can be rejected if the probability (p-value) that a cut-off value $F > F^*$ is less than or equal to a specified maximum probability, in the present case established as 0.05. The F-statistic F^* will be defined at a later stage (7.23).

$$H_0: \beta_1 = \beta_2 = \cdots = \beta_k = 0 \quad (7.12)$$

The model parameters in (7.11) are computed with IBM SPSS Statistics [207] using the ordinary least squares method, i.e. by minimizing the sum of squared prediction errors for the data points of the sample. A so called “backward elimination” regression procedure is applied. This procedure consists in initially including all the potential explanatory variables and in each subsequent step eliminating the least significant one, as determined by the t-test. This test, based on the t-distribution by Student, checks the hypothesis H_0 (7.13) that a specific model parameter β_i equals zero, in which case the corresponding

variable X_i would be insignificant. Therefore, the t-statistic t_{bi} , defined by the quotient between regression coefficient b_i (which represents the sample estimate of the corresponding population parameter β_i), and its standard deviation s_{bi} , is employed (7.14). In particular, H_0 can be rejected if the probability (p-value) that $t > t_{bi}$ is less than or equal to a specified maximum probability, in the present case established as 0.1. The regression procedure ends when the p-value for all the variables remaining in the model complies with this criterion.

$$H_0: \beta_i = 0 \quad (7.13)$$

$$t_{bi} = \frac{b_i}{s_{b_i}} \quad (7.14)$$

The coefficient of determination R^2 can be used to evaluate the goodness-of-fit of the regression model to the data points. It indicates the proportion of the variation in the dependent variable Y that can be explained by the model through a comparison of the model errors to the errors of a benchmark model in which the sample mean $\bar{Y}$ is used to predict Y [206]. Formally, R^2 can be expressed by (7.15), as the ratio between the explained sum of squares (ESS) and the total sum of squares (TSS). The former (7.16) quantifies the deviations of the n predicted values $\hat{Y}_i$ from the mean value $\bar{Y}$, while the latter (7.18), defined as the sum of ESS and the residual sum of squares (RSS), is indicative of the variation of the n data points Y_i around $\bar{Y}$. The residual sum of squares (7.17) is a measure for the model error, in terms of the discrepancy between data points Y_i and predicted values $\hat{Y}_i$.

$$R^2 = \frac{ESS}{TSS} = 1 - \frac{RSS}{TSS} \quad (7.15)$$

$$ESS = \sum_{i=1}^n (\hat{Y}_i - \bar{Y})^2 \quad (7.16)$$

$$RSS = \sum_{i=1}^n (Y_i - \hat{Y}_i)^2 \quad (7.17)$$

$$TSS = \sum_{i=1}^n (Y_i - \bar{Y})^2 \quad (7.18)$$

It should be noted that adding explanatory variables X_i to a regression model implies a decrease of the RSS while the TSS remains constant. Hence, according to (7.15), R^2 increases with each X_i added to the model. Consequently, R^2 does not provide an objective basis for the evaluation and comparison of the goodness-of-fit of models with different numbers of explanatory variables X_i . The adjusted coefficient of determination $\bar{R}^2$ (7.19), which takes into account the number of degrees of freedom, can be used in lieu of the standard R^2 to correct for the over-parameterisation of the latter [206, 208].

$$\bar{R}^2 = 1 - \frac{(n-1)}{n-(k+1)} (1 - R^2) \quad (7.19)$$

Another parameter that provides an overall measure of how well the regression line fits the data sample is the standard error of the estimate (SEE). It represents the average distance between the observed values Y and the regression line and is hence an estimate for the model's errors standard deviation. Formally, it is defined by the square root of the mean square error (MSE), as expressed per (7.20). The MSE (7.21) is a function of the residual sum of squares (7.17) and its corresponding number of degrees of freedom, $n-(k+1)$. Similarly, the regression mean square (MSR) is defined by the quotient between the explained sum of squares (7.16) and the corresponding number of degrees of freedom, k (7.22). The ratio between MSR and MSE delivers the previously introduced F-statistic F^* (7.23), required for testing hypothesis (7.12) in order to check the global significance of the regression model.

$$SEE = \sqrt{MSE} \quad (7.20)$$

$$MSE = \frac{RSS}{n-(k+1)} \quad (7.21)$$

$$MSR = \frac{ESS}{(k+1)-1} = \frac{ESS}{k} \quad (7.22)$$

$$F^* = \frac{MSR}{MSE} \quad (7.23)$$

Model requirements

At least the following requirements are to be verified for a multiple linear regression model:

- Linearity between dependent and explanatory variables
- Normality of model's errors
- Homoscedasticity of model's errors
- Independence of model's errors

Linearity of the model is provided with previous transformation of the variables, while normality of the model's errors can be checked by means of Q-Q plots. The analysis of studentized residuals t_i is used to confirm a constant variance of the model's errors, known as homoscedasticity, and their independency. Moreover, the analysis of t_i allows for identification of unusual observations (outliers). In the present study, outliers were identified as values with t_i outside the range ± 3.0 .

Studentized residuals t_i (7.24) are obtained by dividing the ordinary residuals e_i (7.25) by an estimate of their standard deviation ($\sqrt{MSE}$) and the so-called leverage h_{ii} , which quantifies the influence that the observed response Y_i has on the predicted value $\hat{Y}_i$. The leverage h_{ii} of an observation Y_i corresponds to the i^{th} diagonal element of the hat matrix $\mathbf{H}$, which relates the vector of the observations Y_i to the vector of model predictions $\hat{Y}_i$ (7.26). It should be noted that the subscript (i) added to the mean square error (MSE) in (7.24) denotes that externally studentized residuals (also called deleted residuals) are referred to here, which use the mean square error based on the estimated model with the i^{th} observation deleted, in case this observation constitutes a potential outlier. In contrast, internally studentized residuals are based on the MSE corresponding to the regression model including all observations Y_i .

$$t_i = \frac{e_i}{\sqrt{MSE_{(i)}(1-h_{ii})}} \quad (7.24)$$

$$e_i = Y_i - \hat{Y}_i \quad (7.25)$$

$$h_{ii} = [\mathbf{H}]_{ii} \quad (7.26)$$

7.5.3 Results

A large number of trials was carried out under variation of different combinations and mathematical transformations of the variables N_{col} , A_{col} and Ocu_{col} to adjust a multiple linear regression model to the $n = 91$ data points. A common finding in these trials was the impossibility to verify the homoscedasticity and independency of the residuals. This problem could be attributed to the relatively large number of collapse incidents with zero fatalities, which account for 33 observations within the sample (Figure 7.9). This problem is illustrated in Figure 7.10, which shows the externally studentized residuals t_i plotted

versus the predicted values $\hat{Y}_i$ of the dependent variable Y , in this particular case $Y = \ln(N_{col}/(Ocu_{col}/A_{col})+1)$. It can be observed that the t_i are not homogenously distributed around the abscissa. The t_i corresponding to the zero fatality incidents, highlighted in the figure, depart from the abscissa in a systematic manner denoting that the regression model is inappropriate to explain the variability of the dependent variable. Only when the 33 data points were removed from the sample, this problem could be satisfactorily solved. A description of the two most significant models, adjusted to the remaining 58 observations, is given below.

Figure 7.10 Externally studentized residuals t_i versus predicted values $\hat{Y}_i$ for $\ln(N_{col}/(Ocu_{col}/A_{col}))$

Model 1

Regression model 1 is given in equation (7.27) below. All variables were log-transformed to stabilize the variance of the regression model error. The response variable of model 1 corresponds to the number of fatalities N_{col} divided by the occupancy ratio of the area affected by the collapse, Ocu_{col}/A_{col} . The backward elimination procedure (section 7.5.2) concluded the explanatory variables Ocu_{col}/A_{col} and A_{col} to be significant. The almost zero p-values associated with the corresponding regression coefficients b_1 and b_2 (Table 7.5) indicate strong evidence to reject hypothesis (7.13).

$$\ln\left(\frac{N_{col}}{Ocu_{col}/A_{col}}\right) = \alpha + \beta_1 \ln(Ocu_{col}/A_{col}) + \beta_2 \ln(A_{col}) + \epsilon \quad (7.27)$$

Table 7.5: Estimation of regression coefficients of model 1

Variable	Non standardized coefficients			t	p-value
	Designation	Estimate	s		
Intercept	a	-0.329	0.583	-0.564	0.575
$\ln(Ocu_{col}/A_{col})$	b_1	-0.485	0.110	-4.406	$\cong 0.000$
$\ln(A_{col})$	b_2	0.566	0.087	6.536	$\cong 0.000$

The ANOVA (analysis of variance) findings for model 1 are listed in Table 7.6. The value obtained for test statistic F^* (7.23) is 36.12, while the cut-off value associated with the adopted maximum probability of 0.05 (section 7.5.2), $F_{k;n-(k+1);0.05}$ is 3.17. Given that F^* is much larger than $F_{k;n-(k+1);0.05}$, equation (7.27)

was concluded to provide a statistically significant explanation for the differences in the dependent variable $\text{Ln}(N_{\text{col}}/(\text{Ocu}_{\text{col}}/A_{\text{col}}))$.

Table 7.6: ANOVA (analysis of variance) of model 1

Source of variance	Sum of squares	Degrees of freedom	Mean square	F*	p-value
Regresión	ESS = 32.51	2	MSR = 16.28	36.21	$\cong 0.000$
Residual	RSS = 24.72	55	MSE = 0.45		
Total	TSS = 57.27	57			

Based on the ANOVA results given in Table 7.6, the adjusted coefficient of determination ($\bar{R}^2$) is determined by (7.19) and summarized in Table 7.7. The obtained value of 0.553 indicates that the considered explanatory variables explain 55.3 % of the variance of the dependent variable $\text{Ln}(N_{\text{col}}/(\text{Ocu}_{\text{col}}/A_{\text{col}}))$, what is indicative of a reasonable model fit to the 58 data points.

Table 7.7: Goodness of fit indicators for model 1

Model	n	R	R^2	$\bar{R}^2$	SSE
1	58	0.754	0.568	0.553	0.677

Normality was checked with a normality plot of the studentized residuals t_i (Q-Q plot), shown in Figure 7.11 (left). The straight pattern along the bisecting line ($y = x$) provides evidence that it is reasonable to assume normally distributed residuals. This hypothesis is corroborated by a comparison of the coefficient of correlation r_Q associated with the quantile plot and the critical quantiles $r_{Q,\alpha}$. For a sample size of $n \approx 60$ and a statistical significance of 10%, the critical value is $r_{Q,\alpha=0.1} = 0.984$ [208]. Since $r_Q = 0.991 > r_{Q,\alpha}$, the hypothesis of normally distributed residuals cannot be rejected.

The diagram relating externally studentized residuals t_i to predicted values $\hat{Y}_i$ is shown in Figure 7.11 (right). In comparison to Figure 7.10, the t_i are homogeneously distributed around $t_i = 0$. No systematic patterns are detected nor does the variance of the t_i seem to experience any significant changes. Hence, Figure 7.11 (right) confirms homoscedasticity and independence of the model's errors. Moreover, the Figure shows that no potential outliers have to be analyzed since $t_i < 3$ for all 58 observations (section 7.5.2).

Figure 7.11 Q-Q plot (left) and studentized residuals versus predicted value (right) of model 1

In summary, model 1 complies with the general requirements associated with multiple linear regression models (section 7.5.2). After resubstituting the log-transformed variables in (7.27), regression model 1, which allows for an estimation of the fatalities due to a collapse scenario characterized by an area A_{col} and an occupancy Ocu_{col} , is given by expression (7.28):

$$N_{col} = 0.72 \cdot Ocu_{col}^{0.515} \cdot A_{col}^{0.051} \quad (7.28)$$

Model 2

Regression model 2 is defined by equation (7.29). Again, all variables were log-transformed. The response variable is the ratio between the number of fatalities N_{col} and the occupancy of the area affected by the collapse, Ocu_{col} . Term “+1” is introduced in order to avoid negative values of the logarithm. Since $\ln(N_{col}/Ocu_{col}) \approx \ln(N_{col}/Ocu_{col}+1)$, adding this term does not introduce a significant distortion of the results.

The explanatory variables are the same as in model 1 (7.27). Variables Ocu_{col}/A_{col} and A_{col} were found to have a significant influence as denoted by the almost zero p-values associated with the corresponding regression coefficients b_1 and b_2 (Table 7.8). Evidence for the global statistical significance of the model is provided by the F-statistic, $F^* = 22.61$ (Table 7.9), which significantly exceeds the cut-off value $F_{k;n-(k+1);0.05} = 3.17$.

$$\ln\left(\frac{N_{col}}{Ocu_{col}} + 1\right) = \alpha + \beta_1 \ln(Ocu_{col}/A_{col}) + \beta_2 \ln(A_{col}) + \epsilon \quad (7.29)$$

Table 7.8: Estimation of regression coefficients of model 1

Variable	Non standardized coefficients			t	p-value
	Designation	Estimate	s		
Intercept	a	0.539	0.140	3.861	$\cong 0.000$
$\ln(Ocu_{col}/A_{col})$	b_1	-0.127	0.026	-4.820	$\cong 0.000$
$\ln(A_{col})$	b_2	-0.111	0.021	-5.361	$\cong 0.000$

Table 7.9: ANOVA (analysis of variance) of model 2

Source of variance	Sum of squares	Degrees of freedom	Mean square	F*	p-value
Regresión	ESS = 1.162	2	MSR = 0.584	22.61	$\cong 0.000$
Residual	RSS = 1.422	55	MSE = 0.260		
Total	TSS = 2.591	57			

The adjusted coefficient of determination ($\bar{R}^2$) of 0.431 (Table 7.10) denotes that the model explains 43.1 % of the variance of the dependent variable $\ln(N_{\text{col}}/(\text{Ocu}_{\text{col}}/A_{\text{col}}))$. It should be noted that the $\bar{R}^2$ corresponding to models 1 and 2 are not comparable since the response variables in both models are not the same. A comparison of the accuracy of both models is subject of section 7.5.4.

Table 7.10: Goodness of fit indicators for model 2

Model	n	R	R^2	$\bar{R}^2$	SSE
2	58	0.672	0.451	0.431	0.162

The Q-Q plot for model 2 is shown in Figure 7.12 (left). The straight pattern along the bisecting line provides evidence that it is reasonable to assume normally distributed residuals. The associated coefficient of correlation r_Q equals the critical value at a significance level $\alpha = 0.1$, $r_{Q,\alpha=0.1} = 0.984$.

Figure 7.12 Q-Q plot (left) and studentized residuals versus predicted value (right) of model 2

The diagram relating externally studentized residuals t_i to predicted values is shown in Figure 7.12 (right). It is shown that the t_i are homogenously distributed around $t_i=0$ confirming homoscedasticity and independence of the model's errors. Neither in this case any potential outliers have to be analyzed since $t_i < 3$ for all 58 observations.

In summary, model 2 likewise complies with the general requirements associated with multiple linear regression models. After resubstituting the log-transformed variables in (7.29), regression model 2, which allows for an estimation of the fatalities due to a collapse scenario characterized by an area A_{col} and an occupancy Ocu_{col} , is defined as per expression (7.30):

$$N_{\text{col}} = \text{Ocu}_{\text{col}}(1.7 \cdot \text{Ocu}_{\text{col}}^{-0.127} \cdot A_{\text{col}}^{0.016} - 1) \quad (7.30)$$

7.5.4 Discussion

Model accuracy

Several statistical criteria are in place for establishing regression model accuracy. A commonly employed measure is the M ratio between the observation (Y_i) and the model estimate ($\hat{Y}_i$). In the present context, this ratio is defined in terms of the observed number of fatalities induced by a specific collapse scenario ($N_{col,i}$) and the corresponding predicted value ($\hat{N}_{col,i}$):

$$m_i = \frac{N_{col,i}}{\hat{N}_{col,i}} \quad (7.31)$$

The statistical description of the M ratio as obtained from a comparison of the model predictions $\hat{N}_{col,i}$, by means of model 1 (7.28) and 2 (7.30), to the 58 observations $N_{col,i}$ used to derive these models is given in Table 7.11. Mean values μ_M larger than unity indicate that, on average, both models tend to underestimate the expected number of fatalities in a collapse event, N_{col} . Judging from the mean μ_M , model 2 yields comparatively more accurate predictions. However, coefficients of variation CoV_M close to 70% for both models are indicative of a relatively large associated scatter.

Table 7.11: Statistical evaluation of M ratio (mean μ_M , standard deviation s_M and coefficient of variation CoV_M), predictive residual mean (PRM) and cut-off point corresponding to models 1 and 2

Model	μ_M	s_M	CoV_M	PRM	Cut-off point
1 (7.26)	1.24	0.85	0.69	1.12	1.15
2 (7.28)	1.07	0.71	0.66	0.51	2.89

The predictive residual mean (PRM) is an estimate of the systematic error in model predictions. Defined by (7.32) as the difference between the mean value of the observed number of fatalities, $\bar{N}_{col,i}$, and the mean of the predicted values, $\bar{\hat{N}}_{col,i}$, it is related to the centroid of the observed-predicted scatter plot ($\bar{N}_{col,i}$, $\bar{\hat{N}}_{col,i}$), shown in Figure 7.13.

$$PRM = \bar{N}_{col,i} - \bar{\hat{N}}_{col,i} \quad (7.32)$$

The positive values obtained for the PRM (Table 7.11) confirm that, on average, the predictions delivered by both models underestimate the number of fatalities. This is illustrated in Figure 7.13, where the centroid of the scatterplot is located above the bisecting line ($N_{col,i} = \hat{N}_{col,i}$). The fact that the PRM associated with model 2 is smaller indicates a comparatively smaller degree of under-prediction.

The intersection between the bisecting- and the orthogonal regression line (minimizes the sum of squares of the Euclidean distances to the regression line), delivers the so-called cut-off point, shown in Figure 7.13. It divides the observed-predicted values graph in two areas: on the left, the over-prediction area and on the right, the under-prediction area. The cut-off points are 1.15 fatalities in model 1 and 2.89 in model 2 (Table 7.11). In other words, fatalities upward of those values predicted with model 1 or 2, respectively, will be, on average, below the expected n^o of fatalities in a structural collapse event.

Figure 7.13 Observed versus predicted number of fatalities: Model 1 (left) and model 2 (right)

Application boundaries

Figure 7.14 offers a graphical representation of equations (7.28) and (7.30) corresponding to, respectively, model 1 (left) and 2 (right). Precisely speaking, the figure shows the predicted number of fatalities in a collapse scenario (N_{col}) as a function of the area affected by the collapse (A_{col}) and the occupancy ratio of this area, Ocu_{col}/A_{col} . The area A_{col} , shown on the abscissae, is represented from $A_{col} = 15$ to 2400 m^2 , corresponding to, respectively, the minimum and maximum areas among the sample data to which the models were adjusted (see Figure 7.9 (left)). The occupancy ratio, i.e., the number of persons Ocu_{col} per square meter of area A_{col} , at the time of collapse, is varied between $1/96$ and $1/3$. The latter value corresponds to the maximum occupancy ratio among the sample data. The minimum Ocu_{col}/A_{col} ratio among these data is $1/270$.

The transition between the continuous lines and the dashed lines shown in Figure 7.14 corresponds to the maximum number of persons present on the area affected by the collapse among the sample data, $Ocu_{col,max} = 88$ persons (see Figure 7.9 (right)). For instance, assuming an occupancy ratio of $Ocu_{col}/A_{col} = 3$ (orange curves), the required area to accommodate $Ocu_{col,max} = 88$ persons is $A_{col} = 264 \text{ m}^2$. Beyond this limit value, the model predicts outside of its application boundaries. Irrespective of Ocu_{col} , the previously mentioned minimum and maximum areas of, respectively, $A_{col,min} = 15 \text{ m}^2$ and $A_{col,max} = 2400 \text{ m}^2$, likewise constitute boundaries to the scope of application of the models, outside of which predictions should be generally analyzed under extreme caution. The same applies to the minimum and maximum occupancy ratios $(Ocu_{col}/A_{col})_{min} = 1/270$ and $(Ocu_{col}/A_{col})_{max} = 1/3$.

Figure 7.14 Graphical representations of model 1 (left) and 2 (right)

Figure 7.14 (left) shows that model 1 reasonably predicts a higher number of fatalities N_{col} with increasing A_{col} . Moreover, for a certain A_{col} , N_{col} rises with larger occupancy ratios Ocu_{col}/A_{col} , what is likewise a logical finding. The larger A_{col} , the stronger is the influence of ratio Ocu_{col}/A_{col} .

Model 2 (Figure 7.14, right) likewise estimates a higher N_{col} with increasing A_{col} . However, only up to certain point, beyond which the predicted N_{col} significantly decrease. Interestingly, this point is located close to the before-mentioned application boundary, represented by the transition between continuous and dashed lines. This suggests, that upon approaching this boundary, the uncertainties associated with the model predictions largely increase, what is supported by the observation that incidents with large occupancies Ocu_{col} are generally scarce among the sample data used to derived the model (Figure 7.9, right). It is concluded that model 2 delivers a priori reasonable predictions only up to a certain area A_{col} , depending on the occupancy ratio. For instance, assuming $Ocu_{col}/A_{col} = 1/30 \text{ m}^{-2}$, which represents a mean value for residential buildings (section 7.4.3), model 2 should not be used for N_{col} predictions on areas A_{col} larger than approximately 2000 m^2 .

Comparison to prior models

In previous studies [9, 10, 13], consequence models were derived to predict the number of fatalities N_{col} due to structural collapse events in buildings associated with consequence classes CC2 and CC3 according to [11]. The approach for deriving these models was generally the same as followed herein, with two mayor differences however. The first is that, in contrast to the present study, the majority of the compiled incidents occurred under normal building-use circumstances, i.e. without the influence of a collapse-triggering accidental action. The second mayor difference to the present approach is that N_{col} is predicted in terms of a single explanatory variable, the area affected by the collapse, A_{col} . The regression models derived in [9, 10, 13] are given by equations (7.33) and (7.34).

$$N_{col} = 0.27A_{col}^{0.5} - 1 \quad \text{for CC2 buildings} \quad (7.33)$$

$$N_{col} = 0.59A_{col}^{0.56} - 1 \quad \text{for CC3 buildings} \quad (7.34)$$

The m_i ratios (7.31) for the adjustment of models (7.33) and (7.34) to the 58 collapse incidents comprising data sample employed for deriving models 1 (7.28) and 2 (7.30) are shown in Table 7.12. As was observed before for the herein derived models, the CC2 model (7.33) tends to underpredict the number of fatalities in a structural collapse scenario. The mean M-ratio, $\mu_M = 1.54$, is larger than the corresponding values found for models 1 ($\mu_M = 1.24$) and 2 ($\mu_M = 1.07$) what indicates a comparatively stronger degree of underprediction. Moreover, the coefficient of variation CoV_M associated with model (7.33), $CoV_M = 1.90$ (Table 7.12) is significantly larger than the values close to 0.7 (Table 7.11) found for models 1 (7.28) and 2 (7.30) what denotes a comparatively larger scatter inherent to the model fit.

The comparatively better adjustment of the herein derived models to the data sample is partly attributable to the incorporation of the number of persons present on the area affected by the collapse, Ocu_{col} , which, as shown in section 7.5.3, turned out to be a statistically significant influence variable for explanation of the variability in N_{col} . In other words, the inclusion of variable Ocu_{col} in addition to A_{col} enhances the model's prediction capacity.

Related to variable Ocu_{col} is the physical nature of, respectively, an explosion-induced collapse scenario and a collapse under normal building use conditions. An explosion is likely to induce a sudden and unannounced collapse event that does not allow persons evacuating the building. Hence, the number of persons present on the area affected by the collapse (Ocu_{col}) in such events is possibly higher compared to non-accidental (referring to the leading action or influence) collapse scenarios, which due to their

eventually less abrupt nature, provided that the failure mechanism is ductile, might enable building users to leave the building before collapse occurs. Keeping in mind that CC2 model (7.33), which does not consider Ocu_{col} in an explicit manner, was deduced from data on mainly non-accidental collapse scenarios, hence contributes to explain the comparatively worse fit to the data sample constituted by explosion-induced collapse scenarios. The fact that, on average, model (7.33) underestimates the number of fatalities in a more pronounced way than the herein derived models (7.28) and (7.30) corroborates this theory.

Table 7.12: Statistical evaluation of M ratio (mean μ_M , standard deviation s_M and coefficient of variation CoV_M), predictive residual mean (PRM) and cut-off point corresponding to models derived in [9, 13]

Model	μ_M	s_M	CoV_M	PRM	Cut-off point
CC2 (7.33)	1.54	2.92	1.90	0.51	≈ 0
CC3 (7.34)	0.31	0.26	0.84	-10,6	≈ 0

The CC3 model given by (7.34) is based on collapse incidents where persons congregated on the area affected by the collapse [9, 10, 13], characterized by large occupancy ratios Ocu_{col}/A_{col} . Fitting this model to the 58 explosion-induced collapse events in CC2 buildings (residential or office use), with comparatively lower Ocu_{col}/A_{col} ratios, furnishes unsurprising results. Table 7.12 shows that model (7.34) clearly overpredicts the number of fatalities in CC2 buildings. This is manifested by a μ_M ratio far below unity and a negative predictive residual mean (PRM). The observed versus predicted N_{col} -plot and the corresponding orthogonal regression, which falls significantly below the bisecting line, confirms the overpredictive character of model (7.34) in the context of a consequence analysis in CC2 buildings.

Finally, Figure 7.15 shows a very good qualitative concurrence between the previous consequence models (7.33) and (7.34) for, respectively, CC2 and CC3 buildings, and the herein derived model 1 (7.28). Assuming for the latter an occupancy ratio of Ocu_{col}/A_{col} of about $1/24 \text{ m}^{-2}$, entails a practically coincident result to the CC2 model (7.33). It should be observed that this value is close to the mean occupancy ratio in Spanish residential buildings, of about $1/30 \text{ m}^{-2}$ (section 7.4.3). On the contrary, the graphical representations of the CC3 model (7.34) and model 1 (7.28) roughly match if ratio Ocu_{col}/A_{col} is assumed to be approximately $1/1.8 \text{ m}^{-2}$. Keeping in mind that in typical buildings classified under CC3, such as assembly halls, theatres, cinemas or grandstands, the available area per building user is estimated to range from 0.5 to 2 m^2 [13], this is likewise a reasonable finding. In summary, these results suggest that different consequence classes can be adequately accounted for by an explicit consideration of the building occupancy ratio in the consequence model. Future studies should envisage deriving a unique model from a joint database comprising collapse events due to all kind of influences in and on buildings associated with all types of consequence classes.

Figure 7.15 Comparison of model 1 (7.28) to previous models [9, 10, 13] derived for collapse incidents in buildings associated with consequence class CC2 (7.33) and CC3 (7.34)

7.6 Summary

Chapter 7 addresses the development of a consequence model which provides a prediction of the number of fatalities N_j associated with a specific structural collapse scenario j . A basic requirement to such a model is that it can be properly linked to the defined mathematical framework for risk quantification (section 4.2.3), what is briefly addressed in section 7.1. Previous work [9, 10, 13] showed that this link can be suitably established by damage parameter A_{col} , which represents the building area affected by the collapse. Based on the adopted hypothesis for description of a system collapse (section 4.5.4), the area affected by the collapse of a specific structural member, $A_{col,j}$, is defined in terms of the parameters describing the global geometry of the structure (Figure 4.4) and the specific member location within this structure.

In order to establish a relationship between N_j and $A_{col,j}$, data on explosion-induced structural collapse events in Spain and other western countries was gathered in the framework of a systematic press-survey (section 7.2). A database was established to classify the information depending on whether it refers to the characteristics of the affected building, of the incident or the incident consequences. Three categories are established in order to distinguish heavily damaged buildings involving structural collapse scenarios from light- or medium damage to buildings (section 7.2.3, Figure 7.2). This distinction is not always straight forward, however. Especially less “attractive” incidents involving collapse of far smaller extensions, and only few or no fatalities, are rather scarcely addressed in the media, lacking detailed damage descriptions in many cases. Moreover, such descriptions are generally subjective, subject to the perception of damage by the informing journalist. Hence, in order to reliably identify a structural collapse scenario and to perform a trustworthy, quantitative estimate of its extensions, the available information had to be cautiously analyzed and evaluated, requiring in many cases a tedious, comparative study of several data sources, including local media if appropriate.

The systematic data collection concluded with a total of 233 recorded explosion incidents. In section 7.3, descriptive statistics are used in first place to provide an overview on the compiled data (section 7.3.2). In the following, 107 incidents that are potentially relevant to the specific purpose of deriving a consequence model are screened out of the data sample according to previously established selection criteria (section 7.3.3). A detailed evaluation of selected parameters characterizing these incidents (Table 7.1)

shows that information on the fundamental damage parameter A_{col} is lacking in almost 80% of the incidents, what would suppose a severe limitation to the objective persuaded. However, in many of those cases the extension of the collapse can be quantified by means of secondary damage parameters, i.e., the number of collapsed housing units ($n_{hu,col}$) or storeys ($n_{s,col}$).

Following this observation, in section 7.4, information provided by the National Statistics Institute (INE) is incorporated into the database, which allows for an estimation of A_{col} based on the mentioned secondary damage parameters (section 7.4.2). Moreover, INE provides data for estimation of the occupancy of this area (Ocu_{col}), which could be deduced from the original data sources in about 54% of the 107 potentially relevant incidents (section 7.4.3). The results of the statistical data complementation are summarized in section 7.4.4 (Table 7.4). It is shown that in comparison to the original data sample (Table 7.1) the n^o of records has largely increased. In summary, 91 relevant collapse incidents are available for purpose of the multiple linear regression analysis aimed at deriving a consequence model for estimation of structure-related risks to persons.

This regression analysis is subject of section 7.5. Orienting scatter plots suggest that, in addition to the area affected by the collapse A_{col} , the occupancy Ocu_{col} has a significant statistical influence on the estimation of the number of fatalities associated with the collapse, N_{col} (section 7.5.1, Figure 7.9). A large number of trials was carried out under variation of different combinations and mathematical transformations of the variables N_{col} , A_{col} and Ocu_{col} to adjust a regression model to the 91 data points. A common finding in these trials was the impossibility to verify the homoscedasticity and independency of the residuals, a problem attributed to the relatively large number of collapse incidents with zero fatalities (Figure 7.10). Only when the corresponding data points were removed from the sample, this problem could be satisfactorily solved.

A description of the two most significant regression models fitted to the remaining 58 data points (section 7.5.3), which comply with all previously established statistical requirements and criteria (section 7.5.2), is followed by a discussion of the corresponding model accuracy (section 7.5.4). The commonly employed M-ratio between the observation $N_{col,i}$ and the model estimate $\hat{N}_{col,i}$ is evaluated for this purpose (Table 7.11). Mean values μ_M larger than unity indicate that, on average, both models tend to underestimate the expected number of fatalities in a collapse event. Coefficients of variation CoV_M close to 70% are indicative of a relatively large associated scatter. Judging from a comparison of mean μ_M , model 2 (7.30) yields more accurate predictions than model 1 (7.28). This is corroborated by the predictive residual mean (PRM), which is an estimate of the systematic error in model predictions. The cut-off points, beyond which the model predictions will be, on average, below the expected n^o of fatalities in a structural collapse event, correspond to about 1 fatality in model 1 and about 3 fatalities in model 2 (Figure 7.13).

From Figure 7.14 it can be drawn that model 1 and 2 deliver similar predictions for N_{col} up to a certain limit value for A_{col} . However, upon approaching the boundaries of the data sample to which the model was adjusted, the predictions of both models largely diverge. While model 1 reasonably predicts an increasing N_{col} with increasing A_{col} , model 2 predictions close to and beyond these boundaries are unreasonable, what is attributable to large associated statistical uncertainties. It is concluded that model 2 delivers a priori reasonable predictions only up to a certain area A_{col} , depending on the occupancy ratio.

In previous studies [9, 10, 13], consequence models were derived to predict the number of fatalities due to structural collapse events in buildings associated with consequence classes CC2 and CC3 according to [11]. The models were obtained from data on collapse scenarios that mainly occurred under normal

building use circumstances, i.e. without the influence of a collapse-triggering accidental action. An adjustment of the CC2 model to the data sample constituted by the 58 explosion-induced collapse scenarios reveals a lower accuracy compared to the herein derived model 1 (7.28), what seems to be partly attributable to parameter Ocu_{col} included in the latter, which turned out to be statistically significant for the explanation of the variability in N_{col} (section 7.5.3). A comparison of the CC2 model predictions to those of model 1 (7.28) reveals almost identical results if an occupancy ratio $Ocu_{col}/A_{col} \approx 1/24 \text{ m}^{-2}$ is assumed for the latter, whereas the CC3 model roughly matches the model 1 results for $Ocu_{col}/A_{col} \approx 1/1.8 \text{ m}^{-2}$, what are reasonable findings (Figure 7.15). The results suggest that possible future studies should envisage a unique model, derived from a joint database comprising collapse events due to all kind of influences in and on buildings associated with all types of consequence classes.

Chapter 8 Risk-based acceptance criteria

In the present study, a hazard scenario is defined as a particular collapse event triggered by failure of one of the principle loadbearing members constituting a particular building structure (section 4.2). The corresponding occurrence frequencies p_j , are represented by the implicitly acceptable member failure probabilities $p_{f,adm}$ (Chapter 6). The associated failure consequences N_j are expressed in terms of the number of fatalities, which are estimated by means of the consequence model previously derived (Chapter 7). As a function of both p_j and N_j , the present chapter addresses the determination of the *implicitly acceptable risks* associated with each of the representative building structures defined under section 4.4. Based on the obtained findings, *acceptance criteria* for structure-related life safety risks are deduced. These tasks correspond to steps 7 and 8 of the established analysis procedure (Figure 8.1).

After a summary of the main assumptions adopted for the establishment of p_j and N_j (section 8.1), the obtained risk profiles (Figure 4.2) are presented in section 8.2. The integration of these risk profiles furnishes the implicitly acceptable risks to persons associated with a particular building structure, for which, in addition to the normal building use conditions, gas explosions are a relevant hazard to structural safety. A comparison to previous studies, dedicated to risks associated with structures where accidental scenarios were excluded from the analysis, is provided. Finally, based on the results described in section 8.2, section 8.3 addresses the development of acceptance criteria.

Figure 8.1 Procedural flow-chart

8.1 Hazard scenarios

8.1.1 Introduction

In section 4.2.3, a hazard scenario was defined as a particular collapse event triggered by failure of one of the principle loadbearing members constituting a particular building structure. Such a failure is char-

acterized by a specific failure mode (k) to arise under a particular load arrangement (l). The corresponding occurrence probability p_j , i.e. the probability for the member to fail in mode k under the influence of load arrangement l , is represented by the implicitly acceptable member failure probability, $p_{f,adm}$. The associated consequences are expressed in terms of the number of fatalities N_j due to the collapse. Sections 8.1.2 and 8.1.3 summarize the main assumptions adopted for the quantification of occurrence probabilities p_j and consequences N_j .

8.1.2 Occurrence probabilities

The implicitly acceptable structural failure probabilities for members potentially exposed to explosion-induced load arrangements were presented in section 6.2, in terms of the implicitly required reliability index $\beta_{req,EX}$. Thereby, account was taken of the occurrence probability $p(EX)$ of a gas explosion event in a housing unit supplied with gas (6.3). In the context of a risk analysis on a building level, it must be further considered that not all constitutive members will be potentially exposed to the effects of such an event for they might be located in a part of the building that is out of reach of these effects. In order to account for this circumstance, the member exposure probability p_e is introduced. Therefore, the following hypothesis are established:

- Each housing unit in a building is equipped with one gas connection
- The area of one housing unit, A_{hu} , is a function of the beam span l (Figure 4.4)
- The tributary area for a structural member under explosion-exposure, A_{EX} , is a function of the distance between frames in the longitudinal sense of the building, l (section 4.5.2)

Based on these assumptions, the exposure probability p_e of a particular structural member is estimated as a function of ratio A_{hu}/A_{EX} . The obtained results are summarized in Table 8.1.

Table 8.1 Member exposure probability p_e as a function of beam span l

l [m]	A_{hu}/A_{EX}	p_e
5	6.0	0.30
10	3.0	0.50
15	2.0	0.67
20	1.5	0.80
30	1.0	1.00
40	1.0	1.00

In section 6.3 it was concluded that the risk analysis of building structures in the context of the present study should account for the fact that in addition to the accidental hazard scenarios defined in section 4.5, certain member failure modes might be triggered by persistent load arrangements associated with normal building use conditions. In particular, it was shown that this is especially relevant when such conditions also govern the member design, since in such situations the associated failure probabilities $p_{f,adm,PER}$ may contribute significantly to the accumulated member failure probability $P_{f,adm}$. Contrarily, if the accidental situation is mandatory for design, the contribution of persistent load arrangements to $P_{f,adm}$ can be neglected without introducing a considerable error.

Considering these findings, all potentially explosion-exposed members in a building structure (fraction p_e according to Table 8.1) shall be designed according to the mandatory design situation. If this situation corresponds to the persistent situation, two separate hazard scenarios shall be distinguished with their

corresponding occurrence probabilities p_j : member collapse induced by, respectively, an explosion load arrangement (EX), with an associated admissible occurrence probability $p_{f,adm,EX}$ (Figure 4.1, path b_1), and a persistent load arrangement (PER), characterized by probability $p_{f,adm,PER}$ (Figure 4.1, path b_2). On the contrary, if the accidental design situation is mandatory, only probability $p_{f,adm,EX}$ corresponding to the accidental load arrangement (EX) shall be considered in the computation of risks (Figure 4.1, path a_1).

All potentially non-exposed members in a building structure (fraction $1-p_e$ according to Table 8.1) shall be necessarily designed according to the persistent situation and only the hazard scenario under the corresponding load arrangement (PER), characterized by probability $p_{f,adm,PER}$, shall be considered in the risk analysis (Figure 4.1, path b_2).

8.1.3 Consequences

In section 7.5.4 it was shown that the derived consequence model 2 (7.30) delivers reasonable predictions of the collapse-induced n^o of fatalities N_j only up to a certain limit value of the area affected by the collapse, A_{col} (Figure 7.14), what constitutes a severe limitation and rules out its application for estimation of failure consequences in the framework of the present study. Consequently, in spite of the comparatively lower accuracy, consequence model 1 (7.28) is chosen for this purpose. This model allows for an estimation of N_j as a function of the area $A_{col,j}$ and the occupancy ratio of this area, $(Ocu_{col}/A_{col})_j$. The area $A_{col,j}$ is established by equations (7.1) and (7.2) as a function of the hypothesis adopted for characterization of system collapse given a member failure (section 4.5.4) and the geometrical building characteristics (section 4.4.2). For the occupancy ratio, a value of $1/30 \text{ m}^{-2}$ is adopted, which corresponds to the inverse value of the theoretical average net room area per person in Spanish residential buildings (section 7.4.3).

For sake of simplicity, model (7.28) is employed irrespective of the nature of the considered hazard scenario (accidental or persistent). As shown in section 7.5.4, a previous consequence model deduced in the context of normal building use conditions entails similar predictions to those of model (7.28), what justifies this simplified approach.

8.2 Implicitly acceptable risks

8.2.1 Introduction

The occurrence probabilities p_j and the consequences N_j for each of the n_j hazard scenarios associated with a particular building structure can be represented in risk profiles (Figure 4.2). These are presented in section 8.2.2. The areas under these risk profiles correspond to the implicitly acceptable risks to persons associated with a particular structure, R_{adm} . Following their analysis and statistical evaluation in section 8.2.3, the admissible risks R_{adm} are compared to findings from prior studies [10, 13] where exclusively hazard scenarios associated with persistent load arrangements were considered (section 8.2.4).

8.2.2 Risk profiles

The lower and upper envelopes of 48 implicitly acceptable risk profiles corresponding to the 48 different parameter combinations studied (section 4.4.5) for each of the 94 defined geometrical building configurations (Table 4.1) are shown in Figure 8.2.

According to the general description of a risk profile in section 4.2.3, the maximum value on the ordinate represents the implicitly acceptable, accumulated failure probability $P_{f,adm}$ associated with the structure, i.e. the probability that the structure will be affected by a collapse scenario induced by failure of one of its constitutive members within the reference period T_{ref} . According to the results obtained, P_f oscillates between approximately $4.4 \cdot 10^{-4}$ and $1.3 \cdot 10^1$ within $T_{ref} = 50$ years.

The maximum value represented on the axis of abscissa of the risk profiles represents the maximum consequences due to a structural collapse event. In the present case, the maximum number of fatalities N_{max} due to a partial- or total collapse in the studied building structures varies between 1 and 40 fatalities, approximately.

The large variation in both P_f and N_{max} is a first indication of the high scatter associated with the implicitly acceptable risks to persons associated with the analyzed building structures.

Figure 8.2 Lower (left) and upper (right) envelopes of 48 risk profiles corresponding to a specific geometrical building configuration

8.2.3 Evaluation and representation of risks

The implicitly acceptable risks R_{adm} associated with all n_i relevant and statistically independent hazard (collapse) scenarios for one particular building structure correspond to the area under the corresponding implicitly acceptable risk profile, as expressed by (4.10). Since the structures analysed are constituted by statically determinate members (section 4.4.4), they may be interpreted as series systems (section 2.4.2) for which the assumption of statistical independency between failure scenarios leads to conservative estimates of the probabilities of failure. The simplification adopted is also justified because the resulting probabilities and risks are interpreted in comparative terms [13].

Figure 8.3 shows R_{adm} plotted against the total net room area A for the buildings analysed; each point represents the risk associated with one of the 4512 analyzed building structures. The expected number of fatalities is observed to be significantly higher in buildings characterised by smaller venting coefficients (A_v/V) what is reasonable for the corresponding member failure probabilities were found to be comparatively larger (section 6.2). In addition to this anticipated result, the figures on admissible risks to persons associated with the structures analysed are observed to vary widely due to the *scale effect* (section 4.2.3). As expressed by (4.10), risks associated with structures designed in strict accordance with the existing legislation rise with the number of hazard scenarios (n_i), which, in turn, rise with the number of structural members (n_m), failure mechanisms (n_k) and load arrangements (n_l) that may trigger these mechanisms. Since, to some extent, the total building net room area A encompasses these parameters, R_{adm} increases likewise with rising A , as shown in Figure 8.3.

Figure 8.3 Risks R_{adm} vs. building area A and explosion venting coefficient A_v/V ; $T_{ref} = 50$ years

A statistical analysis of the results shown in Figure 8.3, based on the assumption that variable R_{adm} has a lognormal distribution, yields the mean values $R_{adm,m}$ and the 5% and 95 % fractile values $R_{adm,5\%}$ and $R_{adm,95\%}$ respectively, shown in Figure 8.4 (left). In a similar vein, Figure 8.4 (right) and Figure 8.5 show the relationships between admissible risks R_{adm} and, respectively, the total number of members comprising the structure analysed (n_m), the corresponding total number of failure mechanisms (n_k) and, as a function of both, the total number of hazard scenarios (n_j) associated with the structure. The previously mentioned scale effect is clearly visible in all figures.

Figure 8.4 Risks R_{adm} vs. building net room area A [m^2] (left) and vs. n° of members n_m (right); $T_{ref} = 50$ years

Figure 8.5 Risks R_{adm} vs. n^0 of failure mechanisms n_k (left) and vs. n^0 of hazard scenarios n_j (right); $T_{ref} = 50$ years

The scale effect can be eliminated by normalizing the results as shown in Figure 8.6 (left), which plots the implicitly acceptable risks R_{adm} normalized for the total building net room area A as a function of A . Here, too, each point represents the result for one building structure. Similarly, Figure 8.6 (right) and Figure 8.7 offer representations of R_{adm} normalized for the number of structural members n_m , failure mechanisms n_k or hazard scenarios n_j . In all the figures, the relationship between the normalized risks R_{adm} and these parameters can be statistically characterised by a constant mean value and an associated scatter, expressed in terms of the lognormal-based 5 and 95% fractile values shown. In part, the scatter may be attributed to the incomplete and not fully consistent calibration of the rules for designing structural members laid down in the existing standards (section 2.3.3).

It might be observed that $R_{adm}/n_m > R_{adm}/n_k > R_{adm}/n_j$ (compare for instance the mean values shown in Figure 8.6 and Figure 8.7) what is a logical finding since each structure is constituted by a specific number of members n_m , which in turn might fail under different failure mechanisms n_k . Since moreover some of those failure mechanisms might be potentially triggered by different load arrangements n_l of persistent- or accidental character (section 4.2.3, section 8.1.2), it is obvious that $n_j > n_k > n_m$.

Figure 8.6 Risks R_{adm} normalized for building net room area A vs. A [m^2] (left) and R normalized for n^0 of structural members n_m vs. n_m (right); $T_{ref} = 50$ years

Figure 8.7 Risks R_{adm} normalized for n^o of failure mechanisms n_k vs. n_k (left) and R normalized for n^o of hazard scenarios n_j vs. n_j ; $T_{ref} = 50$ years

8.2.4 Comparison to prior studies

Before approaching the task to derive acceptance criteria based on the previously presented results, it is essential to compare these to the findings from prior studies on risks to persons associated with structures exposed to *persistent design situations* [9, 10, 13]. The comparison is possible since the methodology followed and the basic assumptions and hypothesis adopted in both studies are consistent to each other. The following aspects should be kept in mind when performing the comparison:

- The representative set of structures analyzed in the mentioned previous studies includes structures with members of different constitutive materials, such as RC, structural steel, composite and glue-laminated timber. However, to keep the comparison consistent, only the results corresponding to the RC structures are evaluated here.
- The representative set of RC structures considered in [9, 10, 13] is constituted by a total of 2256 different nominal building configurations, characterized by 94 different building geometries and 24 different assumptions concerning both permanent and variable loads (only gravitational loads) acting on the structures. The same 2256 building configurations were analyzed in the present study, where, in addition, two different explosion pressure scenarios are considered, leading to a total of 4512 analyzed structures (section 4.4.5).
- In the previous studies, each hazard scenario was characterized by a specific structural member failure mode triggered by a particular persistent load arrangement. With regard to the corresponding occurrence probability p_j , it was considered that this load arrangement is dominant over all other potential (persistent) load arrangements. Consequently, the number of hazard scenarios per structure was considered equal to the number of failure modes ($n_j = n_k$).
- In the present study, on the contrary, it has been previously shown that, under certain conditions, load arrangements of different nature entail the potential to trigger a specific failure mode. Precisely, it could be observed that explosion-induced load arrangements might significantly contribute to the occurrence probability of certain member failure modes that are also relevant under persistent load arrangements due to gravitational loads. Consequently, in the present study the number of hazard scenarios per structure is larger than the number of failure modes ($n_j > n_k$), as previously exposed.

- Moreover, the explosion hazard entails the potential to trigger additional failure modes, which are not relevant under the considered persistent load arrangements due to gravitational loads. Among those, the negative bending failure of the beams or the potential shear failure of the columns, with comparatively high failure probabilities in many cases due to the *high uncertainties* involved (section 6.2).
- In consequence of both, a higher number of relevant load arrangements n_l and a higher number of relevant failure modes n_k , the number of hazard scenarios n_j associated with the structures analysed in the present study is higher than in the previous studies [9, 10, 13].

Due to the aforementioned *scale effect*, a larger number of hazard scenarios n_j associated with a specific structure reflects ultimately in comparatively higher implicitly acceptable risks R_{adm} . This is shown in Figure 8.8, which compares the risks R_{adm} associated with the structures analysed in both studies, normalised for the building net room area A . The blue dots represent the R_{adm}/A values found in the present study (shown before in Figure 8.6, left), where, in addition to the gravitational loads, gas explosions constitute a potential hazard for structural safety. The corresponding mean value shown in the figure is termed $(R/A)_{TOT,m}$, where subscript *TOT* alludes to the *total* risks, i.e. due to both the persistent (PER) and the explosion (EX) load arrangements (excluding any other potential accidental scenarios). The red dots reflect the risks per unit of net room area obtained in the previous studies [9, 10, 13] where exclusively persistent (PER) situations were considered. It can be observed that the corresponding mean value $(R/A)_{PER,m}$ is about 4.4 times lower than the herein obtained mean $(R/A)_{TOT,m}$.

Figure 8.8 Risks R_{adm} normalized for building net room area A vs. A [m^2] (blue dots); Comparison to results from prior studies (red dots) [9, 10, 13]; $T_{ref} = 50$ years

Since the number of failure modes n_k per unit of net room area A is comparatively higher in structures where, in addition to the persistent scenarios, the accidental load arrangement is considered, the observed difference between the results obtained in both studies can be reduced by performing the comparison in terms of ratio R_{adm}/n_k , shown in Figure 8.9 (left). The ratio between the corresponding mean values $(R/n_k)_{TOT,m}$ and $(R/n_k)_{PER,m}$ is situated in about 2.2.

Figure 8.9 Risks R_{adm} normalized for n^o of failure mechanisms n_k vs. n_k (left) and R_{adm} normalized for n^o of hazard scenarios n_j vs. n_j (right); Comparison to results from prior studies (red dots) [9, 10, 13]; $T_{ref} = 50$ years

Finally, due to the fact that only in the present study the number of failure scenarios associated with a particular structure n_j is higher than the number of failure mechanisms n_k , a further reduction of the difference between the implicitly acceptable risks obtained in both studies is obtained when normalizing these for n_j . This can be drawn from Figure 8.9 (right). The ratio between the mean values $(R/n_j)_{TOT,m}$ and $(R/n_j)_{PER,m}$ is situated in approximately 1.6. The comparatively high failure probabilities associated with many explosion-dominated hazard scenarios, which may be mainly attributed to the large uncertainties affecting the explosion pressure (section 6.2), reasonably explain that this ratio is larger than unity in spite of the normalization of risks for the number of hazard scenarios n_j .

8.3 Acceptance criteria

8.3.1 Introduction

In the previous section, the implicitly acceptable risks to persons associated with building structures designed in accordance with current *best practice* were evaluated. It was shown, that users of building structures, where gas explosions constitute a potential hazard for structural safety, are exposed to comparatively higher risks than in similar structures, where the occurrence of accidental actions is not considered. This intuitively reasonable finding could be attributed to two circumstances, a higher number of relevant hazard scenarios n_j in explosion-endangered structures (scale effect), on one hand, and the high uncertainties associated with the explosion event, on the other.

The first of the mentioned circumstances (scale effect) constitutes the starting point for the definition of acceptance criteria concerning life safety risks associated with structures potentially exposed to the effects of gas explosions. As shown before in section 6.3, depending on the prevailing circumstances in each case, such effects might influence the failure probabilities of members, which under normal use conditions would be exclusively exposed to persistent load scenarios, in a more or less important manner, reaching from non- or scanty relevant to totally dominating contributions. All relevant influences reflect in a higher number of hazard scenarios and, ultimately, in higher risks to persons. The accumulated or total risk, stemming from all relevant persistent- and accidental situations should be limited to a specific *target value*. Regarding individual risks r_i (4.13), for instance, a performance criteria could be expressed in terms of condition (8.1), where $r_{i,PER}$ and $r_{i,EX}$ represent the structure-related individual risks due to, respectively, persistent (PER) and explosion (EX)-induced load arrangements (considered statistically independent from each other). Added up, $r_{i,PER}$ and $r_{i,EX}$ deliver the total individual risk $r_{i,TOT}$ to be assessed with regard to a target value $r_{it,TOT}$. In the same sense, a performance criteria based on

societal risks to persons (4.10) could be established, as expressed through (8.2), where $F(n)_{TOT}$ represents the accumulated or total frequency of a collapse scenario with $n \geq N$ fatalities and $F(n)_{t,TOT}$ the corresponding target value.

$$r_{i,TOT} = r_{i,PER} + r_{i,EX} \leq r_{it,TOT} \quad (8.1)$$

$$F(n)_{TOT} = F(n)_{PER} + F(n)_{EX} \leq F(n)_{t,TOT} \quad (8.2)$$

Decision criteria as expressed per (8.1) or (8.2) could be useful for explicit risk analysis. In routine practice, however, criteria expressed in terms of structural failure probabilities are preferable, as they may serve as a basis for calibrating partial factors in such contexts. A corresponding acceptance criterion can be defined per (8.3), where $p_{f,TOT}$ is the accumulated or total occurrence probability for a particular failure mode k of a specific structural member potentially exposed to persistent (PER) and gas explosion-induced (EX) load arrangements, and $p_{ft,TOT}$ is the corresponding target value.

$$p_{f,TOT} = p_{f,PER} + p_{f,EX} \leq p_{ft,TOT} \quad (8.3)$$

Based on the results presented in section 8.2, the objective of the following sections (8.3.2 and 8.3.3) is to derive target values for the different performance indicators expressed in (8.1) to (8.3), in terms of individual risks $r_{it,TOT}$, societal risk-based failure frequencies $F(n)_{t,TOT}$ or member failure probabilities $p_{ft,TOT}$. On the grounds of the results obtained, and following their comparison to the findings in previous studies [10, 13], section 8.3.4 addresses the development of an acceptance criterion for the conditional failure probability of structural members, given the occurrence of a gas explosion and the following member exposure to the associated load effects. Some remarks on an implementation of this criterion in the light of the current safety requirements for accidental design situations are made in section 8.3.5.

8.3.2 Individual risk

The graphs in Figure 8.6 (left) can be used to define acceptance criteria for individual risks to persons, or the probability that a building user will lose his or her life due to structural collapse in a specific reference period T_{ref} . They represent the expected number of fatalities R_{adm} per unit of net room area A over a 50- year period which would be acceptable pursuant to the existing legislation. In line with the mathematical framework defined in section 4.2.3, the individual risk can be estimated by relating normalized risks (R/A) to the building occupancy rate, i.e. the number of risk-exposed people (n_i) per unit of net room area (A), which depends on building use. For residential buildings the available average net room area A per person i (inverse of occupancy rate) found from Spanish Statistics Institute data [112], $A_i = 30 \text{ m}^2/\text{person}$ (section 7.4.3), is adopted here as a representative value for consequence class CC2 buildings. Factoring this value into (8.4) yields the target value $r_{it,TOT}$ given in Table 8.2, based on, respectively, the mean value $(R_{adm}/A)_{TOT,m}$ and the 5- and 95 % fractile values, $(R_{adm}/A)_{TOT,5\%}$ and $(R/A)_{TOT,95\%}$ (Figure 8.6, left). From Table 8.2 it can also be drawn that the obtained $r_{it,TOT}$ exceed the target values $r_{it,PER}$ deduced in prior studies on risks associated with structures exposed to normal use conditions [9, 10, 13] in about 3 to 5 times. This can be attributed to the comparatively higher implicitly accepted risks R_{adm} per unit of net room area A (Figure 8.8).

Table 8.2 Mean, 5 and 95 % fractile values for individual risk target values r_{it} and associated target failure probabilities p_{ft} ; Comparison to results from prior studies [9, 10, 13]; $T_{ref} = 50$ years.

Scenarios	Explosion and Persistent (TOT)		Only Persistent (PER) [9, 10, 13]	
	$r_{it,TOT}$	$p_{ft,TOT}$	$r_{it,PER}$	$p_{ft,PER}$
5% fractile	2.5E-04	5.0E-03	9.3E-05	1.9E-03
Mean	1.0E-03	2.0E-02	2.4E-04	4.7E-03
95% fractile	2.5E-03	5.0E-02	4.7E-04	9.4E-03

$$r_{it,TOT} \cong \left(\frac{R_{adm}}{A} \right)_{TOT} \cdot A_i \quad (8.4)$$

The $r_{it,TOT}$ values given in Table 8.2 refer to a 50-year period. They can be converted to a 1-year reference period via equation (8.5). This yields values which oscillate between $5 \cdot 10^{-6} \text{ y}^{-1}$ and $5 \cdot 10^{-5} \text{ y}^{-1}$, corresponding to the 5 and 95% fractile values of R_{adm}/A , respectively, with a mean value of around $2 \cdot 10^{-5} \text{ y}^{-1}$. These findings are generally consistent with the criteria laid down in standards, guidelines and technical papers on the safety requirements for built systems in general and structures in particular. As reported in section 2.1.3, the consensus is that the individual risk to persons due to structural collapse should be limited to values ranging from 10^{-6} y^{-1} to 10^{-5} y^{-1} . The close agreement between the implicitly accepted individual risk levels calculated here and the values set out in the standards and recommendations cited, supports the validity of the former as a basis for rational decision-making in connection with structural safety.

$$r_{it,TOT,1y} \cong \frac{r_{it,TOT,50y}}{50} \quad (8.5)$$

Target failure probabilities ($p_{ft,TOT}$) consistent with the $r_{it,TOT}$ values can be deduced based on equation (2.5) introduced in section 2.1.3. The conditional probability of death of a person present in the building at the time of collapse (p_{dlf}), required for this purpose, can be estimated from the statistical analysis of the database built to derive the consequence model (Chapter 7). However, the results obtained have to be interpreted with caution, as the fairly high coefficients of variation of about 100 % denote significant uncertainties in the estimation of p_{dlf} . In order to enable a consistent comparison to the previous studies on persistent design situations [10, 13], the estimate proposed by Steenbergen et al. [58] is adopted here: $p_{dlf} = 0.05$ (for CC2 structures). Factoring this value into (2.5) yields the individual risk-based target failure probabilities $p_{ft,TOT}$ given in Table 8.2 ($T_{ref} = 50$ years). The fact that the obtained $p_{ft,TOT}$ exceed the corresponding target probabilities $p_{ft,PER}$ obtained in previous studies on persistent situations [10, 13] is attributable to the higher acceptable individual risks, r_{it} , as seen before (Table 8.2).

8.3.3 Societal risk

The *scale effect* observed in Figure 8.4 and Figure 8.5 rules out the possibility of defining an acceptable level for societal risks in absolute terms. Rather, the acceptance criterion may be expressed as a function of some parameter accounting for the size of the reference system considered, such as the total net room area A of the buildings.

Figure 8.4 (left) shows the admissible risks R_{adm} as a function of A , where R_{adm} represents the expected number of fatalities associated with a particular building structure in a 50-year period. In turn, the expected number of fatalities associated with a specific system $E(N)$ is given by the area under the respective $F-n$ curve (2.7), introduced in section 2.1.3, which can be calculated approximately with equation

(2.9) if a risk aversion exponent $\alpha > 1$ is assumed. Hence, when R_{adm} is substituted for $E(N)$ in (2.9), the target 50 year cumulative frequency of occurrence for structural collapse events with $N \geq 1$ fatalities, $F(1)_t$, can be deduced as a function of net room area A , whereby the size of the reference system associated with a given F-n criteria is being defined. Given $F(1)_t$ and risk aversion exponent α , assumed as $\alpha = 2$ [10, 13], the F-n acceptance criteria are defined as per equation (2.6). This procedure avoids possible contradictions in the expected number of fatalities and, consequently, ensures that the acceptance criteria calculated for individual and societal risks are fully consistent, as expressed by (2.13) and (4.13).

The F-n curves obtained for building structures with a range of net room areas A between 10^2 and 10^6 m² are reproduced in Figure 8.10 and Figure 8.11 (blue curves). The former shows the criteria based on the mean value of R_{adm} while the latter is based on the 5%- (left) and the 95 % (right) fractile values for R_{adm} . The *scale effect* is visible here in the form of the higher cumulative frequencies $F(n)_t$ for larger reference systems, measured in terms of the net room area A . This is reasonable, since, as mentioned earlier, larger building structures comprise a larger number of structural elements and consequently the number of possible hazard scenarios contributing to the cumulative probability of an accident with $N \geq n$ fatalities is also larger in such cases.

For sake of comparison, the F-n curves deduced in the prior studies on structure-related risks associated with persistent situations [10, 13] are also plotted in Figure 8.10 and Figure 8.11 (red curves). It is observed that the herein obtained target frequencies $F(n)_{t,TOT}$ are remarkably higher what is reasonably explained by the accordingly higher implicitly acceptable risks R_{adm} based on which these frequencies were derived.

Figure 8.10 F-n acceptance criteria for building structures with a range of net room areas A , based on the mean value of expected number of fatalities $R_{adm,m}$; Comparison to results from prior studies (red lines) [10, 13]; $T_{ref} = 50$ years.

Figure 8.11 F-n acceptance criteria for building structures with a range of net room areas A , based on the 5%- (left) and 95% (right) fractile value of expected number of fatalities, $R_{adm,5\%}$ and $R_{adm,95\%}$; Comparison to results from prior studies (red lines) [10, 13]; $T_{ref} = 50$ years.

When the reference system is reduced from whole buildings to individual failure scenarios, i.e. *a specific failure mode in a given structural element*, the F-n curves can be more readily interpreted. In the present context, this can be achieved by establishing F-n criteria consistent with the expected number of fatalities R normalized for the total number of failure modes n_k associated with a specific building structure, plotted in Figure 8.9 (left). Therefore, the target failure frequency $F(1)_t$ is derived from equation (2.9), where $E(N)$ is replaced by, respectively, the mean $(R_{adm}/n_k)_m$ or the fractile values, $(R_{adm}/n_k)_{5\%}$ and $(R_{adm}/n_k)_{95\%}$. Assuming $\alpha = 2$, as above, supplies the normalized F-n curves shown in Figure 8.12 (blue curves), associated with $T_{ref} = 50$ years. The left part contains the criteria consistent with $(R_{adm}/n_k)_m$, the right part shows the results based on $(R_{adm}/n_k)_{5\%}$ and $(R_{adm}/n_k)_{95\%}$. It can be observed that the deduced target frequencies $F(n)_{t,TOT}$ are about 2 times higher compared to the corresponding values obtained in prior studies on persistent situations, $F(n)_{t,PER}$ [10, 13] (red graphs). This is reasonable taking account of the fact that $F(n)_{t,TOT}$ accounts for both persistent and explosion-induced hazards with comparatively high uncertainties in case of the latter.

Figure 8.12 Normalized F-n acceptance criteria $F(n)_{t,TOT}$ for structural members based on the mean value (left) and the 5% or 95% fractile values (right) of the expected number of fatalities $R_{adm,TOT}$ normalized for the number of failure modes n_k ; Comparison to results from prior studies (red lines) [10, 13]; $T_{ref} = 50$ years.

The $F(n)_{t,TOT}$ frequencies shown in Figure 8.12 are converted to target failure probabilities $p_{ft,TOT}$ with equation (2.11), where $p_{N|f}$ describes the conditional probability of $N \geq n$ fatalities in the event of failure. As a reasonable approximation, $p_{N|f}$ can be replaced by the conditional probability $p_{d|f}$ [10, 13] introduced in the previous section. The obtained result is shown in Figure 8.13 (left: mean values; right: 5% and 95% fractile values), where the variable on the horizontal axis has been converted from the number

of fatalities (n) to the area affected by the member collapse (A_{col}) using consequence model (7.28) employed to derive the implicitly acceptable risks (section 8.2). It can be observed that the higher the potential structural damage a member collapse might induce, the smaller is the target failure probability $p_{ft,TOT}$. Once more, the results are compared to the values derived in the previous studies, $p_{ft,PER}$ (red graphs). It is found that $p_{ft,TOT}$ exceeds the corresponding $p_{ft,PER}$ in about 2-3 times.

Figure 8.13 Target failure probabilities $p_{ft,TOT}$ for structural members based on the mean value (left) and the 5% or 95% fractile values (right) of the expected number of fatalities $R_{adm,TOT}$ normalized for the number of failure modes n_k ; Comparison to results from prior studies (red lines) [10, 13]; $T_{ref} = 50$ years.

8.3.4 Conditional failure probability

In the previous sections, target values for risk-based performance indicators related to safety of RC structures have been derived and compared to the results obtained in previous studies [10, 13] where the occurrence of accidental scenarios was excluded from the analysis. It was shown that due to the comparatively higher level of intrinsically acceptable risks associated with structures where in addition to persistent load scenarios the explosion hazard must be accounted for, the derived safety requirements are accordingly lower. This is a principally sound finding and could be traced back to a comparatively higher number of hazard scenarios (*scale effect*) on one hand, and the *high uncertainties* associated with the explosion scenario, on the other.

The mentioned high uncertainties deliver additional arguments to establish the safety requirements for potentially explosion-exposed members at a comparatively lower level. In this sense, the Probabilistic Model Code issued by the Joint Committee on Structural Safety states that in case of “*large uncertainty in either loading or resistance (coefficients of variations larger than 40%), as for instance the case of many accidental or seismic situations, a lower reliability class should be used. The point is that for these larger uncertainties the additional costs to achieve a high reliability are prohibitive....*” [80]. Following this reasoning, a proposal in [55] is to establish a lower target reliability level for wind load-dominated structural members ($\Delta\beta = 1.0$). It is argued that due to the high uncertainties associated with wind loads, the use of the corresponding partial safety factor $\gamma_Q = 1.5$ would entail reliabilities below the target levels established for members where wind is not the dominating influence. Since, on the other hand, increasing γ_Q would be prohibitive due to large economic consequences, compliance with safety requirements for wind-load dominated structures is assured by a reduction of the target reliability index.

Applying this line of argumentation - *more demanding safety levels involve prohibitive costs in case of large uncertainties* - to the present study, higher target failure probabilities $p_{ft,TOT}$ would seem to be defensible in case explosion-induced effects would be dominating the failure probability associated with the failure mechanism of the structural member in question. For the elements studied herein, this would be appropriate, for instance, for the explosion-induced negative bending failure mode of the beams, or

the shear failure scenario of the columns, where under the adopted assumptions, exclusively persistent load scenarios are not relevant for failure. For these member failure modes, the corresponding reliability level is in any case dominated by the explosion-induced load arrangements. On the contrary, in case of other member failure modes, such as positive bending or shear failure of the beams, and failure due to M-N interaction of the columns, it is not straightforward to decide a priori to which extent the explosion load dominates the corresponding failure probability. As shown in section 6.3, there are multiple situations in practice where, notwithstanding that a member might be potentially exposed to accidental loads, the normal use conditions and the associated gravitational loads, have a considerable influence on the failure probability, being totally dominant in many cases, i.e. $p_{f,EX} \approx 0$ in (8.3). For such situations, alluding to the high uncertainties associated with the potential accidental scenario and the associated required higher costs to achieve a certain reliability level would not justify the adoption of higher target failure probabilities. Indeed, these considerations suggests that the target value for members potentially exposed to the effects of gas explosions should rather be a function of the degree of influence of the normal load conditions on the member reliability. As a rule, the lower this influence, the higher should be the target failure probability.

In the following, a proposal will be developed based on this approach. The starting point is the general performance criterion given by (8.3), which can be reformulated as expressed per (8.6), where $p_{ft,EX}$ represents the target probability for a specific member failure mode k to occur in consequence of an explosion-induced load arrangement (EX).

$$p_{f,EX} \leq p_{ft,EX} = p_{ft,TOT} - p_{f,PER} \quad (8.6)$$

The occurrence probability of an explosion event $p(EX)$ can be factored into (8.6) as shown by (8.7), where $p_{ft|EX}$ is the conditional failure probability given an explosion event and the subsequent member exposure to the corresponding action effects, and $p_{ft|EX}$ the corresponding target value.

$$p_{f|EX} \leq p_{ft|EX} = \frac{p_{ft,EX}}{p(EX)} = \frac{p_{ft,TOT} - p_{f,PER}}{p(EX)} \quad (8.7)$$

According to expression (8.7), the target failure probability for an explosion-exposed member decreases with rising contribution of the persistent load arrangements to the member failure probability what is in line with the general idea outlined above. The maximum value for $p_{ft|EX}$ corresponds to situations where this contribution is not relevant or non-existing ($p_{f,PER} \approx 0$). A minimum value for $p_{ft|EX}$ can be defined by limiting probability $p_{f,PER}$ to a target value $p_{ft,PER}$, whereby consistency to the prior studies [10, 13] is assured:

$$p_{f,PER} \leq p_{ft,PER} \quad (8.8)$$

The mean and 5% or 95% fractile values of the target failure probabilities $p_{ft,TOT}$ based on *individual risks* to persons are given in Table 8.2 ($T_{ref} = 50$ years). Substituting these into (8.7) along with a 50 year occurrence probability $p(EX) = 5 \cdot 10^{-4}$ (section 6.2.1), the following expression is obtained for the individual risk-based target value for the conditional failure probability, $p_{ft|EX}$, where C_1 is a constant and $p_{f,PER}$ is associated with a reference period $T_{ref} = 50$ years.

$$p_{ft|EX,IR} = C_1 - 2 \cdot 10^3 \cdot p_{f,PER} \leq 1 \quad (8.9)$$

Where:

$C_{1,5\%} = 10$ (5% fractile value)

$C_{1,m} = 40$ (mean value)

$C_{1,95\%} = 100$ (95% fractile value)

On the other hand, the total target failure probabilities $p_{ft,TOT}$ based on *societal risks* are shown in Figure 8.13 as a function of the area affected by the member collapse, A_{col} . Factoring the corresponding equations into (8.7), along with $p(EX) = 5 \cdot 10^{-4}$, $p_{ft|EX}$ is defined by equation (8.10) as a function of constant C_2 and $p_{f,PER}$, associated with $T_{ref} = 50$ years.

$$p_{ft|EX,SR} = C_2 \cdot A_{col}^{-1.14} - 2 \cdot 10^3 \cdot p_{f,PER} \leq 1 \quad (8.10)$$

Where:

$C_{2,5\%} = 36$ (5% fractile value)

$C_{2,m} = 331$ (mean value)

$C_{2,95\%} = 1061$ (95% fractile value)

The target values $p_{f,PER}$ for the member failure probability $p_{f,PER}$ due to persistent load arrangements (8.8), are adopted from [10, 13] and were previously shown in Table 8.2 and Figure 8.13, based on admissible individual and societal risks, respectively. Factoring the former into (8.9) delivers $p_{ft|EX,IR}$ values larger than unity what denotes that individual risks are not decisive for definition of a conditional failure probability of explosion-endangered members. In a wider sense, this finding is consistent with the “philosophy” of the current design codes on accidental actions (section 1.1.2), which claims that structural design against explosions should, in first place, aim at avoiding collapse events with large failure consequences.

In conformity with this general design approach, the societal risk turns out to be relevant for definition of the conditional member target reliability given the explosion and the subsequent exposure to the action. Factoring the equations corresponding to the $p_{f,PER} - A_{col}$ graphs shown in Figure 8.13 (red lines) into (8.10), the *minimum target value* for the conditional failure probability, $p_{ft|EX}$, is obtained, as a function of constants C_2 and C_3 :

$$p_{ft|EX,SR,min} = C_2 \cdot A_{col}^{-1.14} - C_3 \cdot (0.27 \cdot A_{col}^{0.5} - 1)^{-2} \leq 1 \quad (8.11)$$

Where:

C_2 as in (8.10)

$C_{3,5\%} = 0.2$ (5% fractile value)

$C_{3,m} = 2.3$ (mean value)

$C_{3,95\%} = 7.8$ (95% fractile value)

An approximate, simplified expression of (8.11) is given by (8.12) below, where C_4 is a constant and $A_{col,min}$ is the minimum value for the area affected by the collapse for which (8.12) applies (A_{col} values below $A_{col,min}$ correspond to $p_{ft|EX}$ values larger than unity).

$$p_{ft|EX,SR,min} \approx C_4 \cdot A_{col}^{-1.11}, \quad A_{col} \geq A_{col,min} \quad (8.12)$$

Where:

$$C_{4,5\%} = 20 \text{ (5\% fractile value)}$$

$$C_{4,m} = 167 \text{ (mean value)}$$

$$C_{4,95\%} = 565 \text{ (95\% fractile value)}$$

Figure 8.14 (left) plots the *minimum target value* $p_{ft|EX,min}$ (subscript SR will be omitted in advance) according to expression (8.12). The results based on, respectively, the 5% fractile- (dash-dotted line), the mean- (dashed line) and the 95% fractile value (continuous line) of the implicitly accepted risks are shown. The most restrictive criterion (5% fractile) applies for members whose collapse would affect an area of at least $A_{col,min} \approx 15 \text{ m}^2$. When the mean value criterion is referred to, $A_{col,min}$ increases to 100 m^2 , whereas the result based on the 95% fractile value is not valid below $A_{col,min} \approx 300 \text{ m}^2$.

The *maximum target value* $p_{ft|EX,max}$ for the conditional failure probability $p_{ft|EX}$ is given by (8.10) where $p_{f,PER} = 0$. The corresponding graphs are plotted in Figure 8.14 (right). Again, the result based on the 5% fractile- (dash-dotted line), the mean- (dashed line) and the 95% fractile value (continuous line) of the implicitly accepted risks are distinguished. The corresponding $A_{col,min}$ thresholds are 23 m^2 , 160 m^2 and 450 m^2 , respectively.

Figure 8.14 Minimum (left) and maximum (right) target value for conditional failure probabilities $p_{ft|EX}$ for structural members; T_{ref} for concomitant gravitational loads: 50 years.

A comparison between the graphs shown on the left and the right of Figure 8.14 reveals that the maximum values $p_{ft|EX,max}$ (right) are not fundamentally higher than the corresponding $p_{ft|EX,min}$ (left). However, it could still make a difference to adopt one or the other, for the generally high conditional failure probabilities given the member exposure to the explosion effects, to be compared to these criteria, are generally less sensitive to variations in load- or resistance parameters than failure probabilities of usual order of magnitude (10^{-4} - 10^{-6}). In case it could be assured that the explosion loads are dominant for a certain structural failure type, for instance, in case of failure modes not- or only scanty affected by persistent load arrangements, the maximum values $p_{ft|EX,max}$ (Figure 8.14, right) could be established as a target criterion. On the contrary, when a significant contribution of the persistent situations to the failure probability could not be precluded, the target failure probabilities should be reduced according to (8.10) as a function of $p_{f,PER}$. For sake of simplicity and since it is a conservative approach, the minimum values $p_{ft|EX,min}$ (Figure 8.14, left) could be of general application for verification of structural safety in relation with explosion-exposure. In addition, requirement (8.8) applies in all cases.

In comparison to the rather small differences between the $p_{ft|EX,min}$ and $p_{ft|EX,max}$ criterion, the discrepancies between the 5% and the 95% fractile values for $p_{ft|EX}$ are substantial (more than one order of magnitude). The election of an appropriate criterion should comply with the national safety policy what

requires tackling a number of issues that fall out of the purview of structural engineers. Some considerations and recommendations regarding this election can nevertheless be made:

The established criteria are based on implicitly acceptable risks within the framework of the employed design codes, for all the members in all the buildings analysed comply exactly with the respective structural safety requirements. For this reason, even the criterion corresponding to the superior fractile value (95%), can be considered to provide acceptable solutions. Figure 8.14 shows that the adoption of this criterion would imply relatively high target values for the conditional failure probabilities. For application in practice, this suggests that most structural solutions would be probably acceptable without need for specific measures for reducing explosion-related risks. Only for very large expected failure consequences, such measures might be required.

In the previous studies on structure-related risks to persons associated with normal building use conditions [10, 13], the 95% fractile criteria was suggested for the assessment of existing structural members, whereas the more demanding criteria based on the mean values was associated with member design. It was argued that more demanding safety levels normally involve lower costs in the design phase than in assessment procedures of existing structures, wherefore higher acceptable risks may be reasonably established for the latter. Interestingly, the mean value of the inferred *individual risk* in [10, 13] is of the order of 10^{-6} y^{-1} , whereas the 95 % fractile value corresponds roughly to an individual risk of about 10^{-5} y^{-1} , what complies with commonly acknowledged values for, respectively, design and assessment of structures and other technical facilities (see section 2.1.3).

As in the mentioned previous studies, in the present study, individual risks and societal risks are consistently related to each other (section 4.2.3). Hence, irrespective of the fact that an individual risk-based target value for the conditional failure probability could not be established, the proposed target values shown in Figure 8.14 are implicitly associated with a specific individual risk level. In section 8.3.2, it was shown that implicitly acceptable individual risks associated with RC structures where gas explosions pose a threat to structural safety oscillate around a *mean value* of around $2 \cdot 10^{-5} \text{ y}^{-1}$. Keeping this in mind, and following the general consensus on acceptable individual risk levels for new and existing facilities, it might be reasonable to adopt the mean value criteria shown in Figure 8.14 for purpose of assessment of members in existing structures, while the 5% fractile criteria, associated with an individual risk of about $5 \cdot 10^{-6} \text{ y}^{-1}$, could be envisaged for design of new elements.

The present findings may also be used to determine risk acceptance criteria in the *As Low As Reasonably Practicable* (ALARP) approach [2, 50] (Figure 2.5). In that context, risks higher than those defined by the 95 % fractile values (or the mean values) could be regarded as intolerable, and risks lower than the mean values (or the 5 % fractile values) as broadly acceptable. Risks between these two limits would be tolerable only where reducing them is impracticable or the cost of the necessary measures is grossly disproportionate to the improvement gained [13].

It should be stressed that the target values for the conditional failure probabilities shown in Figure 8.14 are associated with a reference period of $T_{\text{ref}} = 50$ years for the variable loads (imposed loads on building floors and snow loads on roof), which act concomitant to the explosion pressure. A conversion to different reference periods, e.g. $T_{\text{ref}} = 1$ year, would require, in first place, an adaption of the probabilistic models corresponding to these loads. The reliability level implicitly required by the codes could then again be deduced according to step 5 of the established analysis procedure (Figure 8.1). Finally, following the descriptions in the previous subsections, target values for the conditional failure probabilities could be established based on the implicitly acceptable risks associated with $T_{\text{ref}} = 1$ year. The corresponding developments are out of scope of the present thesis.

8.3.5 Final remarks

The preceding section proposes risk-based acceptance criteria for RC structures and their constitutive members. These criteria are in accordance with the general performance requirement laid down in the current Eurocode on accidental action EN1991-1-7 [17], which claims that *a structure shall be designed and executed in a way that it will not be damaged by accidental events to an extent disproportionate to the original cause*. This criterion calls for measures suited to avoid structural collapse scenarios with important failure consequences and, as such, generally tolerates member or component failure, provided that it would not put in danger the stability of the whole structure nor of a significant part of it (section 1.1.2).

In this regard, the herein deduced acceptance criteria for explosion-induced structural failure suggest that risks would be acceptable without any need for verification of structural safety in relation to the explosion hazard if the potential collapse of the member in question would affect an area less than a specific minimum value $A_{col,min}$ (Figure 8.14). In that case, the member could be designed according to the persistent situation under consideration of normal load conditions only, without any further provisions for the accidental loads. The failure consequences due to a potential member failure induced by such loads would be tolerable. Interesting to note is that the $A_{col,min}$ values corresponding to the obtained mean values for the target failure probabilities, of $A_{col,min} = 100 \text{ m}^2$ and 160 m^2 (Figure 8.14), are of the same order of magnitude as the thresholds suggested by EN 1991-1-7 for *limiting the extent of localised failure due to an unspecified cause* (section 2.3.4). The mentioned standard considers an area of $A_{col} = 100 \text{ m}^2$ or 15% of the floor area, whichever is less, on two adjacent floors caused by the removal of any member, as acceptable and likely to provide the structure with sufficient robustness regardless of whether an identified accidental action has been taken into account [17]. This observation supports the plausibility of the herein obtained criteria for rational decision-making in connection with structural safety related to accidental situations.

In case the potential member collapse would affect an area larger than the established $A_{col,min}$, verification of structural reliability associated with the accidental situation would be required. The diminishing target failure probabilities $p_{ft|EX}$ with increasing A_{col} (Figure 8.14) account for aversion to collapse events with large consequences and, in this regard, constitute a powerful tool for the design or the assessment of *key elements*, upon which, according to the definition in EN 1991-1-7 [17], the stability of (the remainder of) the structure depends. For explosion-endangered buildings classified under consequence class CC2 or CC3, the design of such elements is mandatory and should be based upon static load models or prescriptive design and detailing rules, as already remarked in section 1.1.3. However, besides from the above-mentioned limit for admissible local failure due an unspecified cause, no distinctions are made regarding acceptable failure consequences, nor is the required reliability level associated with the design of such elements explicitly established. The implicit, nominal target ceiling demanded in the Eurocode [11] can be inferred from (8.13) under consideration of the 50 year target reliability index $\beta_{t,code} = 3.8$ and the herein considered explosion occurrence probability during this reference period, $p(EX) = 5 \cdot 10^{-4}$. The result, $p_{ft|EX,code} \approx 0.14$, is compared in Figure 8.15 to the herein derived $p_{ft|EX}$ - criteria. It can be observed that for relatively small areas A_{col} , the Eurocode requirement is stricter than the herein proposed target values based on the intrinsically accepted risks. On the contrary, if the collapse of a structural member entails a very large A_{col} , hence very high expected consequences in terms of loss of human life, the herein deduced acceptance criteria imply lower target failure probabilities than the current Eurocode requirement. For instance, the herein developed mean value falls below $p_{ft|EX,code}$ when member collapse might affect an area larger than $A_{col} = 590 \text{ m}^2$ or $A_{col} = 910 \text{ m}^2$, depending on whether

the minimum or the maximum $p_{ft|EX}$ - value is under consideration (Figure 8.15). This consequence-dependant differentiation of the reliability level supposes a substantial improvement of the current approach for treatment of structural safety in relation to accidental situations.

$$p_{ft|EX,code} = \frac{p_{ft,code}}{p(EX)} = \frac{7.2 \cdot 10^{-5}}{5 \cdot 10^{-4}} \approx 0.14 \quad (8.13)$$

Figure 8.15 Comparison of deduced minimum (left) and maximum (right) target value for conditional failure probabilities, $p_{ft|EX}$, to Eurocode target value $p_{ft|EX,code}$ (red line)

Strictly, the deduced criteria are applicable to reinforced concrete structures, constituted by beams and columns (Figure 1.3). However, in the absence of other guidelines, they might as well be applied to more vulnerable structural typologies, such as panel systems. The incident in Palencia in 2009 (Figure 1.2, Figure 7.3), where a building structure (built in 1973), constituted by loadbearing masonry panels sustaining reinforced concrete slabs, entirely collapsed as a consequence of a gas explosion, shows that such structures are prone to the risk of progressive collapse with corresponding large failure consequences. In Spain, as well as in other countries, for instance the UK [38], loadbearing panel structures, constructed several decades ago, are expected to remain in service for an extended period. However, the current version of the Spanish building code [71] does not contain any safety requirements, nor any associated accidental load models, which would allow for an assessment of the risks associated with the effects of gas explosions on building structures. The herein developed models and acceptance criteria could contribute to this purpose.

In summary, the deduced criteria would facilitate the adoption of rational decisions on both, the need and the appropriate choice of risk-reduction measures to counteract the effects of gas explosions in buildings. They could be used in the framework of an explicit risk- or reliability analysis in the context of performance-based design or evaluation, or serve as a basis for the calibration of the implicit rules in structural codes and standards for verification of structural safety in relation to gas explosions (section 2.3.2).

8.4 Summary

Having established the occurrence frequencies p_j of the n_j hazard scenarios associated with a particular building structure (Chapter 6), and following the derivation of a model, which allows for an estimation of the corresponding failure consequences N_j (Chapter 7), Chapter 8 addresses the determination of implicitly acceptable structure-related life safety risks as a function of both, p_j and N_j . Based on the obtained findings, acceptance criteria for such risks are deduced.

A summary of the main assumptions adopted for the determination of p_j and N_j is provided in section 8.1. In addition to the explosion occurrence probability $p(\text{EX})$, the frequencies p_j should account for the probability that the member in question is indeed exposed to the explosion-induced effects (pressure wave) when it occurs (section 8.1.2). For this purpose, the exposure probability p_e is introduced (Table 8.1) and factored into the risk analysis, whereby findings from section 6.3 on the relevance of explosion- and persistent load-controlled hazard scenarios on the accumulated member failure probabilities are taken into account. On the contrary, for the estimation of the failure consequences N_j , the nature of the hazard scenario is not being distinguished (section 8.1.3). Model (7.28) will be employed in association with all n_j hazard scenarios regardless of whether their leading influence is of accidental- or persistent character. An occupancy ratio $(O_{\text{cu,col}}/A_{\text{col}})_j$ of $1/30 \text{ m}^{-2}$ is adopted in (7.28), which corresponds to the inverse value of the average building net-room area per building user in residential buildings in Spain (section 7.4.3).

The implicitly acceptable risk to persons associated with building structures (R_{adm}) are analyzed in section 8.2. These risks are obtained by integration of the implicitly acceptable risk-profiles associated with each of the 4512 building structures analyzed (section 8.2.2, Figure 8.2). Their representation as a function of different parameters characterizing the building size reveals a scale effect (section 8.2.3, Figure 8.4 and Figure 8.5). Risks associated with structures designed in strict accordance with the existing legislation rise with the number of hazard scenarios (n_j), which, in turn, rise with the number of structural members (n_m), failure mechanisms (n_k), and load arrangements (n_l) that may trigger these mechanisms. The mentioned scale effect can be eliminated by normalizing the results (Figure 8.6 and Figure 8.7). Even then, the results are found to be widely scattered what, in part, is attributable to a lack of a fully consistent calibration of the code design rules.

The obtained implicitly acceptable risks R_{adm} are larger compared to findings of previous studies [10, 13] where risks stemmed exclusively from normal building use conditions (section 8.2.4, Figure 8.8). The difference is attributable to the fact that the explosion risk in a building enhances the number of potential hazard scenarios n_j associated with the building structure (scale effect). Interestingly, even after normalization of R_{adm} for n_j , this difference cannot be entirely eliminated (Figure 8.9). The comparatively high member failure probabilities associated with the uncertain explosion load arrangements (section 6.2) reasonably explain this circumstance.

Based on the described results, section 8.3 addresses the development of risk-based requirements for structural safety. Different performance criteria are defined in compliance with the basic requirement that the total risk, stemming from the sum of the relevant persistent- and accidental load-controlled hazard scenarios is to be limited to a specific target value (section 8.3.1). According to the explanations in Chapter 2, a distinction is drawn for structure-related individual (r_i)- and societal risks, the latter expressed in terms of frequencies $F(n)$ of a structural collapse scenario with $n \geq N$ fatalities. Moreover, in view of applications in routine practice, performance criteria in terms of structural failure probabilities (p_f) are established.

Target values ($r_{i,t,\text{TOT}}$, $F(n)_{t,\text{TOT}}$ and $p_{f,t,\text{TOT}}$) for the established risk-based acceptance criteria are derived in accordance with the basic assumptions adopted in prior studies [10, 13] on risks associated with structures under normal use conditions (section 8.3.2 and 8.3.3). A comparison of the results obtained here (Table 8.2, Figure Figure 8.10 to Figure 8.13), to those of the mentioned previous studies, shows that due to the comparatively higher level of intrinsically acceptable life safety risks, R_{adm} , the derived safety requirements are accordingly lower, what is a sound finding. In addition, economical criteria, although not specifically addressed in the present study, deliver reasonable arguments to establish the safety

requirements for potentially explosion-exposed structural members at a comparatively lower level (section 8.3.4). Keeping in mind that more demanding safety levels might involve prohibitive costs in case of large uncertainties, lower requirements, would seem to be defensible in case the reliability level associated with a specific member failure mode is dominated by the uncertain explosion-induced load arrangements. However, in section 6.3 it was shown that there are multiple situations in practice where, notwithstanding that a member might be potentially exposed to accidental loads, the normal use conditions and the associated gravitational loads, have a considerable influence on the member failure probability, being totally dominant in many cases. For such situations, alluding to the high uncertainties associated with the potential accidental scenario, and the associated higher costs to achieve a certain reliability level, would not justify the adoption of less demanding target values. Indeed, this suggests that these values should rather be a function of the degree of influence of the relevant persistent load arrangements on the member reliability. As a rule, the lower this influence the less demanding should be the decision criteria.

Based on these arguments, target values $p_{ft|EX}$ for the conditional failure probability of structural RC members (given that a gas explosion occurs and the member in question is exposed to the associated pressure wave) are proposed in the present study (Figure 8.14). The target value $p_{ft|EX}$ is defined as a function of the contribution $p_{f,PER}$ due to the relevant persistent load arrangement to the total failure probability $p_{f,TOT}$. A maximum value for $p_{ft|EX}$ is established, which corresponds to situations where persistent load arrangements are not relevant to member failure, i.e. where the associated failure probability $p_{f,PER}$ approaches zero. A minimum value for $p_{ft|EX}$ is defined by limiting probability $p_{f,PER}$ in all cases to an admissible value $p_{f,PER}$, whereby consistency to the prior studies [10, 13] is ensured.

The developments show that individual risks are irrelevant to the derivation of $p_{ft|EX}$, what is consistent with the “philosophy” of the current design codes on accidental actions (section 1.1) which claims that structural design against explosions should aim at avoiding collapse events with large failure consequences. Related to such events are societal risks, which turn out to be relevant for the definition of $p_{ft|EX}$. In particular, the $p_{ft|EX}$ are defined as a function of the area affected by the collapse (A_{col}), which is related to the number of fatalities by means of the consequence model employed to determine the implicitly acceptable risks. Minimum areas $A_{col,min}$ are derived below which a specific design solution would be acceptable without need for any intervention regarding explosion-related structural safety, i.e. where member design could be based on the persistent situation under consideration of normal load conditions only, without any further provisions for the accidental load. Beyond these threshold areas, diminishing $p_{ft|EX}$ with increasing A_{col} account for aversion to collapse events with large consequences. In this regard, the deduced criteria constitute a powerful tool for the design or the assessment of key-elements, upon which, according to the definition in EN 1991-1-7 [17], the stability of the structure depends (section 8.3.5). In comparison to the constant, nominal target ceiling in EN 1990 [11], the consequence-dependent differentiation of the target reliability level supposes a substantial improvement of the current approach for treatment of structural safety in connection with gas explosions (Figure 8.15).

Chapter 9 Conclusions and future research

The present and last chapter of the thesis recalls the background of the study, the objectives pursued and the methodology followed to achieve these objectives (section 9.1). Subsequently, the obtained conclusions are presented in section 9.2. In the following, section 9.3 summarizes briefly the most relevant contributions of the thesis. Finally, possible future lines of research are indicated in section 9.4.

9.1 Introduction

Background

In a technical context, *risks* are understood as a mathematical expectation of the consequences of an undesired event (1.1). While not avoidable (Figure 1.1) risks can be analysed, assessed and, if required, reduced by appropriate measures. In daily structural design practice, risks are treated in an implicit manner, i.e. they are not explicitly quantified and the question of their acceptability is judged on the base of prescriptive, codified rules. One of the associated drawbacks is that many objectively known hazards remain undetected. Moreover, the lack of a fully consistent calibration of these rules causes the associated risks to diverge widely what alludes to non-optimal allocation of the available resources for risk reduction (section 1.1.1).

The treatment of risks associated with *accidental actions* on structures is an especially challenging issue. Due to the high uncertainties involved, decision-making related to structural safety accounting for such actions is generally difficult and prone to be based on irrational grounds. Explicit risk analysis offers substantial advantages in this regard. A systematic qualitative risk analysis might contribute to the detection of objectively known, but difficult to recognize accidental actions. In the context of a quantitative risk analysis, specific characteristics inherent to accidental actions, such as low occurrence probabilities, on one hand, and potentially high consequences, on the other, can be judged in a rational manner (section 1.1.2).

Among the accidental actions, *gas explosions in buildings* are a good example. Despite the continuous modernization of gas installations and appliances, available statistics from different countries show that the occurrence rate of such explosions in buildings does not seem to decrease in a significant way. While the hazard potential is known and recognized, and although dealt with in many design codes, gas explosions are seldom accounted for in the design and evaluation of ordinary building structures. The low occurrence probability evokes reluctance to allocate resources to mitigate the associated risks, which, as a consequence, are often ignored and sometimes consciously accepted. Whether this is, or not, a justified practice cannot be easily answered however, since these risks are not quantified nor are acceptable risk levels established (section 1.1.3).

Objectives

On this background, the present thesis aims at exploring practical methods and tools for the analysis and evaluation of structure-related risks associated with the effects of gas explosions on reinforced concrete structures. The particular *objectives* of the study are (section 1.3):

- To deduce and establish methods and models for estimation of both the probabilities and the consequences of a gas explosion-induced collapse in ordinary RC building structures.

- To employ these models in order to determine the implicitly accepted risks to persons associated with such structures.
- To deduce rational acceptance criteria that may be employed for the assessment of explosion-related structural risks within the context of an explicit risk analysis, or serve as a basis for a risk-based calibration of semi-probabilistic models used in daily practice.

Approach

Decision-making related to technical facilities in general, and structures in particular, unavoidably requires addressing *life safety risks* in order to assure that persons are safeguarded from undue threats to their life or health to the highest affordable level [209]. According to the Marginal Life Saving Cost (MLSC) principle established in the international standard [36] on the reliability of structures, this level is closely related to the societal willingness to pay for saving one statistical individual and can be quantified by means of the Life Quality Index (LQI) (section 1.2). However, the standard also states that “an activity which is found to be acceptable should be assessed in regard to the corresponding absolute level of life safety risk”, and that the practical implementation of the MLSC principle by using the LQI might require the specification of “absolute values of the acceptable life safety risks”. Indeed, in the opinion of social scientists the public at large would be unlikely to accept higher failure rates than associated with *current best practice* [10, 13] and practice would change if the frequency of collapse with a certain degree of consequences were to rise to unacceptable levels [3]. For these reasons, the present study explores life safety risk-related acceptance criteria associated with building structures compliant with current best practice, in turn reflected by the structural design codes and standards in force. Risk acceptability therefore depends on the *degree of reliability implicitly required by these codes and standards*, which is unknown and which, as previous studies show, might differ fundamentally from nominal target ceilings established in these standards.

Mathematical framework

In section 4.2, a mathematical framework for the quantification of structure-related risks is defined. The developments, which are in line with the main principles established in prior studies dedicated to risks in buildings under normal use conditions [9, 10, 13], are based on the state of knowledge regarding explicit approaches for the analysis, assessment and management of risks associated with technical systems, outlined in Chapter 2. In the present context, such a system is described by a particular building structure. Each of the n_j *hazard scenarios* (j) associated with such a structure is represented by a specific collapse scenario triggered by failure of one of its principle loadbearing members, where member failure is characterized by a specific failure mode induced by a particular load arrangement. The hazard scenarios are mathematically described by their occurrence *probability* p_j and the associated *consequences* N_j . Assuming statistical independency between the n_j hazard scenarios, these can be represented in a probability (p_j) -consequence (N_j) diagram, also termed *risk profile* (Figure 4.2). The integral of the risk profile corresponds to the *risk R associated with the structure* in question.

Procedure

Based on the established mathematical framework, in section 4.3 a procedure (Figure 4.4) is defined which specifies the tasks performed to comply with the established objectives. The procedure comprises the definition of a representative set of ordinary building structures with RC elements (beams and columns) in section 4.4, followed by an identification of the most relevant hazard scenarios (j) associated with these structures and elements in section 4.5. The determination of the occurrence probabilities p_j and the associated consequences N_j forms part of the subsequent quantitative risk analysis. Probabilities

p_j are expressed in terms of implicitly acceptable member failure probabilities $p_{f,adm}$, which represent the reliability implicitly required by the structural standards. The tasks performed for their determination comprise the translation of the defined hazard scenarios into limit state functions (LSF) (sections 4.6), a strict design of the members ($E_d = R_d$) based on these LSF (section 4.7) and the characterisation of basic variables involved (section 4.8). Moreover, the estimation of probabilities $p_{f,adm}$ called for the quantification of different dynamic effects associated with the defined member limit states (Chapter 5) and, finally, a reliability analysis (Chapter 6). For the estimation of consequences, N_j , in terms of the number of fatalities associated with the hazard scenario, a model is required what is subject of Chapter 7. As a function of both, p_j and N_j , in Chapter 8 the implicitly acceptable risks associated with the selected structures are established and acceptance criteria deduced from the findings.

Codes and standards employed

For the purpose of the present study, the Spanish building codes (CTE) on basis of design and actions are used while structural resistance is based on standard EHE (section 4.7.1). To a large extend, these codes are compatible with the respective Eurocodes (sections 4.7.4, 6.2.4).

9.2 Developments and conclusions

9.2.1 Introduction

In the following, the most relevant developments and the obtained conclusions of the study are outlined, classified according to the three main objectives persuaded (sections 9.2.2 to 9.2.4). Important additional findings are presented in section 9.2.5.

9.2.2 Methods and models for estimation of structure-related risks

Occurrence rate of gas explosions

The occurrence probability of gas explosions is quantified based on a literature survey (section 3.1.2) from which the following conclusions can be drawn:

- The good correlation among the findings of different studies suggests that the probability of an explosion event per time unit λ seems to be almost independent of the considered time period what in light of the continuous modernization of gas installations and appliances could not have been expected (Table 3.1).
- A value of $\lambda = 10^{-5}$ provides a good estimate for the annual occurrence of a gas explosion in a gas-supplied housing-unit in western countries.
- Assuming a uniform distribution in time, this figure can be used to model the occurrence probability $p(EX)$ in a specific time period T_{ref} based on a Poisson process.

Hazard scenarios and limit states

Accidental hazard scenarios and associated limit state functions (LSF) for the defined building structures and their constitutive members were established based on the findings of a previous literature study, complemented by sound assumptions (section 4.5, section 4.6). The following aspects are highlighted in this context:

- Accidental load arrangements for members in building structures are defined under consideration of gravitational loads acting simultaneously to the explosion pressure (Figure 4.8 to Figure 4.10). As a reasonable simplification, this simultaneous action is accounted for by means of standardized coefficients for combination of loads in accidental situations, ψ_1 (section 2.3.4).
- Under the defined load arrangements, bending and diagonal shear failure, in case of the columns influenced by axial forces, are identified as the relevant member failure modes. Direct- or punching shear failure modes are not relevant to member reliability in connection with gas explosions (section 3.2.3).
- The definition of the LSF for column failure under bending moment-axial force interaction takes account of the fact that the reliability level is strongly load-path dependent (Figure 2.18, Figure 4.11). A possible cross-section failure upon reaching the ultimate moment is distinguished from buckling failure (Figure 4.12).

Pressure generation in gas explosions

A probabilistic model for the peak pressure generated in a gas explosion ($p_{EX,peak}$) is derived from a regression analysis on the experimental results which constitute the fundament of the Eurocode load-model. The probabilistic model is characterized by the following features:

- A lognormal distribution.
- A mean value, which is defined in function of the characteristics of the venting panels providing pressure relief during the explosion event, i.e. the resistance pressure p_{stat} and the venting coefficient A_v/V (Figure 4.21).
- An associated coefficient of variation of 0.7, which, under a required confidence level of $\gamma = 0.95$, takes account of statistical uncertainties due to the limited number of available test results (Table 4.9, Table 4.10). Its magnitude corroborates indications in [24] and is indicative of the complex physical relationships and dependencies between the many variables involved in the pressure build-up process (section 3.1.3).

Dynamic effects on loads and structural resistance

Dynamic effects on the structural behavior during blast load exposure, such as the contribution of inertia forces and energy-dissipation due to plastic deformations are accounted for by means of dynamic load factors DLF_E (Figure 4.20). Moreover, the study takes into account the alteration of the material properties (strain rate effects) under relatively high loading rates, for which purpose dynamic increase factors DIF_m are introduced (Figure 4.24).

Simplified dynamic models (section 3.3) are deployed in the framework of a deterministic analysis procedure (Figure 5.6) to quantify both DLF_E and DIF_m under the explosion load exposure of the RC members. For this purpose, the explosion load is represented as an idealised pressure-time function (Figure 4.7). The development of the dynamic resistance function is built upon a moment-curvature analysis what, in case of the columns, required a specific solution algorithm to accommodate the axial force dependent formulation of structural resistance under consideration of the material- and geometrical nonlinearities involved (Figure 5.13 to Figure 5.16). The most important results obtained in the deterministic member analysis are summarized below.

- The DLF corresponding to the flexural failure mode of the beams manifest a significant decrease of the required resistance with rising deformation capacity with up to 50% in case of the members subjected to load arrangement $p_{EX(+)}$ (Figure 5.18), where the explosion load acts in the same direction as the gravitational loads (Figure 4.8). When exposed to load arrangement $p_{EX(-)}$ (Figure 5.21), where gravitational- and explosion load act in an opposed sense (Figure 4.8), this decrease may rise up to approximately 70%.
- On the other hand, it seems likely that the beams are not able to resist the dynamic shear forces that arise during the flexural response close to the member supports. At the time of tensile-controlled shear failure, only about 1/3 of the energy dissipation capacity corresponding to the flexural failure mode is taken advantage of (Figure 5.18). On the contrary, a compression-controlled shear failure of the beams seems highly unlikely to occur (Figure 5.19).
- In comparison to the beams, the analysis of the columns, reveals far less disparity in the energy absorption capacity associated with the different failure modes. Due to a comparatively high flexural stiffness, accentuated by the influence of compression axial forces, this capacity is generally small what manifests in similar DLF for bending and shear failure modes, with values larger than unity in many cases (Figure 5.30). Among them are the buckling-critical members, whose maximum deformation capacity concurs with initiation of yielding of the longitudinal reinforcement.
- Although the material strain rates which gas explosions might induce in RC members are relatively low in comparison to those associated with other explosion types, the mechanical material properties are nonetheless substantially enhanced.
- The highest dynamic increase is experienced by the concrete tensile strength and Young's modulus involved in the computation of the dynamic resistance function for the beams exposed to load arrangement $p_{EX(-)}$, which as an average, increase by around 25% (Table 5.5). The average increase of the concrete compressive strength oscillates between 12 and 17 %, depending on the considered failure mode. Slightly below these values, between 10 and 14%, is the average increase of the concrete compressive strain under maximum stress (Table 5.3, 5.5, 5.7)
- The dynamic increase of the reinforcing steel yield strength is characterized by an average DIF between 1.10 and 1.16, while the alteration of the tensile strength is almost negligible as denoted by an average increase between 3 and 5 % only (Table 5.3, 5.5, 5.7).

The simplified dynamic analysis procedure and the associated assumptions were verified by means of a previously validated non-linear finite element model (section 5.5). A comparative study for 18 RC beams was conducted with the following conclusions:

- The numerical results agree closely with the simplified model predictions, particularly with respect to flexural behaviour (Figure 5.38 to Figure 5.40)
- The simplified methods might underestimate the support reactions of RC members performing in the post-yield response stage (Figure 5.41). This can be attributed to the contribution of higher response modes to the distribution of inertia forces along the members, triggered upon formation of the plastic hinge (Figure 5.43).

- The impact of this finding on the present study is negligible for the critical tension tie shear failure mode occurs far before the yield strength of the longitudinal reinforcement of the beams is reached.

Regarding a stochastic characterization of the dynamic load effects and structural resistance, the following remarks should be made:

- Uncertainties associated with the simplified dynamic models are factored to the DLF_E by means of coefficients $\zeta_{E,dyn}$ (Figure 4.23), characterized by lognormal distributions with mean $\mu_{\zeta_{E,dyn}} = 1.0$ and a standard deviation $\sigma_{\zeta_{E,dyn}}$, based on test observations (section 3.3.6) and/or subjective estimations ($\sigma_{\zeta_{M,dyn}} = 0.2$, $\sigma_{\zeta_{V,dyn}} = 0.3$, $\sigma_{\zeta_{N,dyn}} = 0.05$).
- Due to a lack of evidence, the scatter associated with the dynamic material strength involved in the LSF (concrete compressive strength, yield- and tensile strength of reinforcement) is assumed as under quasi-static loading conditions (Figure 4.25).

Reliability method for columns under M-N exposure

A reliability procedure for the analysis of columns under the considered M-N interaction-sequence due to the occurrence of a gas explosion is presented (section 6.1). The principal objective of this procedure is the determination of the strain distribution in the critical member cross-section, corresponding to the most likely-failure point on the limit state surface, also known as FORM-design point (Figure 6.2, Figure 6.3). Application of the procedure entails the following findings:

- The assumption of mean values for the relevant material properties, including strain-rate effects, provides failure point estimations that approach the FORM design point solution in a sufficiently exact manner (Figure 6.4).
- On the contrary, the use of the strain distribution obtained in the code design stage of the explosion-exposed columns could entail distorted reliability levels for the corresponding failure point location might be completely different from the FORM solution (see also section 9.2.5). This is particularly the case if the code design predicts failure in the compression-controlled region of the M-N interaction diagram (Figure 6.5). For such cases, the developed reliability procedure provides close approximations to the FORM design point in an iterative manner (Figure 6.6).

Failure consequences

For purpose of the establishment of a consequence model, data on explosion-induced structural collapse events in Spain and other western countries is gathered in the framework of a systematic press-survey (section 7.2). Following an evaluation and statistical complementation of the compiled data, a rigorous, multi-linear regression analysis on the relevant observations is carried out (section 7.5). The obtained model predicts the number of fatalities ($N_{col,j}$) due to a specific member collapse scenario. The following conclusions are obtained:

- In addition to the area affected by the collapse ($A_{col,j}$), the occupancy ratio of this area (Ocu_{col}/A_{col})_j is relevant to the explanation of the generally large variability in $N_{col,j}$.
- Under the adoption of reasonable assumptions for (Ocu_{col}/A_{col})_j, the derived regression model shows a good conformity to models derived in previous studies to predict the number of fatalities due to structural collapse events in the context of persistent situations [9, 10, 13] (Figure 7.15).

9.2.3 Implicitly accepted risks to persons

Reliability level implicitly required by the codes

The implicitly accepted risks to persons depend on the reliability level implicitly required by existing structural standards (section 9.1). This level is analyzed in section 6.2 for a representative set of structural RC members, with the following conclusions:

- The general need for accidental design provisions associated with gas explosions is evidenced. It is shown that in many practically relevant situations, the accidental design rules might demand a higher member resistance than provided in an ordinary design for persistent situations. Ignoring this, the reliability level might fall below the level implicitly required by the design codes for the accidental situations (see e.g. Figure 6.9).
- This level is generally subject to a high scatter. Important differences are observed when comparing the obtained reliability indices for different members and, especially, for different member failure modes. In many cases, the nominal 50 year-target ceiling ($\beta_{t,code} = 3.8$) is not met. The tension tie-shear failure mode of the studied members turns out to be especially critical, while the failure of the compression strut is comparatively unlikely, results that corroborate the findings of the deterministic structural analysis.
- The sensitivity coefficients α_i associated with the FORM design point vector reveal that, almost irrespective of the member type and failure mode, the uncertain explosion-induced action effects dominate the failure probabilities of the members where the accidental design situation is mandatory. The influence of geometrical or material variables, as well as static loads, on the likelihood of an explosion-induced collapse of these members is comparatively small wherefore they could be treated as deterministic quantities in the reliability calculations (see e.g. Figure 6.7, right).

Implicitly accepted risks

The determination of the implicitly acceptable risks accounts for the fact that not all members constituting a building structure are potentially exposed to the considered accidental load combinations. Moreover, it is considered that, in addition to the potential accidental load exposure, certain member failure modes might be triggered by persistent load arrangements associated with normal building use conditions (section 8.1.2). Under these assumptions, implicitly acceptable risk profiles (Figure 4.2) are obtained for each of the representative building structures and integrated, what leads to the following conclusions:

- The implicitly acceptable risks R_{adm} associated with structures designed in strict accordance with the existing legislation rise with the number of hazard scenarios (n_j), which, in turn, rise with the number of structural members (n_m), failure mechanisms (n_k), and load arrangements (n_l) that may trigger these mechanisms (Figure 8.4, Figure 8.5).
- In consequence of this scale effect, the obtained R_{adm} are about 4 times larger compared to findings of previous studies, where risks stemmed *exclusively* from normal building use conditions [10, 13] (Figure 8.8). Even after normalization of R_{adm} for n_j , this difference cannot be entirely eliminated what is attributable to the comparatively high member failure probabilities associated with the uncertain explosion load arrangements (Figure 8.9).

9.2.4 Acceptance criteria

Different performance criteria are defined in compliance with the basic requirement that the total risk, stemming from the sum of the relevant persistent- and accidental load-controlled hazard scenarios is to be limited to a specific target value. According to the explanations in Chapter 2, a distinction is drawn for structure-related individual (r_i)- and societal risks, the latter expressed in terms of frequencies $F(n)$ of a structural collapse scenario with $n \geq N$ fatalities. Moreover, in view of applications in routine practice, performance criteria in terms of structural failure probabilities (p_f) are established (section 8.3.1).

Target values ($r_{i,t,TOT}$, $F(n)_{t,TOT}$ and $p_{f,t,TOT}$) for the established risk-based acceptance criteria are derived in accordance with the basic assumptions adopted in prior studies [10, 13] on risks associated with structures under normal use conditions (Table 8.2, Figure 8.10 to Figure 8.13). A comparison of the results obtained here, to those of the mentioned previous studies, shows that due to the comparatively higher level of intrinsically acceptable life-safety risks R_{adm} , the derived safety requirements are accordingly lower, what is a sound finding. In addition, economical criteria, although not specifically addressed in the present study, deliver reasonable arguments to establish the safety requirements for potentially explosion-exposed structural members at a comparatively lower level. Keeping in mind that *more demanding safety levels might involve prohibitive costs in case of large uncertainties*, lower requirements, i.e. higher target failure frequencies, would seem to be defensible in case the reliability level associated with a specific member failure mode is dominated by the uncertain explosion-induced load arrangements. In other words, the lower the influence of persistent load arrangements on the member reliability, the less demanding should be the decision criteria.

In line with these considerations, target values $p_{ft|EX}$ for the conditional failure probability of structural RC members (given that a gas explosion occurs and the member in question is exposed to the associated pressure-wave) are proposed in the present study. The main findings are summarized as follows:

- The target probability $p_{ft|EX}$ is defined as a function of the contribution $p_{f,PER}$ due to the relevant persistent load arrangement to the total failure probability $p_{f,TOT}$. A maximum value for $p_{ft|EX}$ can be established for situations where such loads are not relevant to member failure, i.e. where $p_{f,PER}$ approaches zero. A minimum value for $p_{ft|EX}$ is defined by limiting $p_{f,PER}$ in all cases to a target value $p_{ft,PER}$, whereby consistency to the prior studies [10, 13] is ensured.
- Individual risks are found to be irrelevant for the derivation of $p_{ft|EX}$, what is consistent with the “philosophy” of the current design codes on accidental actions claiming that structural design against explosions should aim at avoiding collapse events with large failure consequences. Related to such events are societal risks, which turn out to be relevant for the definition of $p_{ft|EX}$. In particular, $p_{ft|EX}$ is defined as a function of the area affected by the collapse (A_{col}) which is related to the number of fatalities by means of the consequence model employed to derive the implicitly acceptable risks (Figure 8.14).
- Minimum areas $A_{col,min}$ are derived below which a specific design solution would be acceptable without need for any intervention regarding explosion-related structural safety, i.e. where member design could be based on the persistent situation under consideration of normal load conditions only, without any further provisions for such accidental load. The obtained $A_{col,min}$ values based on the mean value of the implicitly acceptable risks are 100 m² and 160 m², respectively, for the minimum and the maximum $p_{ft|EX}$ criteria (Figure 8.14). The fact that these values are of the same order as the threshold areas suggested by EN 1991-1-7 [17] for *limiting the extent of*

localised failure due to an unspecified cause, supports the plausibility of the herein deduced acceptance criteria.

- Beyond $A_{col,min}$, the $p_{ft|EX}$ diminish with increasing A_{col} expressing aversion to collapse events with large consequences. In this regard, the deduced criteria constitute a powerful tool for the design or the assessment of *key elements*, upon which, according to the definition in EN 1991-1-7 [17], the stability of (the remainder of) the structure depends. In comparison to the constant, nominal target ceiling in [11], the consequence-dependent differentiation of the required reliability level supposes a substantial improvement of the current approach for treatment of structural safety in connection with gas explosions (Figure 8.15).
- In the absence of other guidelines, the deduced criteria could also be applied to more vulnerable structural typologies, such as panel systems. As denoted by several incidents, these systems, which are still relatively common in the existing Spanish building stock, and elsewhere, are prone to the risk of progressive collapse with corresponding large failure consequences. The current version of the Spanish building code [71] does not contain any safety requirements, nor any associated accidental load models, which would allow for an assessment of the associated risks to persons (section 4.7.1). The herein developed models and criteria could contribute to this purpose.
- In summary, the deduced criteria would facilitate the adoption of rational decisions on both, the need and the appropriate choice of risk-reduction measures to counteract the effects of gas explosions in buildings.

9.2.5 Additional findings

The deduced models, methods and criteria could be used in the framework of an explicit risk- or reliability analysis, or serve as a basis for the calibration of the implicit rules in structural codes and standards for verification of structural safety in relation to gas explosions. Regarding such a possible calibration, both the deterministic- and the reliability analysis of the RC members carried out in the present study have furnished a number of important collateral findings, which are summarized below:

- The RC beams, in spite of being designed in a strict manner ($E_d = R_d$) for both, bending and shear, to resist the nominal explosion pressure specified in the applied code, are likely to fail in shear much before the ductile, flexural response might be fully developed. This finding is consistent with the frequently quoted observation that structural members, when exposed to blast loads, might fail in shear, even if they were designed to exhibit a ductile, flexural behavior under quasi-static loading conditions (section 3.2).
- In order to perform in the more energy absorbing flexural mode, the beams would require either a higher shear resistance or a lower bending resistance and/or stiffness, what constitutes a parallelism to the capacity design approach for seismic action. The latter option could be achieved by considering the beneficial effect of energy dissipation explicitly in the flexural design stage of explosion-exposed members, for instance by introducing dynamic load factors (DLF), such as obtained in the present thesis.
- In addition, the very dispersed, failure-mode dependent ratios between the mean and design resistance of the beams, R_m/R_d (Table 5.4), suggest to analyze to which extent the current formulations for the design values of the material parameters are based on rational grounds. The subsequent reliability analysis of the members corroborates the need for such an investigation.

It is shown that the ratios between resistance R^* , computed from the individual components x_i^* of FORM design point vector $\mathbf{x}^*$, to the mean resistance R_m approach unity with increasing influence of the explosion load on the member reliability (Figure 6.8, right, Figure 6.13, Figure 6.16, right). This is attributable to the simultaneous decrease of the impact of the resistance variables (geometry and material parameters) on the member failure probability, what implies that the corresponding design point coordinates x_i^* approach the mean values. On the contrary, the ratios between R^* and the code-based design resistance R_d are found to deviate from unity with rising influence of the explosion load (Figure 6.8, left, Figure 6.14, Figure 6.16, left).

- These observations indicate that a more realistic representation of the design material parameters, including the explicit consideration of failure mode- (and load-type) specific dynamic increase factors (DIF) might contribute to a more rational design approach for certain accidental situations.
- A similar conclusion is drawn from the analysis of the columns under M-N interaction. The analysis reveals that the failure domains of the members might strongly differ from those anticipated in the design stage. In particular, it is shown that the failure point on the interaction diagram based on mean values for the material properties (including strain rate effects) is located close to the FORM design point solution, while it might be situated in a failure domain that completely differs from the code-design predictions (Figure 5.25 to Figure 5.27, Figure 6.19).
- Only the particular characteristics of the explosion-induced load-path 2 (Figure 4.11) make this phenomenon possible. Hence, while current design rules might be appropriate for member design under persistent situations, where mainly gravitational actions prevail, they might deliver irrational results if applied to accidental situations characterized by extraordinary sequences of bending moments and axial forces. For such situations, a shift of the design values for the material parameters towards more realistic boundaries, including dynamic effects, could help to circumvent irrational design solutions.

9.3 Original contributions

The most relevant contributions of the present study can be briefly summarized as follows:

- Definition, implementation and validation of a procedure for the simplified dynamic analysis of structural members under consideration of strain-rate effects, material- and geometrical nonlinearities and the influence of gravitational loads acting previously to the explosion exposure.
- Validation of this procedure and the associated adopted hypothesis by means of a numerical model for RC beams, where especially the analysis of the member reaction forces in the post-yield stage constitute an original contribution to a scanty explored research field.
- Definition and implementation of a procedure for the reliability analysis of RC columns under M-N interaction with an, a priori, unknown failure point.
- Establishment of the reliability level implicitly required by the codes for RC members under the influence of gas explosion-induced loads, including the previous definition of appropriate limit states.
- Improvement of available models for the estimation of structural collapse-related consequences to persons.

- Quantification of the implicitly acceptable risks to persons associated with RC structures potentially exposed to the gas explosion-hazard and comparison to risks stemming exclusively from persistent design situations.
- Inference of a consequence-dependent target reliability level for the design or the assessment of key-elements under potential explosion-exposure, including threshold values for a minimum consequence level ($A_{col,min}$) below which a particular design solution would be acceptable without need for any intervention regarding explosion-related structural safety.

9.4 Future research

Among the many possible subjects for future research related to the present study, the following should be prioritised:

- The scope of this study is limited to RC structures constituted by beams and columns. An extension to other construction materials and structural typologies is important, among them panel structures, which are prone to the risk of explosion-induced progressive collapse.
- In line with the assumption of static determinate structural members, reasonable assumptions were adopted in the present study to describe the extensions of a system collapse scenario given a local member failure (section 4.5.4). Future studies could involve static indeterminate systems including an analysis of structural robustness under consideration of the system collapse probability given a specific local failure.
- The scope of the present study is limited to gas explosions. An extension to other accidental actions is paramount. Among such actions, the seismic action should be prioritised for its crucial role in the context of life safety risks.
- In addition to life safety risks, economic aspects, based on the Life Quality Index (LQI), should be considered for the derivation of structure-related acceptance criteria. Establishing a relation between the results obtained in the present study to economic requirements would also facilitate the evaluation of fire-related risks.
- The comparison between the derived consequence model to those obtained in prior studies suggests to derive a unique model from a joint database comprising collapse events due to all kind of influences in and on buildings associated with all types of consequence classes (section 7.5.4). Moreover, the estimate of the conditional probability for one or more persons to lose their life given a structural collapse event should be improved.
- The derived failure probabilities and risks are to be understood as nominal quantities. Establishing a relation to “real” values, taking account of human error and a certain degree of conservatism in current structural design practice (section 6.2.5), would facilitate a comparison to risks associated with other technologies and thereby contribute to a global approach for rational decision-making.
- The present study reveals significant scope for improving the implicit design rules in structural codes and standards for verification of structural safety in relation to accidental situations (section 9.2.5). Future studies should envisage a consistent calibration of these rules.

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Annexes

Annex A SDOF Analysis of columns including 2nd order effects

A.1 Introduction

The present Annex pretends to illustrate the implementation of the single degree of freedom (SDOF) analysis under the influence of axial forces (section 3.3.4) by means of an example column. Special attention is being paid to the influence of second order effects on the dynamic member performance. The sensitivity of expression (5.7) for the dynamic load factor associated with failure due to M-N interaction to varying second order levels is to be quantified.

A.2 Example

For sake of simplicity, a simply supported column subjected to a uniformly distributed explosion load following a equilateral triangular pressure pulse, $p_{EX}(t)$, and a *constant* axial compression force N_E without initial eccentricity nor initial deformation will be considered here (Figure A.1). For this case, the dynamic second order moment $M_{E,dyn,II}$ is given by the following simple expression, where δ is the mid-span displacement in transverse direction.

$$M_{E,dyn,II} = N_E \cdot \delta \quad (A.1)$$

Figure A.1 Schematic representation of considered column

Assuming, *elastic structural behavior* ($\mu_\delta = 1.0$), the resisting moment $M_{R,dyn}$ is defined as a function of mid-span displacement δ , member stiffness k (inferred from cross-sectional stiffness EI), and the column height, l_0 (section 3.3.3).

$$M_{R,dyn} = \frac{R \cdot l_0}{8} = \frac{k \cdot \delta \cdot l_0}{8} \quad (A.2)$$

Equations (A.1) and (A.2) can be substituted into the definition of second order ratio $\varphi_{M,II}$ given by (5.6). If moreover the difference between a quadratic and a sinusoidal curvature distribution along the member is being disregarded, it can be shown that $\varphi_{M,II}$ is approximately equal to the ratio between axial force N_E and the critical buckling force according to the first Euler case, N_{cr} .

$$\varphi_{M,II} \approx \frac{N_E}{N_{cr}} = \frac{N_E}{\frac{\pi^2 EI}{l^2}} \quad (A.3)$$

The definition of the dynamic load factor DLF_M for the simplified example considered here is given as follows where moment $M_{EX,peak}$ would result from a static application of peak-pressure $p_{EX,peak}$.

$$DLF_M = \frac{M_{EX}}{M_{EX,peak}} = \frac{M_{R,dyn} \cdot (1 - \varphi_{M,II})}{M_{EX,peak}} \quad (A.4)$$

In the following, it will be assumed that the duration of the equilateral triangular load-pulse acting on the described example-column is equal to three times the natural period, i.e., $t_p / T = 3.0$. Under this assumption, Figure A.2 shows, for different $\varphi_{M,II}$ ratios (A : $\varphi_{M,II} = 0$; B : $\varphi_{M,II} = 0.3$; C : $\varphi_{M,II} = 0.6$; D : $\varphi_{M,II} = 0.9$), the resistance, $M_{R,dyn}(t)$ (black, dashed curves), the second order moment, $M_{E,dyn,II}(t)$ (blue curves), the difference between both, $M_{R,dyn}(t) - M_{E,dyn,II}(t)$ (red curves), and moment $M_{EX}(t)$ (green lines) which would be obtained if the lateral pressure $p_{EX}(t)$ would be applied statically to the member. All curves are *normalized to the maximum value of $M_{R,dyn}$* .

Figure A.2 Elastic response $M_{R,dyn}(t)$ to applied moment $M_{EX}(t)$ following a triangular load-pulse of duration $t_p = 3 \cdot T$, under influence of second order moment $M_{E,dyn,II}(t)$: $\varphi_{M,II} = 0$ (a), $\varphi_{M,II} = 0.2$ (b), $\varphi_{M,II} = 0.6$ (c), $\varphi_{M,II} = 0.9$ (d).

From Figure A.2-a it can be deduced that if no second order effects are present ($\varphi_{M,II} = 0$), the required resistance $M_{R,dyn}$ (peak of black curve) is about 15% higher than the peak-pressure related bending moment $M_{EX,peak}$ (peak of green curve), i.e., $DLF_M \approx 1.15$. From an alternative point of view, it may be stated that the column can sustain a moment $M_{EX,peak}$ of not more than $1/1.15 \cdot 100 \approx 87\%$ of the provided maximum resistance $M_{R,dyn}$.

When second order effects (blue curves) are being introduced, the resistance available to oppose the laterally applied explosion pressure diminishes as illustrated by the red curves, which reflect the difference between the total resistance $M_{R,dyn}(t)$ and the second order moment $M_{E,dyn,II}(t)$. Hence, the explosion

peak-pressure, which can be sustained by the member, reduces. As can be inferred from Figure A.2-b to d, Figure A.1, $M_{EX,peak}$ reduces from 87% of the provided resistance $M_{R,dyn}$ (when $\varphi_{M,II} = 0$), to, respectively, 70%, 36% and 7%, when $\varphi_{M,II}$ is being increased to 0.3, 0.6 or 0.9. Since, as can be deduced from these numbers, the decrease of $M_{EX,peak}$ is almost proportional to $(1 - \varphi_{M,II})$, the DLF_M (A.4) experience only small changes from 1.15 ($\varphi_{M,II} = 0$) to, respectively, 1.00 ($\varphi_{M,II} = 0.3$) and 1.11 ($\varphi_{M,II} = 0.6$). Only in case of very high second order effects, i.e. $\varphi_{M,II} = 0.9$, this proportionality gets lost as denoted by the DLF_M of 1.50. However, this finding of reduced importance, since in this unrealistic case the problem has been converted into an almost static second-order analysis, where the laterally applied explosion load is of negligible magnitude (Figure A.2-d), and only serves as triggering event for inducing member buckling.

A.3 Final observations

The previous example suggests that the DLF_M as defined per (5.7) is almost insensitive to the influence of second order effects. The reduction of the maximum resisted explosion peak-load upon introducing such effects is counteracted by the almost proportional reduction of the available structural resistance, what, in consequence, entails only insignificant variations of the DLF_M . Figure A.3 shows that this finding applies to all t_p/T -ratios, which may characterize the dynamic performance of the studied columns.

It should be kept in mind that elastic behavior ($\mu_\delta = 1.0$) was assumed in the present example, which is representative for the buckling-critical members (BLS members), where second order effects are relatively important. On the contrary, non-buckling-critical columns (UMLS members), with certain plastic deformation capacity, are associated with comparatively lower resulting axial force levels or slenderness ratios, i.e. lower second order effects. As a general rule, the lower such effects, the larger is the ability of the columns to undergo significant plastic deformations. This is duly taken accounted for in the definition of the member resistance functions in section 5.3.4.

Figure A.3 Dynamic load factor DLF_M (A.4) as a function of ratio between duration of the triangular load-pulse t_p to the natural period T , ductility ratio μ_δ and second order ratio $\varphi_{M,II}$ (A.3).

Annex B Analysis of the longitudinal oscillations in blast-exposed columns

B.1 Introduction

The simplified analysis of the RC columns by means of the equivalent SDOF system under the inclusion of axial forces provides the flexural response in the transverse direction of the members. However, this system cannot capture the dynamic effects in the longitudinal sense of the columns. In order to analyse these effects, numerical calculations have been carried out under use of the FE-code Ansys [1]. The present Annex briefly summarizes this preliminary study and the results obtained.

B.2 Numerical model

A schematic representation of the numerical model is shown in Figure B.1. The columns are modelled by 2-D elastic BEAM3 [1] elements with tension, compression and bending capabilities. These elements are defined by their cross-sectional area A , moment of inertia I and mass m . In addition, material properties have to be assigned to the beams elements, in the present case, modulus E and Poisson ratio ν . The plastic deformation capacity of the columns due to yielding of the longitudinal reinforcement is being accounted for by a rotational spring-element, COMBIN40 [1], connected to the beam elements at the intermediate cross-section. The element has one degree of freedom at each of its two coincident nodes. Rotation θ_s is triggered when moment M reaches the previously defined yield moment, M_{Ry} . Prior to this point, the assignment of an extremely high stiffness k_s ($k_s \rightarrow \infty$) impedes the rotation of the spring.

Figure B.1 Schematic representation of FE model

The pin-ended columns are subjected to load arrangement $N_{EX(-)}$ (Figure 4.9), with the exception that the initial eccentricity e_{min} is not considered here. The explosion, characterized by the triangular load-pulse of peak-load $p_{EX,peak}$ and duration t_p (Figure 4.6), generates a uniformly distributed explosion load $p_{EX}(t)$ acting laterally on the members. Simultaneously, the explosion induces a tensile force component $N_{EX}(t)$ which opposes the previously present static compression force N_{G+Q} due to the gravitational actions. The fact that the gravitational force N_{G+Q} might act with certain inertia when the columns are stroke laterally by the pressure wave [2], is not considered here.

B.3 Example

A column of $l_0 = 4$ m height is considered. The strict design (section 4.7) of the intermediate cross-section ($b = 0.2$ m; $h = 0.28$ m; $A = b \cdot h$) delivered the required amount of longitudinal reinforcement to resist the combined influence of bending moments and axial forces. Subsequently, the simplified analysis of the column (section 5.2) yielded cross-sectional stiffness $EI_{\text{dyn}} = 5688$ kNm², related to stiffness k_{dyn} (Figure 5.X), assigned to the BEAM3 elements. Moreover, a yield moment $M_{\text{Ry}} = 93$ kNm was inferred from the simplified analysis and allocated to the spring (Figure 5.16).

The example column is subjected to a static axial force of $N_{\text{G+Q}} = 81$ kN. The explosion induces a dynamic tensile force $N_{\text{EX}}(t)$ characterized by a triangular-shaped pulse of duration $t_p = 0.2$ s and a peak value of $N_{\text{EX,peak}} = 62.5$ kN. The simultaneously acting lateral explosion load-reaches its peak-value at $p_{\text{EX,peak}} = 50$ kN/m.

The numerical solution procedure (full transient dynamic analysis [1]), which took account of geometrical non-linear behavior, yielded the results shown in Figure B.2. On the left, the bending moment is plotted versus deflection δ . The yield-plateau at $M = M_{\text{Ry}} = 93$ kNm is clearly visible. The obtained displacement ductility ratio, $\mu_\delta = 1.75$, matches unsurprisingly well the estimations according to the simplified SDOF analysis. This was expected, since the established FE model, by means of linear-elastic beam elements and a rotational spring at the intermediate cross-section, is actually a numerical representation of the SDOF system.

Figure B.2 Moment-deflection (left) and axial force-time (right) relationships of the studied column

The added value of the FE model (with respect to the analytical SDOF model) is the possibility to analyze the evolution of the dynamic response in the longitudinal direction of the columns. To this end, Figure B.2 (right) shows the axial force-time plot. It can be observed that the axial force level follows clearly the applied triangular-shaped pressure pulse, suggesting that oscillations in the longitudinal direction of the column are practically insignificant. The maximum of about $N_{\text{max}} = -15$ kN (compression) is reached at $t_p/2 = 0.1$ s and at $t_p = 0.2$ s, the axial force level has returned to the initial load, $N_{\text{G+Q}} = 81$ kN. Consequently the explosion induced a tensile axial force of about 66 kN in the member, which is only about 5% larger than the peak-value of the externally applied tensile force, $N_{\text{EX,peak}} = 62.5$ kN, hence $DLF_N \approx 1.05$. It should also be observed that the mentioned tensile axial force of 66 kN includes a contribution of around 2 kN due to second order effects (the perpendicular action of the laterally applied load $p_{\text{EX}}(t)$ on the deformed column induces an additional tensile force component in the column), which the simplified analysis does not consider. Without this effect, DLF_N would reduce to about 1.02. Dynamic load factors close to 1.0 are indicative of quasi-static behavior. This is related to the comparatively small natural period corresponding to the axial response mode of the column, in the present case $T \approx 8 \cdot 10^{-3}$ s.

This value is very small in relation to the load-pulse duration t_p ($t_p/T \approx 25$) and hence the influence of inertia effects on the structural behavior is practically negligible.

B.4 Final observations

In an analogous manner to the previous example, a number of columns with varying geometrical and loading properties were modeled and simulated. The main conclusion which can be drawn from these simulations is that a dynamic load factor of $DLF_N = 1.0$ (mean value) is a reasonable assumption for purpose of the simplified analysis of the RC columns. Possible minor deviations from this value, such as detected in the presented example, are accounted for in the reliability analysis (chapter 6) by the corresponding coefficient of variation of 5 % (section 4.8.2).

B.5 References

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Annex C Models for the deformation capacity of RC members

C.1 Introduction

The definition of the resistance function for purpose of the simplified dynamic analysis of the beams and columns under the influence of the explosion load implies assumptions concerning their deformation capacities. In section 5.2.2, member deformation capacity was introduced in terms of the displacement ductility ratio μ_δ , given as a function of the curvature ductility ratio μ_χ through expression (5.10), adopted from [1]. By means of a comparison to alternative approaches, the objective of the present Annex is to validate the usefulness of (5.10) for the purpose of the present study.

C.2 Deformation capacity based on equivalent plastic hinge length

For simple structural elements, the relationship between curvature and displacement ductility ratios, respectively μ_χ and μ_δ , can be expressed by integrating the curvatures χ along the length x of the members [2]:

$$\mu_\delta = \frac{\delta_{max}}{\delta_y} = \frac{\int \chi_{max}(x) x dx}{\int \chi_y(x) x dx} = K \cdot \mu_\chi \quad (C.1)$$

Constant K depends on the shape of the curvature distribution along the member axis at yielding of the tensile reinforcement, $\chi_y(x)$, and at maximum response, $\chi_{max}(x)$. However, the integrations of equation (C.1) are tedious and some approximations are necessary [2]. Figure C.1 [3] shows an idealized bending moment and curvature distribution along a simply supported beam under a concentrated load F applied at mid-span. In the elastic range, the curvature increases linearly from the supports towards the mid-span until moment M_y is reached upon the initiation of yielding of the tensile reinforcing steel. Beyond this point, in the so-called plasticity-, or plastic hinge region, of length l_p , the curvature experiences a sudden increase due to the inelastic deformations. The integral of the corresponding plastic curvature $\chi_p(x)$ (shaded area), i.e., the difference between total and yield curvature, $\chi_{max}(x)$ and $\chi_y(x)$, delivers the plastic hinge rotation θ_p according to (C.2). For purpose of a simplified mathematical description of the plasticity region, θ_p may be conveniently represented by an equivalent rectangle of height χ_p (*plastic curvature at mid-span*) and length l_p , commonly referred to as the *equivalent plastic hinge length*, e.g. [2, 4-6]. It should be noted that θ_p and l_p as defined here, refer to the rotation and the length on one side of the plastic hinge.

$$\theta_p = \int [\chi_{max}(x) - \chi_y(x)] dx = \chi_p \cdot l_p \quad (C.2)$$

Figure C.1 Idealized moment and curvature distribution of a simply supported beam under a concentrated load at mid-span; adapted from [3]

As proceeded with the total curvature, the total mid-span deflection δ_{max} can as well be idealized into an elastic and plastic contribution, respectively, δ_y and δ_p (C.3), and hence equation (C.1) can be rewritten as per (C.4).

$$\delta_{max} = \delta_y + \delta_p \quad (C.3)$$

$$\mu_\delta = \frac{\delta_{max}}{\delta_y} = \frac{\delta_p + \delta_y}{\delta_y} = 1 + \frac{\delta_p}{\delta_y} \quad (C.4)$$

Based on the assumption of a linear curvature distribution along the member (Figure C.1), δ_y can be obtained from (C.5), while δ_p is expressed as a function of the equivalent plastic hinge length l_p (C.6).

$$\delta_y = \int_0^{l/2} \chi_y(x) x dx = \chi_y \frac{l^2}{12} \quad (C.5)$$

$$\delta_p = \int_{l/2-l_p}^{l/2} \chi_p(x) x dx = \chi_p \frac{l_p}{2} (l - l_p) \quad (C.6)$$

Factoring (C.5) and (C.6) into (C.4), the displacement ductility ratio μ_δ is defined as a function of the curvature ductility ratio μ_χ , the shear span l_s (here $l_s = l/2$) and the equivalent plastic hinge length, l_p .

$$\mu_\delta = 1 + 3 \frac{l_p}{l_s} \left(1 - \frac{l_p}{2l_s}\right) (\mu_\chi - 1) \quad (C.7)$$

The total hinge rotation $\theta_{\max}$, which under the rigid body assumption (Figure C.2) is related to the maximum displacement $\delta_{\max}$ by (C.8), can likewise be formulated as a function of the equivalent plastic hinge length l_p , as given by (C.9):

$$\theta_{\max} = \text{atan}\left(\frac{2 \cdot \delta_{\max}}{l}\right) \cong \frac{2 \cdot \delta_{\max}}{l} \quad (C.8)$$

$$\theta_{\max} = \chi_y \cdot \frac{l_s}{3} + l_p \left(1 - \frac{l_p}{2l_s}\right) \cdot (\chi_{\max} - \chi_y) \quad (C.9)$$

Figure C.2 Rotation θ of a simply supported beam assuming the beam-halves to behave like rigid bodies

C.3 Comparisons

By means of equations, (C.7) and (C.9) the displacement-ductility ratio μ_δ and the support rotation $\theta_{\max}$ of a simply supported (or cantilevered) member can be readily derived from a simple cross-sectional analysis at mid-span. The difficulty lies, however, in the estimation of the equivalent plastic hinge length, l_p . A number of experimental studies have been carried out in the past with the aim to quantify l_p , e.g. [2, 4], with quite different results. The large number of influence parameters on the rotation capacity of reinforced concrete members, the complex interaction between those parameters and more basic issues, like differences in the test conditions or a lack of a unique definition of the ultimate limit state [5], hinder a straightforward definition of l_p in practice. Nevertheless, it is possible to indicate an order of magnitude. The study by Panagiotakos and Fardis [6], who systematically analyzed and evaluated a database comprising 875 results of flexure-controlled tests of RC members (beams, columns, walls) under monotonic and cyclic loads, seems to be suitable for this purpose. The study focused on the ultimate rotation capacity θ_u of the members, which was associated with a drop of at least 15% of the applied peak force in the tests. Taking as a base previous research from Paulay and Priestly [2], Panagiotakos and Fardis [6] suggested the equivalent plastic hinge length l_p to be a function of shear span l_s , which they considered to be the most influent parameter on the rotation capacity, and the product between bar diameter d_b and yield strength f_y . Regression analysis showed expression (C.10) to provide the best fit of equation (C.9) to the experimentally obtained ultimate rotations θ_u of the monotonically loaded members, among which a large number were cantilevered or simply supported under concentrated load exposure [6].

$$l_p = 0.18 \cdot l_s + 0.021 \cdot a_{sl} \cdot d_b \cdot f_y \quad (C.10)$$

The second term on the right-hand-side of equation (C.10) accounts for an additional rotation capacity due to the anchorage of the rebars beyond the section of maximum moment according to [2]. For the herein considered simply supported members, this term can be neglected ($a_{sl} = 0$) and hence l_p is found to be proportional to l_s . Factoring the remaining term ($l_p = 0.18 \cdot l_s$) into (C.7), the μ_δ - μ_χ relationship shown in Figure C.3 (blue line) is obtained. Surprisingly, the result practically coincides with equation (5.10), likewise represented in Figure C.3 (red line). Although this probably rather casual coincidence

should not be paid too much attention, it provides evidence for the adequacy of (5.10) for the estimation of the deformation capacity of simple RC members, such as those analyzed in the present study. The fact that the herein considered beams and columns are subjected to uniformly distributed loads, which entail larger plastic rotation capacities compared to members under concentrated loads [5], does not lower the meaningfulness of this statement.

Figure C.3 Relation between displacement- and curvature ductility ratio according to (5.10) and (C.7) (the latter in connection with (C.10))

In order to judge the prediction capacity of equation (C.9) in conjunction with (C.10), Panagiotakos and Fardis [6] statistically evaluated ratio M of predicted to experimental rotation capacities θ_u . The obtained mean value μ_M of 1.37 showed that the experimentally deduced rotations are underestimated by use of equations (C.9) and (C.10). Moreover, a coefficient of variation of $CoV_M = 0.94$ denotes an important associated scatter which partly can be attributed to the fact that several of the important influence parameters on the rotation capacity θ_u are implicitly dealt with through l_p . The study conducted by Panagiotakos and Fardis [6] shows that when considering these influences in a more explicit way, the model accuracy can be significantly enhanced. A regression analysis including all statistically relevant variables for the rotation capacity of the monotonically loaded members included in the database yielded a model with an improved prediction capacity (in relation to the l_p -based model) as denoted by an associated mean value and coefficient variation for ratio M of $\mu_M = 1.17$ and $CoV_M = 0.57$, respectively. When performing the regression analysis on all available test results, including the cyclically loaded members, these values could be further reduced to $\mu_M = 1.06$ and $CoV_M = 0.47$. The corresponding model, given by equation (C.11), depends on:

- the steel type (coefficients α_{st} and α_{cyc})
- the anchorage characteristics of the rebars beyond the section of maximum moment ($\alpha_{sl} = 1$ if bond slippage must be accounted for, 0 if not)
- the element type ($a_{wall} = 1.0$ for shear walls and 0 for beams or columns)
- the influence of axial load ratio $v = N/f_c A_c$
- the relation between compression and tensile reinforcement index ω'/ω
- the cylindrical concrete compressive strength f_c

- the shear span ratio at the section of maximum moment l_s/h
- the influence of confinement (term including confinement factor α [MC90], transverse reinforcement ratio ρ_w and yield strength f_{yw})
- the diagonal reinforcement ratio ρ_d

$$\theta_u(\%) = \alpha_{st} \alpha_{cyc} \left(1 + \frac{\alpha_{sl}}{2.3}\right) \left(1 - \frac{\alpha_{wall}}{3}\right) 0.2^\nu \left[\frac{\max(0.01; \omega')}{\max(0.01; \omega)} f_c\right]^{0.275} \left(\frac{l_s}{h}\right)^{0.45} 1.1^{(100\alpha\rho_w \frac{f_{yw}}{f_c})} 1.3^{100\rho_d} \quad (C.11)$$

Equation (C.11) is used in the following to predict the ultimate rotation capacities θ_u of the 18 RC beams analyzed in section 5.5. The beams are characterized by a ratio between compression and tensile reinforcement index of $\omega'/\omega = 0.2$. A medium reinforcing steel ductility class ($\alpha_{st} = 1.25$; $\alpha_{cyc} = 1.0$) and perfect bond conditions ($a_{sl} = 0$) are assumed for this purpose. For the plastic hinge of the simply supported members is assumed to develop around the mid-span cross-section, no confinement of the concrete core due to transverse reinforcement is considered ($\alpha = 0$). Concrete compressive strength f_c is replaced by the dynamic value $f_{c,dyn}$ including dynamic increase factor. Following these assumptions, equation (C.11) delivers ultimate rotations θ_u between 2.6 and 3.6 degrees for the 18 members analyzed, as shown in the left part of Figure C.4 (blue dots). Interestingly, these rotations are far less dependent on the tensile reinforcement index ω than those deduced from equation (C.8) in conjunction with δ_{max} based on (5.10), assumed in the simplified analysis of the RC members (red dots). When beams characterized by small tensile reinforcement indices ω are referred to, the herein adopted simplified approach overestimates the rotation capacity predicted by the regression model (C.11) and vice versa. However, the degree of over- or underestimation is influenced by other factors too, such as, for instance, the amount of compression reinforcement. This can be drawn from Figure C.4 (right) which compares θ_u according to (C.8) and (C.11) under the assumption that the compression reinforcement is being omitted ($\omega' = 0$) in the calculation of the rotation capacity of the 18 beams. It can be observed that under this hypothesis equation (C.11) is far more sensitive to changes in ω . Both approaches deliver now very similar θ_u for the majority of the 18 example beams considered (significant deviations only for very small ω).

Figure C.4 Rotation capacity θ_u of 18 simply supported beams (section 5.5.2) according to simplified approach (C.8) and empirical regression model by Panagiotakis and Fardis (C.11); Left: $\omega'/\omega = 0.2$; Right: $\omega' = 0$

C.4 Final observations

Without exploring in further detail the numerous influences on the rotation capacity of RC members and the corresponding modelling approaches, the above presented comparisons suggest that equation

(5.10) provides reasonable estimates of the average ductility capacity of simple RC members, such as those analyzed in the present study. Moreover, the presented results indicate that (5.10) tends to underestimate the actual deformation capacity to some extent. This appreciation is corroborated by suggestions in the design manual TM 5-1300 for structures exposed to accidental explosions [7], where rotation capacities $\theta_u < 5$, as obtained for most of the beams analyzed herein (Figure C.4), are associated with a moderate damage level. These observations should be kept in mind when analyzing the influence of energy dissipation on the reliability level of the blast load exposed RC members (chapter 6).

C.5 References

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Annex D Validation of the moment-curvature algorithm for columns

D.1 Introduction

The objective of the present Annex is the validation of the proper functioning and implementation of the developed algorithm for the moment-curvature ($M-\chi$) analysis of the columns under the influence of variable axial forces and bending moments presented in section 5.3.4. The validation will be carried out by means of a case study under simplified load assumptions what facilitates the interpretation of the results.

D.2 Example-column

The main characteristics of the example column are shown in Figure D.1. The simply-supported member of length l_0 is exposed to a monotonically increasing axial force N_E (D.1) applied with an eccentricity $e_{min} = 50$ mm with respect to the center of gravity of the quadratic, symmetrically reinforced cross-section. Since no horizontal load will be considered here, the increase of the first order bending moment $\Delta M_{E,I,S}$ is exclusively defined by the eccentric action of the applied axial force, as expressed by (D.2). Expressions (D.1) and (D.2) substitute (5.36) and (5.39), respectively. All other equations constituting the algorithm maintain their validity.

$$N_{E,S} = N_{E,S-1} + \Delta N_{E,S} \quad (D.1)$$

$$\Delta M_{E,I,S} = \Delta N_{E,S} \cdot e_{min} \quad (D.2)$$

In order to study the influence of second order effects, a short and a slender column will be analysed in the following, characterised by a length of $l_0 = 1$ m (slenderness ratio $\lambda \approx 11$) and $l_0 = 8$ m ($\lambda \approx 92$), respectively.

Figure D.1 Static system and loading of the example-column studied for purpose of the validation of the $M-\chi$ algorithm (left) and corresponding cross-section geometry and principal material characteristics (right)

D.3 Results

D.3.1 Short column

The present section summarizes the results obtained for the short column ($\lambda \approx 11$). Figure D.2 shows the M-N interaction diagram corresponding to the *ultimate resistance* of the column's cross-section. Moreover, the sequentially applied (in $s = n$ steps) action effects are plotted in the figure, which can be splitted into a first and a second-order contribution. As could be expected, for the short-column analysed here, the second order effects are virtually negligible and hence the increase of the bending moment with respect to the axial force is almost proportional. The failure point is defined by the intersection of the applied action effects with the interaction diagram.

The almost negligible influence of second order effects can also be drawn from Figure D.3, which shows the obtained moment-curvature response $M_{R,s}(\chi)$ at the intermediate cross-section to the step-wise applied action effects $M_{E,s}(\chi)$ (red). It is shown that the increase of the first order moments $\Delta M_{E,I,s}$ is far more important than the corresponding second-order component $\Delta M_{E,II,s}$. The latter is characterised by slope $m_{II,s}$ (5.42), which slightly increases with each load-step due to the monotonic increase of both axial force and the member deflection. Nevertheless, due to the small member slenderness, this increase is too small to cause the member to buckle. Indeed, as can be observed in Figure D.4, which offers an amplified view of Figure D.3, at $\chi_s = \chi_y$ slope $m_{II,s}$ is still clearly below slope $m_{R,s}$ of the plastic response stage of the moment-curvature diagram $M_{R,s}(\chi)$ and hence condition (5.42) is not fulfilled. Consequently, the plastic response stage can be taken account of and failure will not occur before the ultimate bending moment M_{Ru} is being reached, when concrete deformation ϵ_c at the intermediate cross-section will exceed the ultimate deformation ϵ_{cu} under simultaneous yielding of the compression reinforcement layer.

The resulting first order moment $M_{E,I,y}$ at the yield point ($\chi_s = \chi_y$) can be obtained by summing up increments $\Delta M_{E,I,s}$ corresponding to the n_y load-steps applied up to that point, as expressed by (D.3). It can then be readily verified that the obtained $M_{E,I,y}$ equals the product between the axial force $N_{E,y}$ and eccentricity e_{min} . Assuming $N_{E,y}$ to be a constant quantity ($N_{E,y} = N_E = \text{const.}$), the total moment M_E is expressed by (D.4) as a function of curvature χ . As expected, function $M_E(\chi)$, plotted in Figure D.3 and Figure D.4 (blue, dashed line), intersects $M_R(\chi)$ at $\chi = \chi_y$.

$$M_{E,I,y} = \sum_{s=1}^{n_y} \Delta M_{E,I,s} = N_{E,y} \cdot e_{min} \quad (D.3)$$

$$M_E = N_E \cdot e_{min} + 0.1 \cdot l_0^2 \cdot N_E \cdot \chi \quad (D.4)$$

Figure D.2 Moment-axial force diagram, and sequentially applied action effects to short column ($l_0 = 1$ m)

Figure D.3 Moment-curvature diagram, and sequentially applied action effects to short column ($l_0 = 1$ m)

Figure D.4 Moment-curvature diagram (ultimate resistance), and sequentially applied action effects to short column ($l_0 = 1$ m) – amplified view around the yield point.

D.3.2 Slender column

The present section summarizes the results obtained for the slender column ($\lambda \approx 92$). Figure D.5 shows the M-N interaction diagram corresponding to the *yield resistance* of the column's cross-section. In addition, the sequentially applied action effects, splitted again into a first and a second-order contribution, are plotted. Contrary to the findings of the short column, the second order effects are important in the present case, and consequently, the bending moment gains increasing importance with rising axial force level in the column. This is clearly indicated by the progressively decreasing slope of the graph representing the resulting action effects (Figure D.5, red, continuous line). The intersection of this graph with the M-N interaction diagram signalises failure of the member, in this case due to buckling. The corresponding axial force level of about 1200 kN is far below the value at failure of the short column, of almost 2900 kN.

The importance of second order effects can also be drawn from Figure D.6, which, as before for the short column, shows the obtained moment-curvature response $M_{R,s}(\chi)$ at the intermediate cross-section to the step-wise applied action effects $M_{E,s}(\chi)$. It might be observed that the slope $m_{II,s}$ (5.42) of second-order component $\Delta M_{E,II,s}$ significantly increases with each load-step. Since at $\chi_s = \chi_y$, $m_{II,s}$ exceeds slope $m_{R,s}$ of the plastic response stage of the moment-curvature diagram $M_{R,s}(\chi)$, i.e. condition (5.42) is fulfilled, an irreversible equilibrium loss occurs before the plastic response stage can be entered - the member buckles. The same conclusion is obtained if the axial force and the first order moment corresponding to this point ($\chi = \chi_y$) are assumed to be constant quantities. As introduced before in the context of the short column, the total moment $M_E(\chi)$ is then given by a linear function (D.4) in the M- χ space, which as shown in Figure D.6, is tangent to the $M_R(\chi)$ diagram at $\chi = \chi_y$ (blue, dashed line).

Figure D.5 Moment-axial force diagram (yield resistance), and sequentially applied action effects to slender column ($l_0 = 8$ m)

Figure D.6 Moment-curvature diagram, and sequentially applied action effects to slender column ($l_0 = 8$ m)

D.4 Final observations

The above presented results for the studied example column analysed confirm the proper functioning and implementation of the developed algorithm for the moment-curvature (M - χ) analysis of the RC columns under the influence of variable axial forces and bending moments presented in section 5.3.4.

Annex E Resistance of a short column under dynamic M-N interaction

E.1 Introduction

In section 5.1.3, it was stated that the approach for accounting the material behaviour under dynamic loading in the present study is in several ways a rough simplification. The associated doubts and uncertainties are mainly related to the adequacy of the average strain rates to be employed in the DIF formulations and to the suitability of these formulations for their application in problems beyond the limited scope of the empirical studies from which they were derived. Concerning the latter aspect, it is of special interest to analyse if the DIF models, which were deduced from uniaxial strain-rate tests on concrete or steel specimen, can be suitably employed in the structural response prediction of RC members under more complex load arrangements.

In this context, the objective of the present Annex is to determine the ultimate resistance of a short column tested in [1] and to compare the result to the experimental findings.

E.2 Example-column

The column in question (Figure E.1) was submitted to a dynamic axial force $N(t)$ in a way that a specific, compressive strain rate $\dot{\epsilon}_c$ at the intermediate cross-section was kept constant throughout the test. The imposed strain rates were $\dot{\epsilon}_c = 3.3 \cdot 10^{-6} \text{ s}^{-1}$, representative for quasi-static loading, and $\dot{\epsilon}_c = 1.67 \cdot 10^{-2} \text{ s}^{-1}$. The axial force was applied at the center of gravity of the members ($e = 0$) and, in order to introduce M-N interaction, at an eccentricity of $e = 49 \text{ mm}$.

Figure E.1 Characteristic of column studied by Scott et al. [1]

E.3 Prediction of ultimate resistance

The M-N interaction diagrams associated with the ultimate cross-section resistance of the member (UMLS) are derived under adoption of the stress-strain relationships shown in Figure 4.24. The corresponding material parameters for the quasi-static load assumptions are indicated in Figure E.1. Figure E.2 shows the result obtained (blue curve).

Figure E.2 M-N interaction diagrams obtained for the example-column shown in Figure E.1. Blue: Static material parameters; Red: Dynamic material parameters.

The DIF formulations summarised in section 5.1.3 are used for the determination of the dynamic concrete compressive strength $f_{c,dyn}$, the compressive strain at peak-stress $\epsilon_{c1,dyn}$, as well as for the dynamic yield and tensile strength of the reinforcing steel, respectively $f_{y,dyn}$ and $f_{t,dyn}$. For the estimation of $f_{c,dyn}$ and $\epsilon_{c1,dyn}$, the concrete compressive strain rate imposed in the tests, $\dot{\epsilon}_c = 1.67 \cdot 10^{-2} \text{ s}^{-1}$, is factored into the corresponding DIF equations. The strain rates $\dot{\epsilon}_{s,i}$ in the different tensile and compression reinforcing steel layers $A_{s,i}$ ($i = 1$ to 4) are then deduced from equation (E.1), based on the assumption of proportionality between curvature χ and curvature velocity $\dot{\chi}$ (Figure E.3). It should be noted that this assumption is implicitly laid down in the approach followed in the present study for the estimation of average strain rates under the influence of bending stresses.

$$\dot{\epsilon}_{s,i} = \dot{\epsilon}_c \cdot \frac{\epsilon_{s,i}}{\epsilon_c} \quad (\text{E.1})$$

Figure E.3 Assumed strain and strain rate distribution in the intermediate cross-section of the studied column

Based on the modified stress-strain relationships, the M-N interaction diagram corresponding to the dynamic load exposure of the column is subsequently derived and shown in Figure E.2 (red curve). It is shown that the difference to the static diagram (blue) is considerable, especially in the compression-controlled failure zone what can be attributed to the relatively large DIF for the concrete compressive strength (in relation to the comparatively lower DIF for the reinforcing steel yield strength). In that context it should be highlighted that the relatively low static value of the concrete compressive strength, of $f_{cm} = 25.2 \text{ N/mm}^2$ contributes to a particularly high dynamic increase, as explained in section 3.2.2.

E.4 Comparison

Figure E.4 suggests that the herein followed analytical approach can reasonably well predict the ultimate-resistance of the cross-section. It is shown that the experimentally deduced capacities (dots) obtained for the concentric ($e = 0$) and the eccentric ($e = 0.049$ m) load application under both quasi-static and dynamic conditions are close to the corresponding M-N interaction diagrams (continuous lines). Moreover, Figure E.4 compares these diagrams with those deduced by Soroushian and Obaseki [2] (dashed lines) who integrated strain-rate sensitive material models for concrete in compression and reinforcing steel into a fiber model for RC cross-sections. The parabolic model for concrete in compression takes account of the confinement of the cross-section due to the transverse reinforcement. The properties of the reinforcing steel were represented by an empirically derived model providing similar DIF's for $f_{y,dyn}$ and $f_{u,dyn}$ as the model used in the present study [3]. However, unlike considered here, the model deduced in [2] suggests an increase of the yield strain ϵ_{sy} beyond $f_{y,dyn}/E_s$, and an increase of the ultimate strain ϵ_{su} . Despite these and other differences to the model assumptions laid down in the present study, the M-N curves in both studies are in reasonably good agreement (Figure E.4).

Figure E.4 Comparison of M-N interaction diagrams obtained (continuous lines) to tests results from Scott et al. [1] (dots) and analytical results from Soroushian and Obaseki [2] (dashed lines). Blue: Static material parameters; Red: Dynamic material parameters.

E.5 Final observations

The previous comparisons corroborate the adequacy of the herein followed simplified approach for the estimation of dynamic material properties. It is shown that this approach entails sufficiently accurate predictions of the structural resistance of RC members, even under more complex stress states, such as those induced by M-N interaction in columns.

E.6 References

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Annex F Preliminary FE study

F.1 Introduction

During the first stay at TU Delft, a numerical model has been developed and validated. In the further continuation of the work, the model has been adapted to the geometry, the material characteristics and the loading conditions of each of the beams selected for purpose of the comparative study envisaged (section 5.5).

However, the first analysis trials revealed significant plastic strain concentrations in the tensile reinforcing steel cross sections situated at the locations of the main bending cracks close to the mid-span cross section of the beams. From the physical point of view, such strain concentrations seem to be generally reasonable, since cracking induces a tensile stress transfer from the concrete to the reinforcement with a corresponding increase in tensile strain confined to a relatively small length of the bar. The point is, however, that in case the concentrated strain would approach the ultimate strain, the beam runs the risk to fail in a brittle manner by premature fracture of the tensile reinforcement before significant compressive stresses could develop and induce crushing of the concrete. Consequently, although the critical cross section would be ductile, a premature tensile failure of the reinforcing steel would prevail the plastic hinge from undergoing large rotations and hence, from dissipating significant amount of energy during the explosion event. Experimental evidence for such an undesired, brittle failure mode related to local strain concentrations, is given in [1], where the static load-carrying capacity of one-way slabs containing low-ductility reinforcement has been explored (Figure F.1).

Figure F.1 Curvature diagram prior to collapse of a one-way concrete slab reinforce with low-ductility steel [1]

For the present study, it was of concern to investigate the influence of certain modeling parameters on the mentioned strain concentrations and to evaluate their impact on the global behavior of the beams and their failure modes. With respect to the latter aspect, it had to be judged whether it is generally reasonable to establish the failure criteria for the analyzed beams based on ultimate strain of the materials. The investigated model parameters are the slope of the plastic branch of the reinforcing steel

stress-strain diagram, the bond condition between reinforcing steel and the surrounding concrete and the mesh size of the finite element model.

F.2 Overview

For purpose of the preliminary analysis, the geometry and loading conditions of beam n° 11 (Table 5.8) are modeled. The assumed material properties (based on design values) are given in Table F.1.

Table F.1: Material parameters of an example beam considered in the preliminary analysis

E_c [N/mm ²]	f_c [N/mm ²]	f_{ct} [N/mm ²]	G_r [N/mm]	G_c [N/mm]	f_y [N/mm ²]	f_u [N/mm ²]
27264	19.2	2.56	0.069	30	500	540

A total of 4 model series are studied (Table F.2). The beams corresponding to series A1-A5 are modeled under the assumption of perfect plastic reinforcing steel behavior and perfect bond conditions to the surrounding concrete. The effect of strain hardening of the longitudinal reinforcement steel and bond-slip behavior between steel and concrete on the detected steel strain concentrations is being investigated in model series B and C, respectively. Finally, series D is dedicated to the influence of mesh-size on the results, with particular emphasis on the material response in terms of material strain.

Table F.2: Summary of parameter study carried out (Example beam)

Model Series	Strain hardening	Bond condition	Mesh size	$P_{EX,peak}$ [kN/m]
A1-A5	No	Perfect	fine	25 - 29
B	Yes	Perfect	fine	29
C	Yes	Bond-slip	fine	29
D1-D3	Yes	Perfect	variable (normal-extra fine)	29

As in the simplified dynamic analysis (section 5.2), a symmetrical, triangular shaped pressure pulse of duration $t_p = 0.2$ s is applied to the beams in model series A to D. The corresponding peak pressures are varied between $p_{EX,peak} = 25$ to 29 kN/m in series A (models A1 to A5), whereas in series B, C and D the applied peak-pressure is kept constant to 29 kN/m.

F.3 Results

F.3.1 Series A

Figure F.2 reflects the flexural response of beam models A1 to A5, where perfect plastic reinforcing steel behavior and perfect bond conditions to the surrounding concrete were assumed. The left part depicts the beam's mid-span deflection due to the explosion pressure, δ_{dyn} , as a function of time, whereas the latter shows the evolution of the corresponding bending moment, $M_{E,dyn}$ ($= M_{EX}$), obtained by stress integration over the cross section height, as a function of δ_{dyn} . As could be expected, the maximum deflection caused by the explosion, $\delta_{dyn,max}$, as well as the corresponding point in time, t_{max} , rises with increasing peak-value of the applied explosion pressure pulse, $p_{EX,peak}$. Figure F.2 (left) shows that the maximum deflection of $\delta_{dyn,max} = 0.031$ m (at $t_{max} \approx 0.11$ s), corresponding to a peak-pressure of $p_{EX,peak} = 25$ kN/m (Model A1), increases to $\delta_{dyn,max} = 0.048$ m (at $t_{max} \approx 0.13$ s) under $p_{EX,peak} = 29$ kN/m (Model A5). In accordance with this observation, Figure F.2 (right) illustrates that the beam's displacement ductility

demand increases from $\mu_{\delta, \text{dyn}} = 0.99$, under $p_{\text{EX, peak}} = 25 \text{ kN/m}$ to $\mu_{\delta, \text{dyn}} = 1.56$, if $p_{\text{EX, peak}} = 29 \text{ kN/m}$. Moreover, Figure F.2 illustrates how increasing displacement ductility contributes to energy dissipation reflecting in a reduction of the dynamic load-factor DLF_M (5.2).

Figure F.2 Series A results: Mid-span deflection due to the explosion as a function of time and different peak-loadings (left); Mid-span bending moment – deflection curves for different peak-loadings (right)

In the following, a closer look will be taken to the material response. Figure F.3 (left) shows the evolution of the tensile strain in the maximum curvature cross-section (in advance referred to as critical cross section) of beams A1 to A5, normally coincident with the location of a large bending crack. It can be observed that once the yield strength of the reinforcing steel is being reached at $\varepsilon_{\text{sy}} = 0.0025$, the strain rate experiences a significant increase. Naturally, the higher the applied explosion pressure, the higher is the strain, varying between peak-values $\varepsilon_{\text{s, max}} = 0.0025$ ($p_{\text{EX, peak}} = 25 \text{ kN/m}$) and $\varepsilon_{\text{s, max}} = 0.04$ ($p_{\text{EX, peak}} = 29 \text{ kN/m}$). After reaching the maximum flexural response, the reinforcing steel strain slightly decreases according to the definition of the elastic unloading branch of the stress-strain diagram.

Figure F.3 (right) shows the time dependent evolution of the concrete strain ε_c on top of the critical cross section of the series A beams. It may be observed that the transition from the elastic to the plastic range strain rates, at about $t_y = 0.08$ to 0.1 s , depending on the $p_{\text{EX, peak}}$ applied, is not as abrupt as seen before for the reinforcing steel strain (Figure F.3, left). Once the maximum response (δ_{max}) is reached, the concrete strain decreases according to the unloading branch of the corresponding constitutive material law.

Figure F.3 Series A results: Reinforcing steel strain at maximum curvature cross section for different peak-loadings (left); Concrete compressive strain at maximum curvature cross section for different peak-loadings (right)

Figure F.4 depicts, at different points in time, the curvature distribution between support and mid-span of beam model A5 ($p_{EX,peak} = 29 \text{ kN/m}$), determined from the compressive strain on top of the concrete cross section, ε_c , and the tensile reinforcing steel fiber, ε_s , according to (F.1).

$$\chi(x) = \frac{|\varepsilon_c(x)| + \varepsilon_s(x)}{d} \quad (F.1)$$

$$\chi_y(x) = \frac{4}{L^2} \cdot (x \cdot L - x^2) \cdot \kappa_{y,mid-span} \quad (F.2)$$

Additionally, the theoretical yield curvature distribution, $\chi_y(x)$, according to (F.2), based on elastic beam-theory, is represented by the black, dashed line, where $\chi_{y,mid-span}$ is the corresponding value at mid-span, obtained from the numerical analysis. Indicative of an adequate structural behavior in the elastic range, $\chi_y(x)$ matches reasonably well the shape of $\chi(x)$ at $t_y = 0.078 \text{ s}$, where yielding of the tensile reinforcing steel is being initiated. Of more interest, however, are the curvature peaks for $t > t_y$, which arise as a consequence of plastic strain concentrations in the tensile reinforcement at the dominant bending cracks, visualized by the plot of the first principal strain distribution at maximum response ($t = t_{max}$).

Figure F.4 Crack pattern at maximum response ($t=t_{max}$) and curvature distribution at different points in time along beam A5 as a function of the distance from the support.

As a measure for the plastic strain concentrations, the ratio k_p (F.3) between the maximum plastic curvature at the critical cross section $\chi_{p,max}$ and the average plastic curvature, $\chi_{p,av}$, along the length of the plasticity region l_y is introduced. This can be observed in Figure F.5, which again represents the curvature distribution $\chi(x)$ at $t = t_{max}$ along with the theoretical yield curvature distribution $\chi_y(x)$.

$$k_p = \frac{\chi_{p,max}}{\chi_{p,av}} \quad (F.3)$$

Figure F.5 Theoretical elastic curvature distribution and curvature distribution at maximum response ($t=t_{\max}$) along beam A5 as a function of the distance from the support.

The average plastic curvature $\chi_{p,av}$ is defined as the height of a rectangle with length l_y and an area given by the integral enclosed by the curvature distribution $\chi_{\max}(x)$ at $t = t_{\max}$, and the elastic curvature distribution $\chi_y(x)$ at $t = t_y$. Note that the mentioned area represents the plastic hinge rotation, θ_p , given as:

$$\theta_p = \int [\chi_{\max}(x) - \chi_y(x)] dx = \chi_{p,av} \cdot l_y \quad (F.4)$$

In case of beam model A5 the length of the plasticity region is $l_y = 0.71$ m and the plastic hinge rotation $\theta_p = 0.012$ rad (Table F.3). Thus, according to (F.4) an average plastic curvature of $\chi_{pl,av} \approx 0.017$ m⁻¹ is determined which is about $k_p = 8.2$ times smaller than the maximum plastic curvature at the critical section, $\chi_{pl,max} \approx 0.14$ m⁻¹. It must be stated that factor k_p should be used for comparative purpose only. This is because the rectangular shape of the plastic curvature distribution has been chosen for purpose of a simplified mathematical treatment only but in fact lacks any physical background.

Table F.3: Length of plasticity region l_y , plastic rotation θ_p , average plastic curvature $\chi_{pl,av}$, maximum plastic curvature $\chi_{pl,max}$ and plastic strain concentration factor k_p for beam models A5 and B

Model	Strain hardening	l_y [m]	θ_p [rad]	$\chi_{p,av}$ [m ⁻¹]	$\chi_{p,max}$ [m ⁻¹]	k_p
A5	No	0.71	0.012	0.017	0.140	8.2
B	Yes	0.84	0.011	0.013	0.040	3.1

F.3.2 Series B

In beam model B (Table F.2) strain hardening has been introduced in the constitutive stress-strain relationship of the reinforcing steel. Figure F.6 shows the obtained mid-span deflection-time curve in comparison to the response of beam A5 (perfect plastic behavior), submitted to the same explosion pressure ($p_{EX,peak} = 29$ kN/m). It can be observed that the difference between both solutions is very small, as was to be expected. However, when analyzing the response at the material level, significant differences are obtained as depicted in Figure F.7. Precisely, it is shown that both the maximum tensile strain in the reinforcing steel (left) and the maximum compressive strain in the concrete (right) reduce considerably when strain hardening is introduced in the reinforcing steel stress strain law. While the former is being reduced from about 4% to about 1.3%, the latter diminishes from about 6.1‰ to 3.8‰. It can further be observed that the increase in strain rate upon entering the plastic response stage is less pronounced in case of model B, where strain hardening has been factored in.

Figure F.6 Mid-span deflection as a function of time, for models A5 (perfect plastic behaviour of reinforcing steel) and B (including strain hardening)

Figure F.7 Reinforcing steel strain (left) and concrete compressive strain (right) at maximum curvature cross section for models A5 (perfect plastic behaviour of reinforcing steel) and B (including strain hardening).

In Figure F.8 the plastic curvature distribution at maximum response of beam model B is shown. Again plastic rotation θ_{pl} , average curvature $\chi_{p,av}$ and strain concentration factor k_p are determined as described before. The results, compared in Table X to those of model A5, show that the plastic hinge rotation θ_{pl} remains almost unaffected when introducing strain hardening in the constitutive model of the reinforcing steel. On the contrary, the strain concentrations considerably decrease, as reflected by a comparatively smaller strain concentration factor of $k_p = 3.1$. Figure F.8 shows that the curvature distribution along the plasticity region l_y is more continuous than in model A5 (Figure F.5), and characterized by comparatively smaller peak strains at the crack locations. This comparison clearly affirms that, as long as the ultimate strain of the reinforcing steel is higher than the localized strain peaks (what is the case in the present example), the global structural behavior of the beams is not influenced by the strain concentrations in the cross-sections affected by mayor bending cracks.

Figure F.8 Theoretical elastic curvature distribution and curvature distribution at maximum response ($t=t_{\max}$) along beam model B as a function of the distance from the support.

F.3.3 Series C

The purpose of model series C was to study the impact of the bond-slip mechanism between the longitudinal reinforcement and the surrounding concrete on the structural behavior of the beams, especially at the material level. Therefore, DIANA [2] offers the use of so-called bond-slip reinforcements, which are defined similar to standard embedded reinforcements. The main difference to the latter is the connection, by an interface element (with zero thickness), to the mother element (in the present case the plane stress elements) in which they are located. The interface element characterizes the mechanical behavior of the slip zone between concrete and reinforcement. The linear relationship between the normal stresses, σ_n , and the normal relative displacement, $\Delta\delta_n$, is given by (F.5), whereas the stresses in the tangential direction (bond stresses), τ_t , are defined as a nonlinear function of the corresponding relative slip, $\Delta\delta_t$ (F.6).

$$\sigma_n = k_n \cdot \Delta\delta_n \quad (\text{F.5})$$

$$\tau_t = f(\Delta\delta_t) \quad (\text{F.6})$$

The linear relationship (F.5) assumed in the normal direction requires the assumption of stiffness constant k_n which, based on indications in [3] will be assumed to $k_n = 20000 \text{ N/mm}^3$. In the tangential direction, the model deduced by Dörr [4], implemented in DIANA, will be used to express the non-linear relation between bond stress and slip. The bond-slip model is defined by equations (F.7) and (F.8). It is described by a cubic function up to a certain slip limit $\Delta\delta_{t0}$ beyond which a constant bond stress of $\tau_t = 1.9$ times the concrete tensile stress f_{ct} is being assumed. The recommended slip limit according to Dörr [4] is $\Delta\delta_{t0} = 0.06 \text{ mm}$ what will be adopted herein. Unloading and reloading of the interface behavior is modeled by a secant approach.

$$\tau_t = f_{ct} \left(5 \frac{\Delta\delta_t}{\Delta\delta_{t0}} - 4.5 \left(\frac{\Delta\delta_t}{\Delta\delta_{t0}} \right)^2 + 1.4 \left(\frac{\Delta\delta_t}{\Delta\delta_{t0}} \right)^3 \right) \quad \text{if } 0 \leq \Delta\delta_t \leq \Delta\delta_{t0} \quad (\text{F.7})$$

$$\tau_t = 1.9 f_{ct} \quad \text{if } \Delta\delta_t > \Delta\delta_{t0} \quad (\text{F.8})$$

For the example beam with a total cross-sectional area of $A_{s,\text{tot}} = 683 \text{ mm}^2$, three different bar diameters ($\phi = 12, 16$ and 20 mm) have been assumed. The corresponding number of (fictitious) rebars n_s , their perimeters P_s and the total perimeter $P_{s,\text{tot}} = P_s \cdot n_s$ are given in Table F.4.

Table F.4: Geometrical description of bond-slip rebars

ϕ [mm]	$A_{s,tot}$ [mm ²]	n_s	P_s [mm]	$P_{s,tot}$ [mm]
12	683	6.0	38	228
16	683	3.4	50	171
20	683	2.2	63	137

The first trials under use of the bond slip reinforcements revealed severe numerical problems at the symmetry plane located at the beams mid-span cross section. In order to avoid those problems, it was decided to model the full beam geometry. In a next step, the correct implementation and functioning of the bond slip reinforcement was verified. For this purpose, the equilibrium condition of an infinitesimally small reinforcement element (Figure F.9), given by (F.9), was checked. For sake of simplicity, this was done under assumption of static loading conditions and linear-elastic concrete behavior. The result of this exercise is shown in Figure F.10. It compares the left and the right part of the equilibrium condition (F.9), i.e., the increment in the tensile force (left part of (F.9)) and the bond force along length dx of the reinforcement element (right part of (F.9)), which in the present case is equal to the length of the finite elements defined in the numerical model. As was to be expected, the bond-stress (τ) distribution along the beam is antisymmetric with a maximum at the supports. Towards the mid-span τ diminishes and entirely disappears at the location where the reinforcing steel stress (σ_s) reaches the defined yield strength. From Figure F.10 it can be concluded that the bond and tensile force along the reinforcing bar are in equilibrium suggesting hence the correct implementation and functioning of the bond-slip reinforcements in the numerical model.

$$\frac{d\sigma_s}{dx} \cdot A_s = \tau(x) \cdot P_s \quad (F.9)$$

Figure F.9 Stress distribution in an infinitesimal reinforcement element [5].

Figure F.10 Right and left part of force equilibrium (F.9) along the beam.

After the correct implementation of the bond-slip reinforcements, the example beam was analyzed under the same dynamic loading conditions as before model B (perfect bond), as specified in Table F.2 (series C). Figure F.11 compares the time-dependent mid-span deflection under the perfect bond assumption to the results obtained under use of the bond-slip model described before, assuming rebar diameters of $\phi = 12, 16$ and 20 mm, respectively. It can be seen that the global beam response to the dynamic load is practically unaffected by the bond condition between the reinforcement and the concrete.

Figure F.11 Comparison of mid-span deflection-time curves under perfect bond assumption and under use of bond-slip relation according to Dörr [4], assuming different bar diameters.

However, when analyzing the response at the material level some differences can be detected. This can be observed in Figure F.12 which shows the evolution in time of the strain in the maximum curvature cross section located at mid-span, respectively, in the tensile reinforcement steel (left) and in the concrete (right). It is shown that the steel strain decreases when bond-slip is being considered. As could be expected, the decrease is higher the smaller the total rebar perimeter $P_{s,tot}$ (Table F.4), i.e. the smaller the bond stress between rebars and concrete. On the contrary, in accordance with the equilibrium condition of the internal cross section forces, the concrete strain in the critical cross-section increases as the bond stress diminishes. Nevertheless, the most relevant conclusion, which can be drawn from Figure F.12, is the fact that the results are not altered in a significant manner upon introducing a bond-slip condition into the numerical model. The strain concentrations in the reinforcing steel, detected at the mayor bending cracks, are only scarcely reduced when allowing for slip between concrete and steel.

Since moreover, the need for modeling the full beam implies significantly higher computational costs, it was decided to proceed with the perfect bond condition in the further development of the work.

Figure F.12 Comparison of time dependant evaluation of tensile reinforcing steel strain (left) and concrete compressive strain (right) at maximum curvature cross section under perfect bond assumption and under use of bond-slip relation according to Dörr [4], assuming different bar diameters.

F.3.4 Series D

In series D, the influence of the mesh size on the behavior of the example beam has been analyzed. The mesh is characterized by a declining size of the quadrilateral plane stress elements, via a transition zone, from $e=h/8$ at the beams free end, to a smaller size in the zone around the mid-span cross-section. Three different mesh sizes of the refined mid-span zone have been analyzed here, named “coarse”, “intermediate” and “fine” mesh, with element sizes of $e=h/16$, $e=h/32$, and $e=h/64$, respectively, shown in Figure F.13.

Figure F.13. “Coarse”(upper), “intermediate”(middle) and “fine”(lower) mesh size analyzed in model series D.

The mid-span deflection-time curves obtained for the example beam are compared in Figure F.14. It is shown that, compared to the coarse mesh size, the use of the intermediate mesh size entails an increase of the maximum deflection of the order of 4%. The use of the fine mesh does not produce any further significant increase of the deflection, i.e. the results clearly converge upon mesh-size refinement. The obtained results suggest choosing the intermediate mesh size for purpose of the further calculations.

Figure F.14 Mid-span deflection-time curves obtained with “coarse”, “intermediate” and “fine” mesh size.

In contrast to the deflections, the response at the material level does not seem to converge upon refining the mesh size. The strains in both the tensile reinforcing steel and the concrete fiber on top of the maximum curvature cross section increase moderately as the mesh size is being refined, as can be observed in Figure F.15. Hence, the before-mentioned strain concentrations do have, at least to some extent, a numerical background. In any case, no change in the beam’s failure mode is to be expected when employing finer mesh sizes for purpose of the numerical analysis of the example beam, i.e., the element fails in a ductile manner, after the ultimate concrete compressive strain is being reached in the critical cross-section under simultaneous yielding of the tensile reinforcement. Nevertheless, the mesh-size sensitivity is too significant to establish a failure criterion for the beam based on material strain. Moreover, a further mesh-size reduction is not considered as an option due to sharply increasing computation time.

Figure F.15 Tensile reinforcing steel strain (left) and concrete compressive strain (right) at the maximum curvature cross section obtained with “coarse”, “intermediate” and “fine” mesh size.

F.4 Final observations

The conclusions from the preliminary numerical analysis are the following:

- The numerically analyzed beams are affected by significant strain concentrations in the tensile reinforcing steel, which arise at the mayor bending cracks.
- Although not entirely eliminated, the mentioned strain concentrations are significantly reduced when introducing strain hardening in the constitutive model of the reinforcing steel, whereas

the global structural behavior of the beams remains almost unaffected by this measure. Moreover, considering strain hardening in the constitutive model of the reinforcing steel contributes to numerical stability.

- Considering a bond-slip relationship between the reinforcing steel and the concrete has only a small impact on the magnitude of the material strain. Since, moreover, the introduction of such a relationship into the numerical model entailed the need for representing the full beam geometry, with a corresponding important increase in computational cost, it was decided to proceed with a perfect bond assumption in the further development of the work.
- In contrary to the global structural response, the material strain does not converge upon refining mesh-size. The strain in both the tensile reinforcing steel and the compressed concrete fiber at the maximum curvature cross-section increase as the mesh size is being refined. Hence, the strain concentrations are influenced, at least to some extent, by numerical circumstances what precludes the establishment of a failure criterion based on ultimate material strain.
- The strain concentrations in the reinforcing steel do not entail a potential to change the (ductile) failure mode of the beam, characterized by concrete crushing close to mid-span, after a significant amount of energy, liberated by the explosion, could be dissipated by plastic structural deformations. However, their presence rules out the possibility to draw on the numerical model to verify the usefulness of relationship (5.10), which provides the deformation capacity of the studied RC members as a function of the curvature ductility ratio at the mid-span cross-section. This verification is addressed in Annex C.

F.5 References

- [1] Gilbert RI, Smith ST. Strain localization and its impact on the ductility of reinforced concrete slabs containing welded wire reinforcement. *Advances in Structural Engineering*. 2006;9:117-27.
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Annex G Approach for dynamic material properties in the numerical study

G.1 Introduction

The numerical analysis (NA) of the RC beams exposed to blast loads (section 5.5) required the definition of dynamic material properties. Since the FE-code employed [1] did not include any strain-rate dependent constitutive material models, the dynamic increase was factored into the numerical model in a similar manner as in the simplified analysis (SA) (section 5.1.3, 5.5.3): by applying DIFs to the principal mechanical material properties of concrete and steel to modify the uniaxial stress-strain laws. The DIFs were obtained for the constant average strain rates for the concrete elements under compressive stress and the steel elements under tensile stress, deduced from the NA. After a description of the procedure adopted in the following sub-section, its application will be illustrated by means of an example.

G.2 Procedure

The approach for the determination of dynamic material properties laid down in the numerical analysis is iterative. It comprises the following steps:

1. Definition of the static material properties of concrete and reinforcing steel and association to the constitutive models used in the FE analysis, described in section 5.5.3.
2. Sequential NA (static and dynamic) of the member according to section 5.5.5.
3. Determination of the average strain rates for the concrete element under maximum compressive stress and the longitudinal reinforcing steel element under maximum tensile stress, in advance termed “most loaded” elements.
4. Determination of the DIF for the concrete compressive strength and the reinforcing steel yield- and tensile strength using the DIF-models adopted [2, 3] in conjunction with the deduced average strain rates (step 3).
5. Updating of the uniaxial stress-strain curves for concrete under compression stress and reinforcing steel under tensile stress by means of the DIF determined in step 4.
6. Repetition of steps 2 to 5 until the updated material properties do not deviate from those obtained in the previous iteration step by more than the established tolerance (1%).

G.3 Example

Figure G.1 shows the time-dependent evolution of the strain in the most loaded elements of, respectively, concrete in compression (left) and reinforcing steel in tension (right), obtained in the NA of a RC beam (section 5.5). According to step n° 2 of the above-defined procedure, the analysis was carried out under use of the initially defined static material properties, summarized in Table G.1 (1st iteration). As shown in Figure G.1, a constant, average strain rate $(d\varepsilon/dt)_{av}$ can be deduced (step n° 3) by linear regression analysis on the pairs (ε, t) within the time range $0 < t < t_c$, in case of concrete in compression (left), and $0 < t < t_t$ for the reinforcing steel under the influence of tensile forces (right). The

limit $t_c = 0.082$ s describes the point in time when, in the considered element, the maximum compressive stress reaches the concrete compressive strength, $\sigma_{c,max} = f_c$. This can be well observed in Figure G.2 (left), which shows the time-dependent evolution of σ_c in the mentioned element. The limit $t_y = 0.075$ s corresponds to the time needed to attain the yield strength of the reinforcing steel, f_{sy} , as highlighted in Figure G.1 and Figure G.2 (right). The example shows that the assumption of a mean strain (or stress) rate seems to be reasonable within the specified time ranges, limited by t_c and t_y , respectively.

In step n^o 4 of the above described procedure, the obtained average strain rate for concrete under compression, $(d\epsilon_c/dt)_{av} = -2.2 \cdot 10^{-2} \text{ s}^{-1}$, is employed to determine the dynamic increase factor $DIF_{fc} = 1.22$ for the concrete compressive strength, according to [2]. In an equal manner, average strain rate $(d\epsilon_s/dt)_{av} = 4.4 \cdot 10^{-2} \text{ s}^{-1}$ is used to determine factors $DIF_{fy} = 1.13$ and $DIF_{ft} = 1.04$ from the formulations suggested in [3] for the dynamic increase of the reinforcing steel yield and tensile strength, respectively. The results are summarized in Table G.1 (1st iteration step).

Subsequently, in step n^o 5 of the outlined procedure, the deduced DIF are employed to update the material properties f_c , f_y and f_t (Table G.1, 2nd iteration step), and consequently, the uniaxial stress-strain laws for concrete in compression and steel in tension. As proceeded in the SA of the beams, the reinforcing steel yield strain ϵ_y is implicitly updated by applying a DIF to the yield strength f_y , whereas both the elastic modulus E_s and the ultimate strain ϵ_u are kept constant. Regarding concrete, it should be noted that, on the contrary to the SA, the compressive strain at peak-stress (ϵ_{c1}) is not being explicitly updated since the uniaxial stress-strain relationship for concrete under compression laid down in the NA (Figure 5.32) is entirely defined in terms of concrete modulus E_c and compressive strength f_c only. For sake of consistency to the SA (section 5.3.2), the modulus E_c will be assumed as under static loading conditions.

With the updated stress-strain relationships, the sequential NA of the beam is then being repeated (step n^o 6) and the average strain-rates in the most-loaded elements are deduced as described before (Figure G.3, Figure G.4). In Table G.1 it can be observed that those average strain rates (2nd iteration) do not differ substantially from those obtained in the first iteration. Since the DIF formulations are logarithmical, and hence not sensitive to relatively small variations of the strain rates, the obtained DIF almost equal those of the previous iteration step. Consequently, the difference between the subsequently updated material properties (Table G.1, 3rd iteration) and the previous ones (2nd iteration) is almost inappreciable and falls clearly below the established tolerance of 1%. The analysis procedure has ended.

Figure G.1 Determination of constant mean strain rates in the most loaded element of, respectively, concrete in compression (left) and steel in tension (right) – 1st iteration (static material properties).

Figure G.2 Determination of constant mean stress rates in the most loaded element of, respectively, concrete in compression (left) and steel in tension (right) – 1st iteration (static material properties).

Figure G.3 Determination of constant mean strain rates in the most loaded element of, respectively, concrete in compression (left) and steel in tension (right) – 2nd iteration (updated, dynamic material properties).

Figure G.4 Determination of constant mean stress rates in the most loaded element of, respectively, concrete in compression (left) and steel in tension (right) – 2nd iteration (updated, dynamic material properties).

Table G.1 Summary of material properties, average strain rates $(d\varepsilon/dt)_{av}$, and dynamic increase factors (DIF) obtained in the iteration procedure.

Iteration	f_c	f_y	f_t	$(d\varepsilon_c/dt)_{av}$	$(d\varepsilon_s/dt)_{av}$	DIF_{fc}	DIF_{fy}	DIF_{ft}
	$\frac{N}{mm^2}$	$\frac{N}{mm^2}$	$\frac{N}{mm^2}$	s^{-1}	s^{-1}	-	-	-
1 st	28,1	560	670	$-2,2 \cdot 10^{-2}$	$4,4 \cdot 10^{-2}$	1,21	1,13	1,04
2 nd	34,1	632	698	$-2,3 \cdot 10^{-2}$	$4,8 \cdot 10^{-2}$	1,22	1,13	1,04
3 rd	34,1	633	699	-	-	-	-	-

G.4 Additional considerations

It should be remembered that the DIF formulations employed herein [2, 3] were derived from tests on uni-axially loaded concrete or steel specimen by applying a constant strain rate and measuring the dynamic increase in the material properties. Hence it seems to be a reasonable approach to base the dynamic increase of, for instance, the uni-axial concrete compressive strength on the average strain rate $(d\varepsilon_c/dt)_{av}$ within an interval limited by time t_c where this strength is being reached. It might be noted that beyond t_c , the concrete compressive strain rate experiences a significant increase as can be inferred from Figure G.1 or Figure G.3, which is considered to be meaningless for purpose of the estimation of DIF_{fc} . On the same grounds, the determination of the DIF for the reinforcing steel yield strength, DIF_{fy} , is reasonably based on the average strain rate $(d\varepsilon_\sigma/dt)_{av}$ within an interval limited by time t_y , where f_y is being attained (Figure G.1 or Figure G.3). Regarding the tensile strength f_t , the noticeable increase in the strain rate beyond t_y might have some influence on DIF_{ft} although it is assumed that the dynamic material behavior in the elastic range, prior to t_y , constitutes the prevailing influence. In that context it should be kept in mind that the formulation for DIF_{ft} (3.44, 3.46) is a function of the yield strength f_y what seems to justify the use of the yield point as a reference for the derivation of a corresponding average strain rate, as already stressed in section 5.3.2.

The average-strain rates employed for the determination of the DIF correspond to the most-loaded concrete (in compression) and steel (in tension) elements of the FE model (see step n° 3 of the above-defined procedure). In the example shown before, the average strain rate for the most loaded concrete element in compression, $(d\varepsilon_c/dt)_{av} = -2,3 \cdot 10^{-2} s^{-1}$, was obtained from a regression of the ε_c - t plot within the interval $0 < t < t_c$ (Figure G.3, left). At $t = t_c = 0.1$ s the compressive strength f_c is obtained in the mentioned element under a corresponding strain of $\varepsilon_{c1} = -2,1 \cdot 10^{-3}$. Since the beam has not yet reached its maximum deflection, the concrete stress in the adjacent elements will likewise reach f_c a few instants later. This is evident and can be observed in Figure G.5 (left), which plots strain ε_c versus time in all the concrete elements situated in the upper row of the FE-model mesh (Figure 5.31). At $t = t_{max} = 0.125$ s, when the beam undergoes its maximum flexural response, the “last” element reaches f_c when $\varepsilon_c = \varepsilon_{c1} = -2,1 \cdot 10^{-3}$ (Figure G.5, left). The corresponding average strain-rate $(d\varepsilon_c/dt)_{av}$ deduced by regression on the corresponding ε_c - t plot within the interval $0 < t < 0.125$ s is naturally smaller than found for the most-loaded element. However, the difference is too small to cause any significant changes in the resulting DIF. The fact that the DIF equations are logarithmical assures that the average strain-rate deduced from the most-loaded concrete element provides a sufficiently exact estimation of the dynamic increase of the compressive strength even for the surrounding, less-loaded finite elements. The same conclusion is valid for the strength increase of the reinforcing steel under the influence of tensile forces (Figure G.5, right).

Figure G.5 Strain versus time in the concrete elements corresponding to the upper row of the FE-model mesh (left) and in the tensile reinforcing steel elements (right)

G.5 References

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- [3] Malvar LJ, Crawford JE. Dynamic Increase Factors for Steel Reinforcing Bars. Twenty-Eighth DDESB Seminar Orlando, FL;1998.

Annex H Database for the derivation of a consequence model

H.1 Format for data compilation

Datos relativos edificio			
Caso	15		
Tipo de uso	vivienda	Categoría de uso	A1
Dirección	C/Consuelo de los Aflig	Ciudad	Sevilla
País	España	Año de construcción	
Tipo de estructura	Se reconocen elementos de hormigón armado en la foto		
Superficie por planta (m2)		Superficie total (m2)	
Nº de plantas	3	Nº de viviendas	16
Datos relativos al incidente			
Fecha del incidente	14/08/2003	Tipo del accidente	Explosión
Causa del incidente	Explosión provocada por un vecino (mezcla de butano de una bombona y gasolina)		
Categoría de la causa	Intervención premedita	Tipo de sustancia / vehículo	Butano (bombona)
Descripción del incidente y motivos del fallo	<p>"La explosión en la segunda planta ha provocado el derrumbe parcial del edificio..."</p> <p>"...el segundo y tercer piso, cuyos cascotes descansan sobre el primero."</p> <p>"Incendio posterior...todo el bloque ha sido afectado por las llamas"</p>		
Categoría de gravedad	3	Colapso total ?	no
Superficie afectada (m2)		Superficie colapsada (m2)	
Plantas afectadas	4	Plantas colapsadas	2
Viviendas afectadas		Viviendas colapsadas	
Ocupación del edificio	35	Ocupación de la superficie de colapso	3
Datos relativos a las consecuencias			
Nº de víctimas mortales	3	Nº de víctimas mort. (colapso)	2
Nº de heridos graves	2	Nº de heridos grav. (colapso)	0
Nº de heridos leves	23	Nº de heridos leves (colapso)	
Nº de heridos totales	32	Nº de heridos tot. (colapso)	
Nº de desaparecidos	0		
Consecuencias económicas directas	<p>"El inmueble tendrá que ser demolido parcialmente".</p> <p>"Los edificios colindantes también se han visto afectados."</p>		
Consecuencias económicas indirectas	<p>Unas 20 familias han tenido que ser desalojadas y trasladadas a un centro cívico, donde a estas horas permanecen 70 personas. Según las primeras estimaciones, 41 vecinos de 11 familias se han quedado sin alojamiento.</p>		
Información adicional			
Fuente de la información	http://www.elpais.es/articulo/espana/explosion/provocada/vecino/causa/muertos/32/h		
Otras fuentes			
Comentario	<p>Un muerto es causa directa de la explosión en el segundo piso, los otros dos, son del primer piso, y murieron aplastados por los cascotes de los pisos superiores;</p> <p>23 heridos habían dados de alta inmediatamente;</p> <p>Los 2 heridos graves sufrieron quemaduras.</p> <p>Se desconoce cuantas personas estaban presentes en la superficie colapsada;</p>		
Utilidad	si		

Figure H.1 Database format

H.2 Original database

Table H.1: Selected parameters for 107 potentially relevant incidents

Case ID	A [m ²]	n _s	n _{hu}	A _{col} [m ²]	n _{s,col}	n _{hu,col}	Ocu	Ocu _{col}	N _{col}
1	240	3		120	2			6	0
3		4			2				4
7		4			3				3
8		3			4				3
9	600	3	8	480	4		14	14	11
11		5			3		21	7	2
14		4							0
15		3	16		2				2
16		3			4		20	20	4
19		2			3		5	5	2
20	120	1		60	1		2	2	0
21		3	24						2
22	500	4		175	2		18	6	0
23		1			2		1	1	0
26		4	10		5	10			17
30		1			1		4	4	2
31	400	3		300	3		3	3	1
32		2			3		1	1	1
33		1			2		1	1	1
39									0
41					3				16
42									7
46		4			2		1		0
47		2			4				9
50		3			4				3
54									5
57		1			4				7
58	200	2		200	3				5
59									2
60		1			1		1	1	0
62	54	0		54	1	1	2	2	1
63		1			2		2	2	0
64		3							0
66	320	2		160	1		20	10	0
71		1			2		5	5	3
74	300	3		200	2				0
76	240	2		120	3		11	11	1
77	660	22	22	480	10.5	16			3
80									10
81		2	6		2		15	10	0
83		5			2				0
84		1			2		2	2	0
85		5			6		36	36	1
86		2					1	1	0
87				80	0.5	1			0
89		1			2		2	2	0
91		3			2				0
92		3	12		2	6			5

Case ID	A [m ²]	n _s	n _{hu}	A _{col} [m ²]	n _{s,col}	n _{hu,col}	Ocu	Ocu _{col}	N _{col}
93	1864	1		1864	2		42	42	27
94					0.5	1		1	0
96		3	8		4		11	11	0
98	300	1	6	80	0.5	1		1	0
100		1			1		5	5	0
101		4			2		32	16	0
103		6			3		58	25	4
104					1		50	50	4
105							3	3	2
107					1			4	0
108		1			2		7	7	4
109		2	6		2			8	1
111							1	1	1
112		4							1
113						4	5	5	2
115		12			0.5	1		1	0
116		2			3		1	1	1
118		2						2	0
119		3			4		11	11	10
120		5			1	1		2	1
121		1			0.5		3	3	0
124		2	6		0.5	1		1	0
125							6		0
127		3			1	1	4	4	0
129		8			1				3
133		2			3		1	1	0
136		5							0
137		8			1	1			0
138		3				1			0
139									1
140		5	16		3	8			2
141		4		40	1	1			2
143	240	8		90	3				3
144		4			2				2
145		3			4				3
146		8			9		88	88	7
147		2	11						2
148	120	3	6	40	1				0
149	785	4	10	785	5		20	20	7
150		4				1		1	1
152		7	18	15			41		1
161		5	15		5	15			6
165		4			2				8
169		2			1		3	1	1
172		2				1			1
181									0
184		2			3		3	3	2
193		5			0.5	0.5			0
195		3	12	150	1	3			0
196		4	16	200	1	3			1
197		2		30	1				0

Case ID	A [m ²]	n _s	n _{hu}	A _{col} [m ²]	n _{s,col}	n _{hu,col}	Ocu	Ocu _{col}	N _{col}
201		2		450	2		50	35	17
203		5			3		136	82	19
207		2				1		2	0
208		5				0.5	26		4
210		1			1			2	1
211		0			1			1	1
232		1						5	5
233		4	22		3	15	35	23	12

H.3 Data from the National Statistics Institute (INE)

Table H.2: Number of housing-units as a function of housing-unit net-room area A_{hu} and the number of storeys n_s corresponding to the building to which these units belong (INE, housing and population census 2001)

A_{hu} [m ²]	Total	$A_{hu} \leq 30$	$31 \leq A_{hu} \leq 45$	$46 \leq A_{hu} \leq 60$	$61 \leq A_{hu} \leq 75$	$76 \leq A_{hu} \leq 90$	$91 \leq A_{hu} \leq 105$	$106 \leq A_{hu} \leq 120$	$121 \leq A_{hu} \leq 150$	$151 \leq A_{hu} \leq 180$	$A_{hu} > 180$
n_s											
Total	1.418e07	55797	427630	1527792	2652889	4162623	2276807	1315680	919379	363975	481454
1	2185326	14372	70268	191741	260186	581098	421180	271936	192836	74480	107229
2	2952549	10791	66986	198001	307279	760094	546550	399897	317432	141986	203533
3	1214799	5321	40251	122506	195960	342584	183577	117793	91131	42277	73399
4	1425948	5253	56631	191090	343448	466310	189382	85028	49186	17588	22032
5	1788299	6746	70147	308975	492382	538045	210090	88002	46381	14453	13078
6	1290742	4361	42517	192942	330201	403164	171836	79074	42670	12715	11262
7	837649	2928	26794	99072	181294	268443	131752	64345	39197	12927	10897
8	1111746	2693	24462	101871	235580	360919	189786	96618	62856	20178	16783
9	261232	597	5653	22758	56107	86692	43873	21583	14291	4957	4721
≥ 10	1115736	2735	23921	98836	250452	355274	188781	91404	63399	22414	18520

Table H.3: Number of buildings in Spain comprising a certain number of housing units n_{hu} and storeys n_s (INE, housing and population census 2001)

n_{hu}	Total	1	2	3	4	5 – 9	10 – 19	20 – 39	≥ 40
n_s									
Total	8623875	6885843	623788	174345	121046	356282	302102	129658	30811
1	3457833	3258714	162505	19168	6916	8115	2281	71	63
2	3591827	3120927	353738	50327	36617	22339	5808	1990	81
3	775177	468018	97006	66513	35184	86724	18223	2699	810
4	285973	33711	9014	34369	23031	126049	51635	6798	1366
5	214849	3049	1364	3552	17040	78186	88099	21391	2168
6	121190	1221	126	364	2008	24599	64956	25171	2745
7	61847	124	22	40	203	6182	33184	19303	2789
8	61497	60	6	10	39	2973	25437	27735	5237
9	13509	10	3	2	6	540	5269	6701	978
≥ 10	40173	9	4	0	2	575	7210	17799	14574

Table H.4: Number of persons per housing unit net-room area A_{hu} (INE, housing and population census 2001)

$A_{hu} [m^2]$	Nº of persons
$A_{hu} \leq 30$	106951
$31 \leq A_{hu} \leq 45$	905815
$46 \leq A_{hu} \leq 60$	3652975
$61 \leq A_{hu} \leq 75$	7134239
$76 \leq A_{hu} \leq 90$	12020841
$91 \leq A_{hu} \leq 105$	6831506
$106 \leq A_{hu} \leq 120$	4119937
$121 \leq A_{hu} \leq 150$	2966046
$151 \leq A_{hu} \leq 180$	1210596
$A_{hu} \geq 180$	1646955
Total	40595861

H.4 Complemented database

Table H.5: Selected parameters for 107 potentially relevant incidents

Case ID	A [m ²]	n _s	n _{hu}	A _{col} [m ²]	n _{s,col}	n _{hu,col}	Ocu	Ocu _{col}	N _{col}
1	240	3		120	2		12	6	0
3	1064	4		426	2		35	14	4
7	1064	4		638	3		35	21	3
8	679	3		679	4		23	23	3
9	600	3	8	480	4		14	14	11
11	1483	5		741	3		21	7	2
14	1064	4					17		0
15	1308	3	16	340	2		35	11	2
16	679	3		679	4		20	20	4
19	269	2		269	3		5	5	2
20	120	1		60	1		2	2	0
21	1867	3	24				11		2
22	500	4		175	2		18	6	0
23	139	1		139	2		1	1	0
26	778	4	10	778	5	10	32	32	17
30	139	1		70	1		4	4	2
31	400	3		300	3		3	3	1
32	269	2		269	3		1	1	1
33	139	1		139	2		1	1	1
39	249								0
41				367	3			16	16
42	249								7
46	1064	4		426	2		1	1	0
47	359	2		359	4		14	14	9
50	679	3		679	4		23	23	3
54	249								5
57	279	1		279	4		11	11	7
58	200	2		200	3		9	9	5
59	249			249			11	11	2
60	139	1		70	1		1	1	0
62	54	0		54	1	1	2	2	1
63	139	1		139	2		2	2	0
64	679	3					150		0
66	320	2		160	1		20	10	0
71	139	1		139	2		5	5	3
74	300	3		200	2		10	7	0
76	240	2		120	3		11	11	1
77	660	22	22	480	10.5	16	22	16	3
80	249						49		10
81	569	2	6	180	2		15	10	0
83	1483	5		494	2		49	16	0
84	139	1		139	2		2	2	0
85	1483	5		1483	6		36	36	1
86	269	2					1	1	0
87	249			80	0.5	1	8	3	0
89	139	1		139	2		2	2	0
91	679	3		340	2		23	11	0

Case ID	A [m ²]	n _s	n _{hu}	A _{col} [m ²]	n _{s,col}	n _{hu,col}	Ocu	Ocu _{col}	N _{col}
92	981	3	12	491	2	6	33	16	5
93	1864	1		1864	2		42	42	27
94	249			90	0.5	1	8	1	0
96	679	3	8	679	4		11	11	0
98	300	1	6	80	0.5	1	10	1	0
100	139	1		70	1		5	5	0
101	1064	4		426	2		32	16	0
103	1896	6		813	3		58	25	4
104	249			122	1		50	50	4
105	249			249			3	3	2
107	249			122	1		8	4	0
108	139	1		139	2		7	7	4
109	569	2	6	180	2		20	8	1
111	249						1	1	1
112	1064	4					6		1
113				360		4	5	5	2
115	2273	12		87	0.5	1	76	1	0
116	269	2		269	3		1	1	1
118	269	2					2	2	0
119	679	3		679	4		11	11	10
120	482	5		80	1	1	16	2	1
121	139	1		35	0.5		3	3	0
124	569	2	6	95	0.5	1	19	1	0
125	249						6		0
127	327	3		82	1	1	4	4	0
129	2401	8		267	1		80	9	3
133	269	2		269	3		1	1	0
136	1483	5					17		0
137	788	8		88	1	1	26	3	0
138	679	3		82		1	23	3	0
139	249			249			8	8	1
140	1286	5	16	643	3	8	43	21	2
141	200	4		40	1	1	13	2	2
143	240	8		90	3		14	14	3
144	1064	4		426	2		35	14	2
145	679	3		679	4		30	30	3
146	2401	8		2401	9		88	88	7
147	1043	2	11				9		2
148	120	3	6	40	1		5	1	0
149	785	4	10	785	5		20	20	7
150	967	4		78		1	32	1	1
152	1572	7	18	15			41	1	1
161	1206	5	15	1206	5	15	40	40	6
165	1064	4		426	2		35	19	8
169	269	2		90	1		3	1	1
172	269	2		95		1	9	2	1
181									0
184	269	2		269	3		3	3	2
193	482	5		40	0.5	0.5	16	1	0
195	981	3	12	150	1	3	43	11	0
196	1245	4	16	200	1	3	36	7	1
197	90	2		30	1		3	1	0

Case ID	A [m ²]	n _s	n _{hu}	A _{col} [m ²]	n _{s,col}	n _{hu,col}	Ocu	Ocu _{col}	N _{col}
201	675	2		450	2		50	35	17
203	1483	5		741	3		136	82	19
207	269	2		95		1	9	2	0
208	1483	5		40		0.5	26	4	4
210	139	1		70	1		5	2	1
211	116	0		116	1		4	1	1
232	139	1						5	5
233	1712	4	22	1167	3	15	35	23	12