UNIVERSIDAD POLITÉCNICA DE MADRID

Escuela Técnica Superior de Ingenieros de Caminos, Canales y Puertos
Departamento de Mecánica de Medios Continuos y Teoría de Estructuras
Grupo de Hormigón Estructural

hormigón estructural

BEHAVIOUR OF TRANSVERSAL JOINTS IN PREFABRICATED TUNNEL LININGS. EXPERIMENTAL AND THEORETICAL STUDY

PhD THESIS

COSIMO IASIELLO

MsC Civil Engineer

DIRECTOR

ALEJANDRO PÉREZ CALDENTY

Civil Engineer, PhD

Madrid, 2017

UNIVERSIDAD POLITÉCNICA DE MADRID

D.15

Tribunal nombrado por el Magfco.. y Excmo. Sr. Rector de la Universidad Politécnica de Madrid, el día.....

Presidente D.

Vocal D.

Vocal D.

Secretario D.

Suplente D.

Suplente D.

Realizado el acto de defensa y lectura de la Tesis el día.....de.....de 2017 en la *E.T.S. de Ingenieros de Caminos, Canales y Puertos de la U.P.M.*

Calificación:

EL PRESIDENTE

LOS VOCALES

EL SECRETARIO

To Patricia

INDEX

FIGURES INDEX.....	IV
TABLES INDEX.....	VI
SUMMARY.....	VII
RESUMEN.....	IX
RIASSUNTO.....	X
PUBLICATIONS.....	XI
ACKNOWLEDGMENTS.....	XII
1 INTRODUCTION AND MOTIVATION.....	1
2 THESIS OVERVIEW AND OBJECTIVE.....	3
3 STATE OF THE ART.....	5
3.1 TBM description.....	5
3.2 Construction Loads in Reinforced concrete (RC) Ring.....	6
3.3 Gap backfilling procedure.....	7
3.4 Ring ovalisation.....	9
3.5 Joints design in TBM rings.....	10
3.5.1 Joints as Equivalent Beam.....	11
3.5.2 Joints as D-region: the concept of the partially loaded area.....	14
3.5.2.1 Previous Experimental studies.....	15
3.5.2.2 Cracking in D-regions.....	16
3.6 Concrete constitutive laws.....	18
3.7 Concrete plasticity theory applied to concentrated loads.....	21
3.8 State of art of TBM lining defects.....	24
4 METHODOLOGY FOR THE CLASSIFICATION AND MEASUREMENT OF DEFECTS IN TUNNEL LINING CONSTRUCTION.....	27
4.1 Introduction.....	27
4.2 Tolerances.....	32
4.3 Defects Encountered.....	33
4.3.1 Manual Mapping.....	33
4.3.1.1 Misalignments.....	33
4.3.1.2 Joint openings.....	37
4.3.1.3 Cracking.....	39
4.3.1.4 Ruptures.....	39
4.3.2 Automated Mapping.....	40
4.3.2.1 Ovalisation.....	41
4.3.2.2 Misalignments/rotations.....	42
4.3.3 Comparison with direct field data.....	43
5 METHODOLOGY FOR THE ANALYSIS OF TUNNEL LININGS UNDER THE HYPOTHESIS OF LACK OF BACK-FILLING.....	49
5.1 Introduction.....	49
5.2 Field Measurements.....	50
5.3 Non-linear analysis.....	51
5.4 Model Results.....	60

6	FORCE TRANSMISSION BETWEEN SEGMENTS: DESIGN OF THE EXPERIMENTAL CAMPAIGN	65
6.1	Purpose	65
6.2	Tests Programme	66
6.3	Test analytical design	69
6.4	Specimen construction.....	71
6.4.1	Concrete.....	71
6.4.2	Steel.....	72
6.4.3	Monitoring.....	73
6.4.3.1	Internal monitoring.....	73
6.4.3.2	External instrumentation.....	74
6.5	Test setup.....	75
7	EXPERIMENTAL RESULTS	77
7.1	Tests results	77
7.2	Stress strain response.....	80
7.3	Geometrical vs stress confinement	86
7.4	Cracking	87
7.5	Finite element approach.....	96
7.5.1	Software employed.....	96
7.5.1.1	Computational Aspects.....	96
7.5.1.2	Model parameters.....	98
7.5.2	FEM Results	99
8	CONCLUSIONS AND FUTURE RESEARCH	103
8.1	Conclusion regarding the characterization and measurement of tunnel lining defects.....	103
8.2	Conclusion regarding the behaviour of a tunnel lining under lack of back-fill	104
8.3	Conclusions regarding geometrical confinement.....	104
8.4	Conclusions regarding cracking	105
8.5	Conclusions regarding NLFEA analysis.....	105
8.6	Future research	106
	APPENDIX A: LABORATORY TEST DATA	107
A.1	C25 50: specimen 1.....	107
A.1.1	Instrumentation nomenclature	107
A.1.2	Instrumentation definition	108
A.1.3	Load Displacement curves	109
A.1.4	Strain Gauges curves	112
A.1.5	Failure Mechanism and remarks.....	114
A.2	C25 50: specimen 2.....	115
A.2.1	Instrumentation nomenclature	115
A.2.2	Instrumentation definition	115
A.2.3	Load Displacement curves	115
A.2.4	Strain Gauges curves	118
A.2.5	Failure Mechanism and remarks.....	122
A.3	C25 8: specimen 1.....	122
A.3.1	Instrumentation nomenclature	122
A.3.2	Instrumentation definition	124
A.3.3	Load Displacement curves	124
A.3.4	Strain Gauges curves	128
A.3.5	Failure Mechanism and remarks.....	132
A.4	C25 8: specimen 2.....	133
A.4.1	Instrumentation nomenclature	133
A.4.2	Instrumentation definition	133
A.4.3	Load Displacement curves	133
A.4.4	Strain Gauges curves	137

A.4.5 Failure Mechanism and remarks.....	142
A.5 C25 25 (BF25-12): specimen 1	142
A.5.1 Instrumentation nomenclature	142
A.5.2 Instrumentation definition	144
A.5.3 Load Displacement curves	145
A.5.4 Strain Gauges curves	148
A.5.5 Failure Mechanism and remarks.....	153
A.6 C25 25 (BF25-12): specimen 2.....	153
A.6.1 Instrumentation nomenclature	153
A.6.2 Instrumentation definition	153
A.6.3 Load Displacement curves	153
A.6.4 Strain Gauges curves	157
A.6.5 Failure Mechanism and remarks.....	161
A.7 BF25 8: specimen 1	162
A.7.1 Instrumentation nomenclature	162
A.7.2 Instrumentation definition	163
A.7.3 Load Displacement curves	164
A.7.4 Strain Gauges curves	168
A.7.5 Failure Mechanism and remarks.....	175
A.8 BF25 8: specimen 2	175
A.8.1 Instrumentation nomenclature	175
A.8.2 Instrumentation definition	175
A.8.3 Load Displacement curves	175
A.8.4 Strain Gauges curves	180
A.8.5 Failure Mechanism and remarks.....	187
A.9 BF25 16: specimen 1	188
A.9.1 Instrumentation nomenclature	188
A.9.2 Instrumentation definition	189
A.9.3 Load Displacement curves	190
A.9.4 Strain Gauges curves	192
A.9.5 Failure Mechanism and remarks.....	196
A.10 BF25 16: specimen 2	196
A.10.1 Instrumentation nomenclature	196
A.10.2 Instrumentation definition	196
A.10.3 Load Displacement curves.....	196
A.10.4 Strain Gauges curves	199
A.10.5 Failure Mechanism and remarks.....	203
APPENDIX B: FINITE ELEMENT ANALYSIS	204
B.1 C25 50.....	204
B.2 C25 8.....	209
B.3 C25 25 (BF25-12).....	213
B.4 BF25-8	218
B.5: BF25-16	222
BIBLIOGRAPHY	227

FIGURES INDEX

Figure 3-1 Shield TBM for rock. Principal components at left and cycle of work at right [68].	6
Figure 3-2 Typology of lining [68]	7
Figure 3-3 Gap-Fill procedure undertaken on field	8
Figure 3-4 Gap Filling phases	8
Figure 3-5 Ovalisation mechanism [21]	9
Figure 3-6 Joints nomenclature along a tunnel concrete ring	11
Figure 3-7 Jansenn's Equivalent beam approach (see [105])	12
Figure 3-8 Comparison between Jansenn and Gladwell approaches	14
Figure 3-9 Load distribution for partially loaded areas (see [33])	15
Figure 3-10 Rectangular block loaded by a concentrated strip load (at left) and Failure mechanism for a rectangular block loaded by a concentrated strip load far from the edge (at right) [123].	22
Figure 3-11 Yield line patterns formulas for a point load (b is small so that lateral confinement is not effective)	23
Figure 4-1 Positioning possibilities of the segments. From top left, position 1, 2, 3 (bottom left) and 5	29
Figure 4-2 Histogram and frequencies of ring positions employed along the excavation of the 4 tunnels	30
Figure 4-3 Joints geometries	30
Figure 4-4 Tunnels longitudinal profile	31
Figure 4-5 Hard fill Embankment area	31
Figure 4-6 Misalignments: longitudinal (left – compare with the theoretical joint geometry of Figure 4-3) and transversal (right)	34
Figure 4-7 Longitudinal misalignment measures	34
Figure 4-8 Phase 1 Measure of Keystone Defect (rotation and misalignment)	35
Figure 4-9 Phase 2 Measure of Keystone Defects	35
Figure 4-10 Quadranfix® provided with circular bars to measure misalignments and rotations of keystone joints	36
Figure 4-11 Possible edges ruptures observed	36
Figure 4-12 Joint openings: front view (at left) and lateral viewing (at right)	37
Figure 4-13 Sketch of joint rotation calculations	38
Figure 4-14 Joint rotation measurements: opening (at left) and penetration of normalized steel rod (at right).	39
Figure 4-15 Cracks at tunnel crown: mapping and monitoring (at left) and a bottom viewing (at right)	39
Figure 4-16 Edge rupture at left and sketch with measurements carried out at right	40
Figure 4-17 Sketch of ring ovalisation over the undeformed ring	41
Figure 4-18 Histogram of ovalisation ratio (maximum measured diameter divided by the inner diameter)	41
Figure 4-19 Comparison between laser scanner and Global survey	44
Figure 4-20 Comparison between global survey and local mapping for joint rotations at tunnel crown (pos 3-5)	45
Figure 4-21 Longitudinal Misalignments	45
Figure 4-22 Crack openings (mm)	46
Figure 4-23 Plane joints openings	47
Figure 4-24 Keystone joints openings	47
Figure 5-1 Measured ovalisation (mm) versus Tunnel cover (m)	50
Figure 5-2 Non-linear model	52
Figure 5-3 Joint geometry	52
Figure 5-4 Ring saddle (80°, split in in 40°+40°). Bolts are visible between segments	53
Figure 5-5 Subgrade moduli variation	55
Figure 5-6 Bending moment along the tunnel perimeter Subgrade moduli variation	56
Figure 5-7 Model employed in analysis (at top) vs real ring (at bottom)	56
Figure 5-8 Bolt section employed in non-linear analysis	57
Figure 5-9 Concrete (joint) constitutive law with no tensile strength	58
Figure 5-10 Concrete (segments) constitutive law with tensile strength	58
Figure 5-11 Steel employed in the segments (B-500SD) constitutive law	58
Figure 5-12 Bolt steel constitutive law	59
Figure 5-13 Moment curvature diagram	59
Figure 5-14 Model "A" internal forces redistribution	63
Figure 5-15 Model "B" internal forces redistribution	63
Figure 5-16 Model "C" internal forces redistribution	64
Figure 6-1 Reinforcement detailing	67
Figure 6-2 Strut and Tie for concentrated load [21]	69
Figure 6-3 Abrams con (at left) and vibration of concrete (at right)	71

Figure 6-4 Evolution of the concrete compressive strength employed in the tests	72
Figure 6-5 Spacers installation before concrete pouring (at left) and moving bars to move the concrete block (at right).....	72
Figure 6-6 Strain gauges installation.....	73
Figure 6-7 Mechanical strain gauges installed during a test (extensometer base is artificially painted in the figure).....	75
Figure 6-8 Neoprene base (at left) and plate employed as loaded area (at right)	75
Figure 7-1 Failure pressure versus $\sqrt{A_{co}/A_{cl}}$	79
Figure 7-2 Comparison of tests carried out with EC2 Approach	80
Figure 7-3 Comparison of tests carried out with EC2 Approach	80
Figure 7-4 Load/strain for top, central and lower bars for specimen C25-8-1	81
Figure 7-5 Load/strain for top, central and lower bars for specimen C25-8-2	82
Figure 7-6 Load/strain for top, central and lower bars for specimen C25-25-1	82
Figure 7-7 Load/strain for top, central and lower bars for specimen C25-25-2	83
Figure 7-8 Load/strain for top, central and lower bars for specimen C25-50-1	83
Figure 7-9 Load/strain for top, central and lower bars for specimen C25-50-2	84
Figure 7-10 Penetration of bearing plate after failure for C25-8 specimen 2.....	84
Figure 7-11 Load/strain for vertical extensometers at transversal direction	85
Figure 7-12 Load/strain for vertical extensometers at longitudinal direction	85
Figure 7-13 Relationship between geometric confinement and stress confinement	86
Figure 7-14 Average failure pressure for the cracking (#BF) specimen.	88
Figure 7-15 BF-25-Ø8- specimen 1: strain stress curves for longitudinal bars.....	89
Figure 7-16 BF-25-Ø8- specimen 1: strain stress curves for transversal bars.....	89
Figure 7-17 BF-25-Ø8- specimen 2: strain stress curves for longitudinal bars.....	90
Figure 7-18 BF-25-Ø8- specimen 2: strain stress curves for transversal bars.....	90
Figure 7-19 BF-25-Ø12 specimen 1: strain stress curves for longitudinal bars	90
Figure 7-20 BF-25-Ø12 specimen 1: strain stress curves for transversal bars	90
Figure 7-21 BF-25-Ø12 specimen 2: strain stress curves for longitudinal bars	91
Figure 7-22 BF-25-Ø12 specimen 2: strain stress curves for transversal bars	91
Figure 7-23 Specimen BF-25-Ø8-1 comparison between ext. central base and steel bars.....	92
Figure 7-24 Specimen BF-25-Ø8-2 comparison between ext. central base and steel bars.....	92
Figure 7-25 Specimen BF-25-Ø12-1 comparison between ext. central base and steel bars.....	93
Figure 7-26 Specimen BF-25-Ø12-2 comparison between ext. central base and steel bars.....	94
Figure 7-27 Specimen BF-25-Ø16-1 comparison between ext. central base and steel bars.....	94
Figure 7-28 Specimen BF-25-Ø16-2 comparison between ext. central base and steel bars.....	95
Figure 7-29 Crack width versus reinforcement stress for LS-G1 bar.....	96
Figure 7-30 ATENA's screenshot to define 3D Nonlinear Cementitious 2	97
Figure 7-31 Localization bands in tension and compression [37].....	99
Figure 7-32 Bar reinforcement stress comparison between ATENA and measured tests for C25-8-1	100
Figure 7-33 Bar reinforcement stress comparison between ATENA and measured tests for C25-8-2	100
Figure 7-34 Bar reinforcement stress comparison between ATENA and measured tests for BF-25-12 (C25-25)	101
Figure 7-35 Bar reinforcement stress comparison between ATENA and measured tests for BF-25-8.....	101
Figure 7-36 Bar reinforcement stress comparison between ATENA and measured tests for BF-25-16.....	101
Figure 7-37 Bar reinforcement stress comparison between ATENA and measured tests for C25-50-1	102
Figure 7-38 Comparison between cracking width predicted by NLFEA and average measured crack width ...	102

TABLES INDEX

Table 3-1 Tunnel geometrical definitions.	24
Table 4-1 Tunnel geometrical definitions.	28
Table 4-2 Geometrical positions of the joints depending of the ring position	30
Table 4-3 Tolerances for TBM lining updated by the author.....	32
Table 4-4 Ovalisation statistical parameters	42
Table 5-1 Cracks widths along the four tunnels.....	51
Table 5-2 Cracks at crown at the maximum ovalized ring.....	51
Table 5-3 Geometrical characteristics and gross mechanical properties of structural elements	53
Table 5-4 Different Subgrade scenarios.....	55
Table 5-5 Scenarios analysed depending of the saddle extension.....	60
Table 5-6 Summary table among the different scenarios and their evolution after the redistribution steps	61
Table 6-1 Tests Definition nomenclature.....	68
Table 6-2 Reinforced bars for cracking tests.....	68
Table 6-3 Dimensioning of the block reinforcement for concentrated load.....	69
Table 6-4 Measured concrete compressive strength (uniaxial tests).....	71
Table 6-5 Strain gauge features characteristics.....	74
Table 7-1 Dates of tests.....	77
Table 7-2 Summary of the tests results	78

SUMMARY

This work deals with the characterization and analysis of construction defects in TBM tunnel construction, with more emphasis on the behaviour of joints between segments.

Contributions are made towards defining a methodology for classification, measurement and analysis of various defects arising in this type of construction including: lack of backfill and resulting cracking of segments, ovalisation of rings, misalignment, joint opening and ruptures at joints. Measurement techniques are described including an innovative instrument to accurately measure misalignment and rotation at key-segment joint.

Then, a methodology for the analysis of cracking and yielding of segments due to lack of backfill is presented making use of plastic analysis. The principal virtue of this approach is that it can be easily applied with very simple means, available to the average designers. It also makes manifest the critical structural role played by assembly bolts which are normally considered only as secondary non-structural elements.

Finally, an experimental program carried out to investigate the compressive resistance of joints and the development of confining stresses is presented. An equivalence between geometric and stress confinement is analysed. It observed that a common type of defect, rupture at edges of segments near the longitudinal joints may be due to a lack of longitudinal stress confinement and the absence of geometric confinement.

RESUMEN

Este trabajo se centra en la caracterización y análisis de los defectos aparecidos durante la ejecución de túneles excavados mediante TBM, con particular atención al comportamiento de las juntas entre dovelas.

La contribución de este trabajo debe encontrarse en la definición de una metodología para la clasificación, medida y análisis de los varios defectos que suelen aparecer en este tipo de obras inclusive: la falta de relleno entre terreno y túnel, ovalización de los anillos, cejas, abertura y giros de las juntas. Las técnicas de medida empleadas se describen incluyendo una nueva herramienta capaz de medir de forma muy precisa las cejas y los giros de las juntas de la dovela de clave.

Además, se presenta una metodología para el análisis de las fisuras y de la tensión en las dovelas por efecto del relleno parcial del gap empleando la teoría de la plastificación. La principal virtud de este análisis es que puede desarrollarse con simple medios disponibles por la mayoría de los proyectistas. Se quiere subrayar con este análisis el rol fundamental que desempeñan los bulones de montaje que normalmente se consideran como elementos no estructurales.

Finalmente, se ha desarrollado una campaña experimental para analizar la resistencia a compresión de las dovelas y el desarrollo de la tensión de confinamiento. Se analiza la equivalencia entre la discontinuidad geométrica y aquella tensional. Se ha observado que un defecto común, las roturas en las esquinas de las dovelas, puede deberse a la falta de confinamiento longitudinal y a la ausencia de un confinamiento geométrico.

RIASSUNTO

Questo lavoro tratta sulla caratterizzazione e analisi dei difetti apparsi durante la costruzione di gallerie scavate mediante TBM, con enfasi sul comportamento dei giunti tra i conci.

Il contributo di questo lavoro va nella direzione di una definizione di una metodologia per la classificazione, misura ed analisi dei vari difetti apparsi in questo tipo di opere incluso: riempimento parziale del gap tra terreno e galleria, ovalizzazione degli anelli, disallineamenti, aperture e rotture dei giunti. Le tecniche di misura sono descritte includendo un nuovo strumento capace di misurare in forma accurata i disallineamenti e le rotazioni dei giunti di chiave.

Poi, si presenta una metodologia per l'analisi delle fessure e della tensione nei conci dovuta al riempimento parziale del gap utilizzando la teoria della plasticizzazione. La principale virtù di questa analisi è che può essere svolta con semplici analisi utilizzabili dal progettista medio. Si vuole sottolineare con questa analisi il ruolo critico che svolgono i bulloni di assemblaggio che normalmente si considerano come elementi non strutturali.

Finalmente, si è portato a termine un programma sperimentale per analizzare la resistenza a compressione dei giunti e lo sviluppo della tensione di confinamento. Si analizza l'equivalenza tra la discontinuità geometrica e quella tensionale. Si è osservato che un difetto comune, le rotture degli spigoli dei conci, può essere dovuta alla mancanza di un confinamento longitudinale e all'assenza di un confinamento geometrico.

PUBLICATIONS

Some ideas and figures have appeared previously in the following publications:

- Analysis of TBM Lining under lack of Gap Fill: Practical case of Ovalization and Crack Patterns**
2017 Cosimo Iasiello, Alejandro Pérez Caldentey, Giancarlo Groli
In Journal of Performance of Constructed Facilities.
DOI:10.1061/(ASCE)CF.1943-5509.0001010 Vol. 31, Issue 4 (August 2017)

ACKNOWLEDGMENTS

I am enormous grateful to my thesis director Alejandro Pérez Caldentey for infinite patience shown during our discussions along these years.

Special thanks to Giancarlo Groli for his support during the last two years and during the test conceptual design and data interpretation.

I would also like to thank Hugo Corres Peiretti for his suggestions during the first steps of the tests campaign.

Many thanks to my colleagues of the Structural Concrete Research Group Francesco Marchetto, Freddy Ariñez, Miguel Ángel Peña Cobos and Isidro Garcia for their help during the last years.

Special thanks to my former Ferrovial technical office colleagues and friends Félix Lorenzo Martín, Luis Martín Tereso, Rafael Rodriguez Molina, Fernando Gutierrez Manzanedo, Roberto Menendez, Juan Carlos Guerra and Dario Brambilla.

Last but not the least, I would like to express my thank you to my friends and my family who always encouraged me to finish this job.

NOTATION

A_{c0} = is the loaded area

A_{c1} = is the maximum design distribution area with a similar shape to A_{c0}

C_r = stiffness of the flexural spring defined by Blom

E_c = Young's modulus of concrete

I = modulus of inertia of the section

I_n = modulus of inertia of segment section

I_r = equivalent ring modulus of inertia

I_s = modulus of inertia of joint section

N = axial load

M = bending moment

P_u = failure load

R = square root of ratio between the bearing area and the effective area ($\sqrt{A_{c0}/A_{c1}}$)

b = joint thickness (m)

c = ring ovalisation causing collapse

d_c = is the bottom cover

f = thickness of the section

f_{cd} = design compressive strength of concrete

f_{ctd} = is the design tensile concrete strength

f_{ck} = is the characteristic concrete strength

f_c = mean compressive strength of concrete

h = joint height (m)

j = design joint opening (19mm).

$k_{ct} = 3.0$ is known as "confinement" factor.

q = number of segments (small key segment not counted)

m = measured joint opening

r = inner external radius

s = permissible bar spacing

T = surface joint opening

α = crack amplification factor

β = ATENA's model calibration factor

χ = is the maximum curvature defined as the ratio between the maximum moment and the flexural stiffness ($E_c I$)

ν = Poisson's modulus

1 INTRODUCTION AND MOTIVATION

This thesis is motivated by the personal experience of the author acquired during the building of four tunnels bored using a TBM machine. During the construction, many defects appeared and although this is not an extraordinary situation, both the design and construction were questioned by the owner. This experience forced a critical review of current design methods in which allowance for less-than-ideal construction conditions does not seem to be sufficient.

Some of these construction problems which are not normally considered in design are :

- ✓ Uneven backfill after segment installation
- ✓ Ruptures and misalignments at the segment joints which increase already high stress at the joints.

Furthermore, the normal design process is very simplistic and does not include verification regarding the joint behaviour prior to the application of ground pressure, at which point the stability of the joints is only guaranteed by means of standard bolts. In this situation, uneven ground pressures due to imperfect back-filing can become critical, at least for the serviceability of the structure.

Another point of controversy is the compression capacity at the joint. The theoretical stress at this point exceeded by a wide margin that allowed by the current standards and were justified on the basis of tests carried out for another tunnel. The transmission of these high stresses relies on the generation of a triaxial state of compression. However, it is not clear that confining pressures are really present in the longitudinal direction, particularly near the transversal joints.

All these questions, the need to justify the fact, that in spite of all the tunnel did not show any structural distress and was considered safe, the need to have a good methodology for serviceability and resistance have all presented the need for a more profound study and are at the root of this study.

2 THESIS OVERVIEW AND OBJECTIVE

The objectives of this thesis are:

- ✓ To establish a methodology for the classification and measurement of tunnel lining defects based on the experience acquired in the field studying a real case
- ✓ To establish a methodology for the analysis of the behaviour of a tunnel lining under the hypothesis of lack of back-filling, allowing to determine its stability taken into account the redistribution of forces
- ✓ To study the behaviour of joints between segments of TBM linings subjected to high compressive stresses

The organisation of the work has been divided into:

- Chapter 3: State of art regarding the TBM design
- Chapter 4: Real case observation
- Chapter 5: Global analysis of cracking and joints sections
- Chapter 6: Experimental Tests
- Chapter 7: Experimental Results
- Chapter 8: Conclusions and future works

3 STATE OF THE ART

In this chapter, a state of art of the most relevant elements is collected, focusing on the features discussed in this thesis.

3.1 TBM description

A tunnel-boring machine is a machine capable of excavating a tunnel with a complete cross section, and can be equipped, according to the different cases, with auxiliary models for the positioning of provisional or definitive lining [68] and [157].

The advance generally takes place by the pushing of a series of perimeter jacks that act against the precast lining (ring composed of segment elements), made of concrete (reinforced or not), which are put in place once the previous excavation cycle has finished. In some cases, as in the so called “moles” the pushing is carried out against the walls of tunnels.

A steel shell known as a “shield” protects all the operations and elements of the machine (cutters support, driving part, debris conveyor belt, jacks, erector for the placement of segments).

The placement of the prefabricated ring, which in most tunnels acts as the definitive lining, is carried out at the end of the shield.

The fundamental parts, shown in Figure 3-1 (left) are:

- 1) Cutter-head
- 2) Driving part
- 3) Pushing jacks (thrust arms)
- 4) Erector for placement of prefabricated segments
- 5) Conveyor belt for evacuation of debris from the face

Figure 3-1 Shield TBM for rock. Principal components at left and cycle of work at right [68].

The cycle of work is displayed in Figure 3-1 (right) and is comprised of:

- Excavation
- Partially withdrawn jacks
- Placement of the prefabricated ring
- Following excavation advance

In general, due to the heterogeneity of the material to be excavated, the TBM design and its constitutive parts must tolerate the worst conditions to avoid any problems [7].

3.2 Construction Loads in Reinforced concrete (RC) Ring

Except for the gripper mole, the rest of the tunnel boring machines are designed, due to the system itself, to include the placement of a lining of prefabricated segments.

Therefore, it is necessary to use a system of precast segments, which the shield can react against, to provide the necessary reaction on the tunnel face. A first discussion arises about the choice between a single lining or a combination of a primary lining and a secondary lining (so-called double lining) as shown in Figure 3-2:

Figure 3-2 Typology of lining [68]

During the ring installation, segments are subjected to a series of loads, so-called construction loads, which must be considered during design process.

As stated in the introduction, these loads introduce an increment of internal forces, mainly bending moment, in a segment where structural checking have been previously calculated under compressive loads and whose steel reinforcement normally complies only with minimum reinforcement requirements.

Japanese shield tunnelling standards, see that article 32 of [84], states that: “if it is judged that excessive stress and deformation of segmental lining will be generated by construction loads, the shape and the dimensions of the segments and planned construction facilities need to be reviewed until it can be assured that the segmental rings will not be damaged during construction”

3.3 Gap backfilling procedure

The injection of the space between the ring and the ground (the so-called gap), created by the difference between the rear of the shield and the ring, is an element related more to the construction process of the tunnel than to design as stated in [81] and [82].

Gap backfill is meant to perform several functions:

- It ensures a uniform distribution of the ground pressures.
- It reduces decompression of the ground that could have negative effects on subsidence (in particular for shallow tunnels)
- It works as an additional barrier for waterproofing.
- The alkalinity of the contact between the ground and the lining increases, so the durability of the ring improves.

The typology of backfill depends on the ground conditions, the system implemented in the machine and, in general with construction issues as stated in [32] and [131].

The procedure of backfilling is to inject the material firstly into the invert arch, then into side walls and finally into tunnel crown as shown in the Figure 3-3.

Figure 3-3 Gap-Fill procedure undertaken on field.

Figure 3-4 shows the procedure of gap filling for tunnels in rock/soil. The phases of gap filling are:

- Phase 0: the ring has been installed and is held in place by longitudinal and circumferential bolts within the shield
- Phase 1: Execution of the so-called “saddle”. This operation starts just after the ring installation. The ring, at this stage, is outside the shield.
- Phase 2: Execution of the drift-walls gap filling. This operation takes some time to be covered and starts after the installation of 2 to 4 other rings.
- Phase 3: gap-filling completion. This operation takes place after the installation of another 2 to 4 rings

Figure 3-4 Gap Filling phases

In summary, the time each ring has to “wait” before its gap is fully filled can vary from four to eight other ring installations. If the TBM advance does not encounter any problems (geological formation, local presence of water table) or any maintenance problems (cutters, convoy belt and so on), the time between phase 1 and phase 3 theoretically can vary from one to three days to months. During this time, the ring is experimenting deformations, which can alter its internal stress distribution

3.4 Ring ovalisation

Extensive research has been carried out to study the ovalisation of tunnels and the stress distribution within the ring. It is important to point out that none of the major standards [84], [9] or [156] take into account the ovalisation of the TBM-tunnel as a design state to be analysed. Nevertheless, several authors, along the years, have highlighted the influence of ovalisation stresses.

The first studies were based on the global effect of ovalisation, i.e. the lack of the circular form of a ring, and were carried out by Mindlin [117].

If the tunnel ring is thought of as a thin ring, some Authors have studied theoretically the buckling of an ovalized ring under a bending moment pair (see references [21] and [44]) which is governed by the following closed formula:

Figure 3-5 Ovalisation mechanism [21]

$$c = \frac{3r\chi^2}{5\chi^2 + \frac{6f^2}{r^4(1-\nu^2)}} \quad (3-1)$$

where:

r= inner external radius

c= ring ovalisation at the onset of buckling

χ = is the maximum curvature defined as the ratio between the maximum moment and the flexural stiffness ($E_c I$)

E_c = Young´s modulus of concrete

I= modulus of inertia of the section

f = thickness of the section

ν = Poisson's modulus

This formula can be used as a first approximation, for example for pipe design, although with some changes [158].

This approach, however, was superseded by the investigations of Peck [130] regarding the relationship between ground settlements and ring ovalisation for shallow tunnels.

Sagasetta [143] completed Peck's approach for an incompressible soil. Verrujit [166] presented an approximate analytical solution considering the soil as a linear elastic material.

Later, the same Author ([167], [168] and [169]) completed Mindlin's solution presenting a complex variable solution of the interaction between tunnel and ground.

All these approaches take into account the ground loads with different ratios between vertical and horizontal loads of a ring without considering the joints.

3.5 Joints design in TBM rings

Joints can be defined as geometrical discontinuities along the ring because they are the sections where segments are connected with each other.

These surfaces have to support compression and shear stress, generated both by the TBM construction loads and by the ground. To adsorb these shear stresses, it is necessary to employ, in some cases, screws or even bolts in transversal and longitudinal directions.

The importance of joints in tunnel geotechnical simulations has been studied by Lee [93] and by Xiaochun [172], who concluded that the differences in terms of internal forces between a tunnel simulation with or without the presence of joints can easily reach 50%.

In other words, the absence of joint simulation can lead to an unsafe estimation of internal forces simulation [26].

Gruebl [66] analysed the behaviour of a tunnel with or without the joints simulating the lining as a shell, reaching the conclusion that for a 3D model, the joint simulation is almost mandatory. Some authors such [85] and [159] have modelled the joints to study their influence on tunnel segments with fiber reinforcement.

The influence of the joint has been analysed in literature in some specific case, such as tunnels in soft grounds [50], the tunnel deformations in combination with surrounding gap deformations [64], to evaluate the loads along the lining during ring installation [112] and [113] or, finally to evaluate the behaviour of the ring under seismic loads [162].

The design and the geometry of the joints depend on the functionality of the tunnel (see for instance AFTES [8]) and on the ring typology (tapered or universal).

Another important issue is to correctly model the ring positions and of joints within the lining. Hefny et al. [71] carried out a parametric study of the joint positions for the Singapore metro reaching the conclusion that if the joint is located exactly at the tunnel crown the influence on the bending moments is the lowest.

The transversal joints, shown in Figure 3-6, should be designed to ensure correct transmission of internal forces within the ring so they could have different geometrical shapes; however, to build a simple formwork, it is very common that longitudinal and circumferential joints have the same geometrical shape

Figure 3-6 Joints nomenclature along a tunnel concrete ring

As stated in the Introduction, the partially backfilling of the ring could lead to the so-called snap-through which can be defined as the lack of support in the joints because of lack of axial force. A study regarding this effect was carried out by Van der Waart [161] focused on a parametric study of snap-through of three large diameters shield driven tunnels.

Engineering practice distinguishes two modelling steps for the design of joints:

- equivalent beam, mainly used in numerical simulations where the soil structure interaction is studied in geotechnical simulations, to determine the distribution of forces
- D-region: as geometrical discontinuity, a joint can be studied as a D-region. This approach is widely employed for structural detailing

3.5.1 Joints as Equivalent Beam

Historically the design of a TBM ring, as collected in Wood [121], did not expressly consider the joints within the segment: the only expedient used was to reduce the global rigidity of the ring employing the following formula:

$$I_r = I_s + I_n \left(\frac{4}{q} \right)^2 \quad (3-2)$$

Where:

I_r = equivalent ring modulus of inertia

I_s = modulus of inertia of joint section

I_n = modulus of inertia of segment section

q = number of segments (small key segment not counted)

In the 1980s, with the increasing TBM tunnel construction, and with the contribution of some authors such as Gladwell [63] and Jansenn [83], whose tests were based on Leonhardt and Riemann [95], the design approach radically changed. The idea was to simulate the joint as a “finite” discontinuity in the segment as shown in the Figure 3-7 where the length of the equivalent beam is equal to the height of the joint and collected in [132] and [27]:

Figure 3-7 Jansenn's Equivalent beam approach (see [105])

Although this approach seems to fit with the reality, the real joint behaviour is more complex and this does not take into account the following issues which, in some cases, could play a fundamental role in design:

- The behaviour of the concrete within the joint is typical of confined concrete, due to longitudinal stress, and a section reduction at the joint along the same ring, and the circumferential pressure because of the TBM jacking action.
- The presence of bolting is not expressly taken into account in this configuration and in some cases, such low coverage, fault areas and construction loads, the bolting system is essential for the segments to work as a monolithic structure.

In reality, this approach allows to recreating a Bernoulli region (B-region) in a zone where these conditions are not fulfilled such the joint segments.

This artifice allows simulating the joint rotation and its curvature using a beam of variable depth. In reference to Jansenn's approach it is possible to see how the linear relationship between bending moment, M , and joint rotation is a function of the axial force, N , and of the joint geometry:

$$\varphi = \frac{Mh}{E_c I} = 12 \frac{M}{E_c h^2 b} \text{ if } M < \frac{1}{6} Nh \quad (3-3)$$

$$\varphi = \frac{8N}{9bhE_c \left(\frac{2M}{Nh} - 1 \right)} \quad \text{if } M \geq \frac{1}{6}Nh \quad (3-4)$$

where:

h= joint height (m)

b= joint thickness (m)

E_c= Young's modulus of concrete

I= modulus of inertia of the section

N, M are the internal forces (Axial load and bending Moment).

Janssen's approach is more accurate for thick joints because in the equivalent 'beam' a curvature will develop resulting in a rotation. In reality, when the joint is very thin, only a very minor additional rotation will occur.

Gladwell proposed another Moment/joint rotation, which is very similar to the Janssen's but also accounting for dilation in the perpendicular directions through Poisson ration. The formula is:

$$\varphi = \frac{32(1-\nu)^2 M}{E_c b h^2} \quad \text{if } M < \frac{1}{4}F_n \quad (3-5)$$

$$\varphi_2 = \frac{4(1-\nu)^2 N}{\pi E_c b h \left(\frac{2M}{Nh} - 1 \right)} \quad \text{if } M \geq \frac{1}{4}F_n \quad (3-6)$$

As shown by Van Der Vliet [160] and afterwards by Peña [132], there are three main differences between the Gladwell's and Janssen's approach:

- The initial stiffness obtained by Gladwell is greater than Janssen's. This is due to the type of analysis carried out by Gladwell in which the contact stresses are concentrated on the edges of the joint.
- The non-linear branch proposed by Gladwell reaches very quickly the maximum theoretical bending moment because in Gladwell's approach the stiffness is higher and the joint stays close longer than in Janssen's one.

Figure 3-8 Comparison between Jansenn and Gladwell approaches

- The differences encountered between the two solutions, in particular in the linear branch, are due to the different formulation of joint and the segment height.

Van Der Vliet [160] showed that the joint simulation in finite element modelling, seems to better follow the Gladwell's approach than Janssen's.

In order to better simulate the joints in finite elements models, Blom [27] considered the longitudinal joints as flexural springs with a stiffness, " cr ", defined as:

$$c_r = \frac{1}{\varphi} \text{ if } M < \frac{1}{6}Nh \quad (3-7)$$

$$c_r = \frac{1}{\varphi} \text{ if } M \geq \frac{1}{6}Nh \quad (3-8)$$

Where the term φ is that defined in equations (3-3) and (3-4). Equation (3-8) is employed when the rotational stiffness is non-linear but the ultimate compressive strains are in the elastic branch. The circumferential joints are considered as shear springs too.

Arnau [15] has adopted this formulation in his experimental study with good results.

3.5.2 Joints as D-region: the concept of the partially loaded area

From structural point of view, the geometric discontinuities in concrete elements are defined as D-regions, or discontinuity/disturbed regions. Partially loaded areas have been widely analysed and major Standards analyse the problem based on the Strut&Tie method. EN 1992-1-1 offers the following formulation for the concentrated loads, which are typical of joints between segments.

$$F_{rd} = A_{c0} f_{cd} \sqrt{\frac{A_{c1}}{A_{c0}}} \leq k_{ct} f_{cd} A_{c0} \quad (3-9)$$

Where:

A_{c0} is the loaded area

A_{c1} is the maximum design distribution area with a similar shape to A_{c0}

$k_{ct}=3.0$ is known as “confinement” factor.

The geometrical features of the problem are well known (see Figure 3-9)

Figure 3-9 Load distribution for partially loaded areas (see [33])

3.5.2.1 Previous Experimental studies

Experimental studies on the bearing capacity of concrete have been undertaken to study different aspects. The tests carried suggest that the compressive strength of concrete increases if it is confined. Experimental investigations (see for instance [149],[70], [124], [125], [116], [146] and [16]) show that the bearing resistance of concrete depends on:

- The loaded area
- The cross section of the loaded member
- The loading conditions
- The strength of materials (concrete and steel)

Experimental trials on concrete elements for tunnel joints under the fourth river Elba in Germany ([150] and [151]) have shown that the increase in compressive resistance of confined concrete can reach 6.0. Experimental trials carried out in Sheffield and in Japan [86] have shown that this increment can reach 10.

In fact, the analysis performed by Bairan and Mari [18], on 3D models, demonstrates that the tri-dimensional effects that take place in linear reinforced concrete elements can be very relevant to the actual behaviour of these structures for some load cases, especially when the tangential forces are large or there is enough reinforcement to sufficiently confine the concrete.

The traditional elements for modelling beams and columns that are based only on the behaviour of concrete under normal forces can be very useful and satisfactory in a great number of practical cases, such as the case of joints between concrete segments, even if they are unsuitable in other cases. Spieth [149] was the first to suggest that a high stress can be reached in concrete blocks. Meyerhof [116] and Shelson [146] found that the failure of concrete subjected to concentrating loading was similar to that observed in triaxial compression tests of soils.

Au and Baird [16] developed a theory on the formation of an inverted pyramid under the loading plate. They assumed that this distribution of pressures within the concrete block would produce a combined tension and bending in the concrete block.

Failure would occur when the maximum tensile stress at the top of the block exceeded the tensile strength of the concrete. Hawkins [70] proposed the first realistic approach on the bearing strength of concrete by performing tests on concrete blocks on a stiff support loaded through a rigid plate.

Niyogi ([124],[125]) investigated the effect of the factors such as specimen geometry, nature of support, eccentricity, mix properties strength of concrete and specimen size on the bearing resistance of concrete.

Ince and Arici [79] investigated the size effect of bearing concrete. These investigations, which are part of the theoretical base of current Standards, provided with some difference the following formula:

$$q = \frac{P_u}{A_{c0}} = f'_c \sqrt{R} \quad (3-10)$$

Where

P_u = failure load

f'_c = mean compressive strength of concrete

R = square foot of ratio between the bearing area and the effective area ($\sqrt{A_{c0}/A_{c1}}$)

3.5.2.2 Cracking in D-regions

Many code provisions and recommendations intend to limit the crack width aiming to preserve the strength of the strut and maintain equilibrium after cracking or avoiding the reinforcement corrosion which could be caused by large cracks. Usually, in engineering practice, the amount of reinforcement is calculated on the basis of ultimate limit states, but often is required a verification of the steel stress for serviceability limit states, is also required. In any case, a minimum amount of reinforcement is required. Leonhardt [96] recommended that in case of bursting forces, the tension in steel reinforcement be limited to 180-200 MPa in service, in order to reduce the width of cracks and improve the bond with the surrounding concrete. However, Rizkalla et al. [141] have shown that "Leonhardt's prediction for the average crack width at the stabilized crack pattern, is highly overestimated, since the ratio between the measured values and those calculated by Leonhardt suggest an average value of 2.38. TxDOT Bridge Design Manual – LRFD [13] also requires the stress in longitudinal reinforcement to be limited to 22 ksi (151.7 MPa) to minimize cracking due to the only action of dead loads.

3.5.2.2.1 ACI

In ACI appendix A, the crack widths in a tie can be controlled using the same provisions for flexural cracks, that is, a maximum bar spacing is specified directly to control cracking. ACI provides no direct limitation on crack width. Instead, it recognizes the need of preserve the strut

strength and proposes the indirect procedure derived by Frosch [58], who, starting from a physical model, derived an equation for the maximum crack width and rearranged it for the permissible bar spacing as follows:

$$s = 2 \sqrt{\left(\frac{w_c E_s}{2 f_s \alpha} - d_c^2 \right)} \quad (3-11)$$

where d_c is the bottom cover, α a crack amplification factor $= \varepsilon_h / \varepsilon_d = (h-c)/(d-c)$, w_c the admissible crack width and f_s the reinforcement stress (until 2005 it was permitted to take f_s as 60% of the specified yield strength. Due to changes in Φ -factors and load combinations, the 2014 version was reformulated to be consistent with the higher service load stresses, taking $f_s = 2/3 f_y$).

ACI318R-14 [11] defines a tie as "reinforcement or prestressing steel plus a portion of the surrounding concrete that is concentric with the axis of the tie". ACI recognizes that, as previously explained, stress obtained from a S&T model will be higher when it states: "The concrete in a tie is not used to resist the axial force in the tie. Although not considered in design, the surrounding concrete will reduce the elongations of the tie, especially at service loads". The need of preserve the strut strength is recognized at §.23.5.3: "Reinforcement placed to resist the splitting force restrains crack width, allows the strut to resist more axial load, and permits some redistribution of force".

The transverse tensile force resulting from the compression force spreading in the strut, has therefore to be resisted by reinforcement crossing the strut. In absence of reinforcement, the effective compressive strength (f_{ce}) of a normal concrete in a bottle-shaped strut (§.23.5.3), is to be taken as the 80% of the same strut crossed by adequate reinforcement. Also, it is worth of notice that in ACI Chapter 9 (§9.9.3.1) additional minimum shear reinforcement for deep beams requirements of 0.0025 for reinforcement perpendicular or parallel to the flexural tension reinforcement are specified. ACI paragraph §23.2.3 states that distributed reinforcement in similar types of D-regions will improve serviceability performance. In addition, crack widths in a tie can be estimated using a space function of bonded stress assuming the tie is encased in a prism of concrete corresponding to the area of the tie.

3.5.2.2.2 LRFD

AASHTO LRFD uses the same crack control concept of ACI §A.9.9.3.1: a minimum amount of reinforcement should maintain the strength of the struts and limit the crack width. The only difference with respect to the ACI is that in this case a ratio of 0.003 is required in both direction, not only in a direction perpendicular to the strut (§5.6.3.6 Crack control reinforcement)

3.5.2.2.3 EC2

EN1992-1-1[33] limits the tension in steel under the characteristic combination of loads to $k_3 f_{ck}$, the recommended value of k_3 being 0.8. EN1992-1-1 also propose a direct calculation of the characteristic crack width w_k (§7.3.4), which has to be less than the recommended values included in table 7.1N. SLS checks (steel stress and crack width control) can also be carried out using strut-and-tie models, if approximate compatibility is ensured (position and direction of important struts should be oriented according to linear elasticity theory).

3.5.2.2.4 EHE 08

Spanish standard, EHE08 [118] limits the maximum deformation of the tie to 2‰ (400 MPa) in ULS. This condition is indicated to indirectly control cracking. This condition can be overridden if a compatibility study is carried out.

3.5.2.2.5 Model Code 2010

MC2010 [57] (§7.6.3) considers the tensile stresses in reinforcement should be limited with an appropriate safety margin below the yield strength, preventing uncontrolled open cracks. It provides a direct calculation method with crack width limitations generally between 0.2 and 0.4 mm. Fib also recognizes that “[...] concerning limiting the tensile stress in the steel reinforcement, under the characteristic combination of actions, the limit $0.8 f_{yk}$ should not be exceeded” [19]

3.6 Concrete constitutive laws

In the calculation of tunnels and, in general in soil-structure interaction, reinforced concrete is usually represented as a homogenous, isotropic and linearly elastic material with a tensile modulus function of resistance to compression, f_{ck} , with a weight of 24/25 KN/m³ and a Poisson’s ratio of about 0.2 [14].

In reality, concrete has a much more complex behaviour non-linearities, fractures or loads that do not act as foreseen in the project.

Moreover, depending of the concrete strength characteristics, concrete properties can be different [54].

Many authors such as, Lombardi [102] and Ribacchi [139] have solved analytical equations for determining the internal forces within tunnels considering plastic, elasto-plastic and viscoplastic behaviours of the rock (see for instance reference [103]). Their solutions were always based on the general hypothesis of considering the support of the tunnel as something perfectly elastic.

In recent years, however, there has been a tremendous effort to develop analytical models that can predict, with certain reliability, the response of concrete to variable loads and implement these models in finite element analysis.

In general, following what has been gathered from several authors, the best model to use for concrete depends on what it is meant to study, test or calibrate in a given analysis.

Thus, there is no perfect model for modelling concrete, so it should be emphasized that in some cases it is perfectly acceptable to use an elastic model for reinforced concrete as this behaviour can truly be a good starting point, but this cannot be considered the end point in evaluating the structural response [76].

In the following paragraphs, there is a brief summary of some of the models used in this thesis, and their most well-known formulations. Particular interest is the overview of the constitutive modelling laws in concrete done by [17]. The first models were based on the theory of elasticity.

More recently, the proposed models are based on solid mechanics of solids, including the theory of Plasticity, the theory of damage and fracture mechanics. So the majority of the proposed

models only predict certain aspects of the response of the concrete with an acceptable reliability and efficiency.

Some of the first constitutive models were developed, as mentioned above, based on the theory of Elasticity and used a variety of methods to represent the non-linear response of concrete under multidimensional forces or under cycles of loading and unloading.

Anyway, the best known constitutive law for instantaneous behaviour of concrete is the one proposed by Sargin and Handa ([144])

$$\frac{\sigma_c}{f_{c0}} = \frac{\frac{A\varepsilon_c}{\varepsilon_{c0}} + (D-1)\left(\frac{\varepsilon_c}{\varepsilon_{c0}}\right)^2}{1 + (A-2)\frac{\varepsilon_c}{\varepsilon_{c0}} + D\left(\frac{\varepsilon_c}{\varepsilon_{c0}}\right)^2} \quad (3-12)$$

Where

A is the ratio between the tangent and secant modulus of concrete from the origin to the peak compressive strength, f_{cm} .

D is a constant that controls the descending branch of the curve

ε_{c0} = is the reference value 0,0022 (see [144]), of the strain corresponding to the maximum stress

ε_c = is the strain considered at the calculated point

This formulation shows a certain deformation-softening behaviour above the stress peak. This equation has been adopted by EN1992-1-1 ([33]) for the design and testing of reinforced concrete structures.

There are a great variety of constitutive models based on the theory of plasticity that try to represent the mechanical properties of the concrete, most of which establish a [28]:

- flow function
- flow rule
- hardening and softening law

The flow function integrated in the plasticity model can arise from the classic fault criteria such the criterion of Tresca, von Mises, Rankine, Mohr-Coulomb and Drucker-Prager (de Souza Neto et al. [49], Linero & Garzón-Alvarado [99]), or from the criteria developed specifically for concrete such as those proposed by Ottosen [129] Willam & Warnke [170], Menetrey & Willam [115], Hsieh et al. [75], and Hu & Schnobrich [77], amongst others.

It seems that for confinement concrete, which is normally understood as a reinforcement of concrete whose restraint is perpendicular to the applied stress, the constitutive law which best fit the reality are those of Ottosen, William & Warnke and Drucker Prager [51].

Ottosen directly correlates concrete compressive and tensile strength with the principal stresses, f_{c1} , f_{c2} and f_{c3} . Moreover, the different geometrical conditions in the two directions ($\sigma_1 \neq \sigma_2 \neq \sigma_3$) does not allow to use the classical predictive formula of the confined compressive strength ($\sigma_1 = \sigma_2$). The equation is:

$$a \frac{J_2}{f_c'^2} + \lambda \frac{\sqrt{J_2}}{f_c'} + b \frac{I_1}{f_c'} - 1 = 0 \quad (3-13)$$

where

$$\lambda = k_1 + k_2 \cos 3\theta \quad (3-14)$$

$$I_1 = f_{c1} + f_{c2} + f_{c3} \quad (3-15)$$

$$s_i = f_{ci} - \frac{1}{3} I_1 \quad (3-16)$$

$$J_2 = \frac{1}{2} (s_1^2 + s_2^2 + s_3^2) \quad (3-17)$$

$$J_3 = \frac{1}{3} (s_1^3 + s_2^3 + s_3^3) \quad (3-18)$$

$$\cos 3\theta = \frac{3\sqrt{3}}{2} \frac{J_3}{J_2^{3/2}} \quad (3-19)$$

Moreover, it is important to determine where the analysed points are on the Haigh-Westergaard coordinates. To do that two parameters must be defined:

a) Hydrostatic length

$$\xi = \frac{I_1}{\sqrt{3}} \quad (3-20)$$

b) Deviatoric length

$$\rho = \sqrt{2J_2} \quad (3-21)$$

The concrete material constitutive law employed in the analysis presented in the paragraph 7.5, is that provided by the finite element software employed, ATENA, for 3D-non-linear cementitious materials and corresponds to the Menetrey-Willam criterion [115] whose governing equation is:

$$F_{3P}^p = \left[\sqrt{1.5} \frac{\rho}{f_c'} \right]^2 + m \left[\frac{\rho}{\sqrt{6} f_c'} r(\theta, e) + \frac{\xi}{\sqrt{3} f_c'} \right] - c = 0 \quad (3-22)$$

In the above equations (ξ, ρ, θ) are Haigh-Westergaard coordinates collected in (3-20) and (3-21) while f_c' and f_t' are the compressive strength and tensile strength respectively and:

$$m = 3 \frac{f_c'^2 - f_t'^2}{f_c' f_t'} \frac{e}{e+1} \quad (3-23)$$

$$r(\theta, e) = \frac{4(1-e^2) \cos^2 \theta + (2e-1)^2}{2(1-e^2) \cos \theta + (2e-1) [4(1-e^2) \cos^2 \theta + 5e^2 - 4e]^{1/2}} \quad (3-24)$$

Parameter e can vary from 0.5 to 1.0 and defines the roundness of the failure surface. The failure surface has sharp corners if $e = 0.5$ and is fully circular around the hydrostatic axis if $e = 1.0$.

For the Menetrey-Willam surface the hardening/softening is controlled by parameter c which evolves during the yielding/crushing process according to the following relationship:

$$c \langle 0, 1 \rangle = \left(\frac{f_c'(\varepsilon_{eq}^p)}{f_c'} \right)^2 \quad (3-25)$$

In the above two formulas, the expression $f_c'(\varepsilon_{eq}^p)$ indicates the hardening/softening law, which is based on the uniaxial compressive test. The softening curve is linear and the elliptical ascending part is given by the following formula:

$$\sigma = f_{c0} + (f_c - f_{c0}) \sqrt{1 - \left(\frac{\varepsilon_c - \varepsilon_{eq}^p}{\varepsilon_c} \right)^2} \quad (3-26)$$

where ε_{eq}^p is the equivalent plastic strain at compressive strength.

The fracture energy of concrete G_F [N/m], defined as the energy required to propagate a tensile crack of unit area, is determined through the Model Code [57] formulation:

$$G_F = 73 f_{cm}^{0.18} \quad (3-27)$$

3.7 Concrete plasticity theory applied to concentrated loads

The plasticity behavior of concrete has been used to study the pure compression of a prismatic body [123] corresponding to the upper bound solution of [40]. The body is separated into three

rigid parts. Part I is translating vertically downward, and parts II and III horizontally apart. The relative displacement of I to II and I to III is a sliding and separation and it makes an angle φ to the yield line. In reference to Figure 3-10 the displacement of I is u_1 , and the displacement of II and III is u_2 , assumed for part II to the left and for part III to the right.

Figure 3-10 Rectangular block loaded by a concentrated strip load (at left) and Failure mechanism for a rectangular block loaded by a concentrated strip load far from the edge (at right) [123].

In this case the maximum allowable pressure is given by:

$$P = 2af_t \left[\frac{h}{a} \tan(2\beta + \varphi) - 1 \right] \quad (3-28)$$

Particular interest covers the case in which the bearing plate is close to the edge so that the lateral geometrical confinement is not ensured. This solution has already been studied in Nielsen [123] for an unreinforced concrete rectangular block the following formula:

$$P = 2a(f_c + 4(\frac{h}{2a} - 1)f_t) \quad (3-29)$$

According to Nielsen the factor 4 in the second term of the above equation is too high and it has been reduced to 2.48 to better match experimented evidence [40] and the formula less than 10%. In this case two different approaches, depending of the possible failure mechanism have been observed:

Figure 3-11 Yield line patterns formulas for a point load (b is small so that lateral confinement is not effective)

If a linear vertical crack is generated (splitting), the governing equation is:

$$\frac{\sigma}{f_c} = 1 + K_3 \left(\frac{bh}{a^2} - 1 \right) \frac{f_t}{f_c} \quad (3-30)$$

whose:

k_3 is a factor equal to 2.48

f_c is the uniaxial compression strength of concrete

f_t tensile strength of concrete

If the vertical crack does not appear (local rupture), the governing equation is:

$$\frac{\sigma}{f_c} = 1 + K_4 \left(\frac{b}{a} - 1 \right) \frac{f_t}{f_c} \quad (3-31)$$

whose k_4 can vary between 25 and 30

These formulas were generalized by Nielsen [123], for 3-D situations with semi-empirical approaches reaching the following corrections regarding the factors K_3 and K_4 , and introducing areas instead of the unidimensional values a , and b .

Splitting Failure:

$$\frac{\sigma}{f_c} = 1 + 2.8 \left(\frac{bh}{2A_{c0}} - 1 \right) \frac{f_t}{f_c} \quad (3-32)$$

Local Failure:

$$\frac{\sigma}{f_c} = 1 + 25 \left(\sqrt{\frac{A_{c1}}{A_{c0}}} - 1 \right) \frac{f_t}{f_c} \quad (3-33)$$

where A_{c0} and A_{c1} are the values defined in 3.5.2

These equations are compared with tests carried out in [127] and [128] with a good agreement. It is important to underline that these formulations do not consider the influence of the reinforcement on the failure mechanism. For instance, in a yield line crossing a reinforcement bar, the contribution to the dissipation of energy from the bar is the displacement discontinuity in the bar direction times the bar area times the yield strength as shown in some experiments by Nielsen.

3.8 State of art of TBM lining defects

During the construction of the TBM linings, inside the machine, some defects may occur; these possible problems are not always foreseen and accounted for during the design phase where it is common to make assumptions reflecting an ideal situation such as perfect gap fill, complete contact between the segments and perfect transmission of jack' forces.

To avoid undesirable concrete-on-concrete contact, during the construction phase, the segments must be assembled into complete rings with a high level of accuracy.

The JSCE, Japanese Society of Civil Engineers, has analysed and classified damages between 2001 and 2005 in over 50 construction sites. The JSCE [84] and [154] have classified the damages according to stage, category and cause of the defects. This classification is shown in the following table:

Table 3-1 Tunnel geometrical definitions.

Stage	Category	Definition	Cause
Design	Ground	D1	Stiff ground
		D2	Soft Ground
		D3	High hydraulic pressure
	Tunnel alignment	D4	Sharp curve
		D5	Sudden slope transition
	TBM	D6	Insufficient tail clearance
		D7	No articulated mechanism
	Segment	D8	Thinner segment
		D9	Wider segment
		D10	Division pattern of segment
		D11	Characteristics of joint
		D12	Characteristic of sealing material
Construction	Shield Tail	C1	Contact between tail and segment
	Segment installation	C2	Non-flat Joint
		C3	Open joint
		C4	Accuracy of segmental installation
	Others	C5	Eccentric grease pressure
		C6	Eccentric grout pressure
		C7	Improper tightening force of joint
		C8	Insufficient grouting

Within the “C” family of Table 3-1, JSCE classifies some construction issues that could cause damages. The authors, based on this table and on field observations, suggest implementing a defect classification in consideration of their theoretical effects, and sub-classifying them as global defects or local defects. In other words, some defects can produce changes of the distribution of the forces within the ring and other defects that could only change the local distribution of stresses [12].

Other Authors [87] and [89] have collected some studies on TBM defects principal on edges ruptures whose repair procedure was only to recreate the original segment geometry.

Cividini et al. [43] analysed the problem of the lack of back-filling for TBM segments reinforced solely with steel fibers. The repair procedure suggested by these authorse was to install steel ribs to reinforce the ring.

4 METHODOLOGY FOR THE CLASSIFICATION AND MEASUREMENT OF DEFECTS IN TUNNEL LINING CONSTRUCTION

4.1 Introduction

The methodology for classification and measurement of defects in tunnel lining construction presented in this chapter derives from the experience gathered from a real case study.

A series of nonconformities were detected during the TBM tunnel execution in the section Andorra–San Lorenzo al Mare [109]

Defects were observed in a significant percentage of the rings installed. After a first review and analysis of the tunnels and an assessment of the potential structural issues deriving from each defect, the following non-exclusive categories of defects were established:

- Ovalisation of the rings
- Misalignment between adjacent segments (longitudinal joints) and between rings (transversal joints)
- Joints openings
- Cracks in the concrete segments of the rings.
- Rupture of the edges between adjacent segments (longitudinal joints) and between rings (transversal joints)

The relevant feature of the measurement campaign described here is that some of the defects required a precision of some millimeters for a proper structural analysis.

The presence of water, dust and some utilities in the tunnels were a relevant limiting factor for field measurements.

The campaign was divided into two surveys: local mapping whose main objective was to measure joint openings, misalignments, edge ruptures and cracking and, global mapping whose main objective was to measure the global deformation of the ring (ovalisation) and thereby locate areas with deficient gap-filling.

The defects appeared during the observation of almost 7000 reinforced concrete segmented lining rings installed in 4 tunnels in Northern Italy as part of the new high speed railway between Milan and Ventimiglia. The tunnels were bored by a Herrenknecht TBM single shield. The geometrical lining definitions are collected in Table 4-1:

Table 4-1 Tunnel geometrical definitions.

Outer diameter	m	11.54
Segmental thickness	m	0.40
Thickness/outer diameter		1/28.85

The rings, with a length of 1.7 m, were designed to assume right and left positions within the tunnel (taped ring). The joint geometry between segments is plane, except at the key segment where a male-female geometry was designed.

The segments' thickness is 0.4 m while the contact height between joints is only 0.17m (see Figure 4-3). The segments were designed in reinforced concrete.

Figure 4-1 shows the four segment positions foreseen in the project. The reinforced concrete lining is composed by 7 segments, 6 ordinary segments and one keystone segment.

Figure 4-1 Positioning possibilities of the segments. From top left, position 1, 2, 3 (bottom left) and 5

During the excavation phase another position was added to those shown in Figure 4-1. This position, called position 4, intermediate to positions 3 and 5 was used to recalibrate the TBM head cutter during the construction process.

Table 4-2 shows the anti-clockwise angles of the joints with respect to the horizontal, for each position; so, for example, the joint between segments S5 and S6 in position 3 is exactly at the tunnel crown:

Table 4-2 Geometrical positions of the joints depending of the ring position

Joints	Position 1	Position 2	Position 3	Position 4	Position 5
K/S1	109°	52°	14°	161°	261°
S1/S2	52°	355°	317°	104°	204°
S2/S3	355°	298°	261°	47°	147°
S3/S4	298°	242°	204°	351°	90°
S4/S5	242°	185°	147°	294°	33°
S5/S6	185°	128°	90°	237°	336°
S6/K	128°	71°	33°	180°	280°

Figure 4-2 shows the frequency distribution of the positions throughout the database of acquired measurements.

Position	Frequency	%
1	1569	22.9%
2	1859	27.1%
3	2224	32.4%
4	14	0.2%
5	1188	17.3%

Figure 4-2 Histogram and frequencies of ring positions employed along the excavation of the 4 tunnels

The geometries of plane and male-female joints are shown in Figure 4-3.

Figure 4-3 Joints geometries

Figure 4-4 shows the evolution of ground cover along the four excavated tunnels. The first tunnel had 1976 rings, the second 1537 rings, the third 1806 and the fourth 1656 rings.

Figure 4-4 Tunnels longitudinal profile

The vertical alignment presented some local zones with low cover, between tunnel 1 and tunnel 2, which were solved through the execution of a hard fill embankment (Figure 4-5). The extension of these areas was 277 m (2.75% of the total alignment).

Figure 4-5 Hard fill Embankment area

The ground in which the tunnels were excavated was heterogeneous, alternating between marls, limestones, sandstones, conglomerates, over-consolidated and normally-consolidated clays.

4.2 Tolerances

Geometric tolerance of the segments is defined as the maximum geometric deviation permitted with respect to the theoretical design. Tolerances depend on the typology of the rings used and the positioning capacity of the TBM erector during the assembly phase.

In particular, the following aspects, as compiled by Gruebl [67], should be considered during the design process and should be clearly specified in the construction drawings.

- formwork tolerances
- tolerance for stacking, transportation and installation of the segment
- tolerances regarding gasket placements and connector positions
- tolerances in the segment execution procedure

Another factor to be taken into account is the assembly tolerance, not only of the segment, but also of the system, including the bolts, whose primary function is to hold the segments of the ring together before compression forces due to earth pressure develop.

Every major Tunnel Standards Institute has presented guidelines on tolerances, from AFTES [9] to BTS in [29],[30] and STUVA in [152].

In reality, as stated by other authors in [87] and [148], both the ring and the TBM are prototypes and so the tolerance values must be calculated on a case by case basis. Table 4-3 collects an update of the tolerances based on technical literature (see for [107] instance and [140])

Table 4-3 Tolerances for TBM lining updated by the author

Reference year		1998 [66]	2001[84]	2005[87]	2006[85]	2006[107]	2014 [107]
Width	mm	0.30	0.60	1.60	1.00	1.00	0.50
Angular deviation of the longitudinal joint	(°)	0.04	0.04	0.05		0.03	0.04
Thickness	mm	2.00	3.00	6.40	3:0	+3:-1	2.00
Circumferential length	mm	0.60	0.80	3.20	1.00	+3:0	0.60
Inner Radius	mm	1.50		3.20	2.00	0.15% of inner radius	
Flatness of the circumferential joint	mm	1.00	0.50	0.08	1.00	0.50	0.30
Flatness of longitudinal joint	mm	1.00	0.50	0.08	1.00	0.30	0.30
Gasket groove depth	mm			0.10		0.50	
Gasket groove width	mm			0.10	0.1:-0.5	0.50	
Position of the screw hole	mm	1.00		1.60	0.50	1.00	
Ovalisation	mm				5‰ of inner diameter	+/- 60 mm	

In conclusion, the idea of designing concrete elements with a certain tolerance stems from:

- Allowing proper segment assembly
- Establishing maximum deviations to be considered
- Establishing acceptance criteria for executed works

However, most projects do not provide a complete description of the tolerances to be used during the construction of the segment. In the case of the infrastructure considered in this paper, the tolerance for ovalisation was set at 110 mm, about 1.0% of the inner diameter, which is the allowable deformation for which it can be assumed that the soil behavior is elastic.

The tolerance for the vertical gap between the male and female parts of the keystone segment was set at 4 mm; this value was exactly the maximum vertical misalignment as shown in figure 4-3(right).

The tolerance of misalignments of plane joints was set at 10 mm. This value is the maximum one which would still avoid lack of compression at joint gasket.

The tolerance for joint opening was set at 0.5° , which is the precision of the manual instrument employed for the measures described afterwards.

4.3 Defects Encountered

As mentioned above defects were divided into global or local depending on whether the possible structural or durability effect of the defect is local or global.

Global Defects

Global defects are those that alter force distribution in the whole ring. Examples treated here are ovalisation and inadmissible cracking along the ring

Local defects

Local defects only alter forces in their vicinity normally at joints. Examples treated here are misalignments, joint openings and ruptures

Manual mapping was used to measure the local defects. This required a precision of some millimeters while automated mapping was used to measure the deformation of the rings.

4.3.1 Manual Mapping

After the tunnels' execution, local mapping was carried out in order to measure the following items:

- ✓ Misalignments
- ✓ Joint openings
- ✓ Ruptures
- ✓ Cracks

4.3.1.1 Misalignments

A misalignment refers to the presence of a radial movement of one segment with respect to its adjacent segment in the same ring (longitudinal) or with respect to its adjacent segment in the following ring (transversal) (see Figure 4-6).

Figure 4-6 Misalignments: longitudinal (left – compare with the theoretical joint geometry of Figure 4-3) and transversal (right)

To measure misalignments, in the local mapping, a calibre with a precision of 0.5 mm was employed. Each longitudinal measurement was taken at three points along the segment: at the beginning (M1) at the middle (M2) and at the end of the segment (M3). The notation given to the measurements is detailed in Figure 4-7.

Figure 4-7 Longitudinal misalignment measures

These measurements provided a good idea of the misalignments of the undeformed segment in space. As shown in Figure 4-3, the keystone has a concave/convex geometry which is needed to “close” the ring. In fact, the keystone is the last segment to be inserted during the ring assembly. The measurement procedure adopted for the plane joints (Figure 4-7) was not applicable for the male/female joints because of the different geometry. In order to solve the problem a new instrument was specifically conceived for this purpose, a Quadranfix® (which is an instrument conceived to measure angles) was altered and fitted with two curved steel bars, having the curvature of the ring (see Figure 4-10).

One of the curved bars was movable being fixed with two screws to one arm of the Quadranfix. The end screw remained fixed, while the central screw was fitted with a gauge to measure its displacement.

This altered instrument allowed to measure both misalignment and rotation. Perfect curvature of the segments would result in a measurement of 0° by the altered instrument (see top right hand side of Figure 4-8).

The measurement procedure was as follows:

- The fixed arm was fitted to the segment adjacent to the key segment
- Phase 1: the central screw was lowered until it touched the key segment. The movement of the screw was recorded and this value provided the measurement (ΔL) of misalignment between segments (Figure 4-8).

Measure of the misalignment ΔL by moving the central screw until it touches the key segment (at right)

Figure 4-8 Phase 1 Measure of Keystone Defect (rotation and misalignment)

- Phase 2: Then the rotation of the Quadranfix was liberated and the movable arm was fitted to the curvature of the adjacent segment. Once contact with the segment was achieved the angle (θ) was measured, providing the rotation between segments (Figure 4-9).

Adjustment of the moving part of the Quadranfix until it fits with the curvature of the key segment (at right). Measurement of the angle between fixed and moving parts of the Quadranfix (θ)

Figure 4-9 Phase 2 Measure of Keystone Defects

The Quadranfix® tests before field measurements are shown in Figure 4-10.

Figure 4-10 Quadranfix® provided with circular bars to measure misalignments and rotations of keystone joints

The measured angle is the result of the operation in Eq.(4-1)

$$\theta_{TOT} = \theta_{phase\ 2} + atan \frac{\Delta_{L(phase1)}}{L} \quad (4-1)$$

The precision of this instrument is 0.5 mm for misalignment and 0.5° for joint rotations.

Figure 4-11 shows types of edges ruptures which were observed during the measurement carried out in the field.

Figure 4-11 Possible edges ruptures observed

The combination of misalignment and joint rotation could create a theoretical overlap between the key-segment and the adjacent one. This situation leads to a rupture of the edge of the adjacent joint, normally compromising the unreinforced part of the segment as shown in the

Figure 4-16. This rupture does not affect the estimation of the rotation and misalignment since the Quadranfix has a total length of 120 cm.

4.3.1.2 Joint openings

Joint openings lead to unexpected stress distributions along the joint contact area. This different stress distribution can result in high local stresses. It can also compromise the gasket closure, and thereby, the water-tightness of the ring. Based on the rings observed, the joint opening is always combined with some misalignment.

Because of the particular female/male geometry of the key segments, the measurement of this defect was of special importance for key segments (see Figure 4-12). For this reason, the altered Quadranfix® instrument (as described above) was used in the key segments. However, this time-consuming procedure could not be generalized to all joints due to the large amount of measurement operations required and to the fact that the precision needed for analysis of the plane joints was much smaller.

Figure 4-12 Joint openings: front view (at left) and lateral viewing (at right)

In the cases of plane joints, the measure of joint rotations was based on simple trigonometrical concepts. As the joints are rigid solids in space, the joint rotation is the angle formed between the joint opening and can be determined from the penetration of a normalized steel bar as shown in Figure 4-9. In order to increase measurement accuracy, three normalized steel rods of different diameters were employed (2, 4 and 6 mm), the adopted rotation value being the mean value determined from the 3 normalized steel bars. The joint opening was measured with a calibre (see Figure 4-13).

Figure 4-13 Sketch of joint rotation calculations

Referring to above figure, the design contact surface is located between 115mm and 285mm from the surface of the segment (point A in Figure 4-13) and the theoretical opening of the joint is taken as 19 mm, at the point where measurements are taken (5 mm from the surface). The rotation of a joint can be determined from the surface opening of the joint (T) and the penetration length of the calibrated bar (p). The Surface opening can be determined from Eq. (4-2):

$$T = m - 19 \text{ mm} \quad (4-2)$$

where:

T is the surface joint opening (see Figure 4-13), T is equal to zero if there is no rotation

m is the measured joint opening (see Figure 4-13)

The rotation can then be determined using Eq. (4-3):

$$\theta = \text{atg} \frac{T-e}{p-5\text{mm}}; T-e \geq 0 \quad (4-3)$$

where

e is the thickness of the normalized steel bar (2, 4 or 6 cm) as shown in Figure 4-14

P is the measured penetration length as shown in Figure 4-13

5mm is the distance from the segment surface to the point where the joint opening is measured.

If the calibrated bars were to penetrate indefinitely this would mean that the joint was completely open. This configuration is particularly undesirable, since stability would only be

achieved through secondary elements not normally considered in the calculations such as provisional bolts. Figure 4-14 shows the measurement procedure in the field.

Figure 4-14 Joint rotation measurements: opening (at left) and penetration of normalized steel rod (at right).

4.3.1.3 Cracking

Cracks are defined as the presence of discontinuities in concrete along the ring. Some Authors [15] have studied the relation between cracking and jack forces during pushing operations.

In this case, the observed cracks, normally at the tunnel crown (at right of Figure 4-13 4-15) could be the direct result of the ring ovalisation because of lack of backfill.

The cracks were mapped, using color codes, depending on their widths, length and relative distance. In some cases, at left in Figure 4-15, the cracks were monitored with strain gauges and LVDTs to study their evolution in time and they were found to be stable.

Figure 4-15 Cracks at tunnel crown: mapping and monitoring (at left) and a bottom viewing (at right)

4.3.1.4 Ruptures

During the installation of the rings, because of the thrust of the jacks or due to a defect in the segment position, edge ruptures can occur at the joint zones (see Figure 4-16). These ruptures can happen along the longitudinal axis of the ring (in the same ring) or along the transversal

axis (between adjacent rings). Another possible cause for edge ruptures can be the lack of the effective geometric confinement at the contact between rings.

On the right part of Figure 4-16, the measurements taken in terms of length, width and height of the rupture are shown. The measure of the height was very important to establish whether the rupture affected the joint contact area, or not. If so, a local structural analysis was carried out to study the possible effect on the joint behavior. In the other cases, only cosmetic repair was needed.

Figure 4-16 Edge rupture at left and sketch with measurements carried out at right

4.3.2 Automated Mapping

As the local mapping was carried out employing essentially manual measures, the results of joint rotations were checked against global mapping. The global mapping was carried out employing the Leica® TMS system to survey the rings.

The main objective of global surveying was to study the ovalisation of the rings. Ovalisation refers to the deviation of the circular form of the ring. Mathematically it is determined as the difference between the maximum measured chord and the theoretical inner diameter of the tunnel. As stated by Baumann [22] ovalisation leads to the presence of segment offsets along the ring and can cause problems in the installation screws and bolts because the tolerances are very small. The ovalisation of a tunnel is rarely related to forces transmitted by the ground [98] and much more related to the general ring installation process [88] when this operation cannot be efficiently carried out. The ring, without the completion of the additional elements (bolts and screws) can be subjected to an extra deformation. This deformation is expedited in those cases where the gap filling operation, e.g. the grouting of the area between the TBM shield and the outer lining diameter, can last a time comparable to that of 3 to 4 ring installations. Figure 4-17 shows a typical shape of an ovalised ring.

Figure 4-17 Sketch of ring ovalisation over the undeformed ring

4.3.2.1 Ovalisation

In order to calculate the ovalisation, e.g. the maximum chord, the points of the survey were used. The maximum chord can be calculated as the maximum distance between the two given points.

The results, in terms of statistical distribution are shown in Figure 4-18, for all the observed rings. The average values of ovalisation are between 0.5% and 0.8% of the inner diameter, i.e. between 54mm and 84mm which is significantly less than the tolerance limit of 1%. However, 153 rings showed an ovalisation larger than the 1.0%.

Figure 4-18 Histogram of ovalisation ratio (maximum measured diameter divided by the inner diameter)

The instrument error is about 10mm, 0.09% of the inner ring diameter. Another error, associated with the ovalisation measure, can be a function of the defects in the tunnel shape. In fact, in some cases, the difference between the real profile and the theoretical profile can be affected by the presence of local ruptures at the joint areas.

These ruptures can locally alter the deformation value. Another factor that can alter the ring deformation measurement is the presence of the elements to accommodate the bolts and screws. In order to evaluate the error associated with the measurement of the ovalisation, some basic statistical techniques are employed. It is possible to see from Figure 4-18 that the ovalisation seems to follow a normal distribution with the parameters shown in Table 4-4.

Table 4-4 Ovalisation statistical parameters

Mean	70
Standard error	0.27
Median	68
Mode	78.4
Standard Deviation	22.1
Sample variance	488
Count	6857

If it is assumed, as seems reasonable, that the distribution is a normal distribution, then the error will also have a normal distribution so, the higher the value of ovalisation, the lower is the influence of the error on the measure.

4.3.2.2 Misalignments/rotations

The global survey allowed checking the local mapping results with the same points employed for the ovalisation measurements, in particular focusing on the misalignments and rotations of joints. The topographical survey allows exporting the points to ASCII format. From ASCII format these points have been introduced in a spreadsheet. The problem of misalignments and joint rotations can be solved by optimizing the survey points and adjusting them to the real position of the joints. The search for the best method was quite complicated because:

- The method should always be simple, constituted by a series of elementary routines that could be programed in the spreadsheet.
- The method had to provide rapid convergence because the 4 tunnels involve 20000 segments.
- Being an iterative method by nature, each step must provide a better estimate of the result than the previous one (convergence)
- The process must be the same for each observed ring.

Gander [60] gives an approach to fit a circle into an ellipse employing a non-linear optimization method based on the conjugate gradient method introduced by Hestenes and Stiefel [72].

The procedure employed can be summarized in the following steps (see [174], [108],[147],[46],[62] and [175]):

- ✓ Calculation of a reference center of the ring: the center of a circumference containing two of the opposite points with the given radius is calculated, for every combination of two opposite points, taken two by two. A cloud of centers is then obtained, and the center of gravity of that cloud is taken as the reference center of the ring.
- ✓ A second iteration is done for step 1: given the calculated center of the ring, the initial series of points is debugged by disregarding the points whose distance to the calculated center differs from the known radius by more than a certain tolerance.
- ✓ Identification of the segment to which each point belongs to. Taking the position of the reference center of the ring calculated in 1 and 2 and knowing which one among the 5 positions corresponds to the ring under analysis, each point is classified as belonging to a certain ring, depending on the angle with respect to the horizontal of the line that joins the point with the center (as the angles are known for the joints between segments). The points that are closer to the joint than a certain tolerance (chosen as 5°) are disregarded for the following calculations.
- ✓ Once the points are grouped in segments, the steps 1 and 2 are repeated only for the subgroup of points of each segment, so the center of each segment is calculated.
- ✓ Once the centers of each segment have been calculated, the intersection of segments is calculated. The angle between two adjacent segments is given by the angle of the radius of each segment at the intersection point. The misalignment can also be calculated. The line passing through the center of the ring and having an angle equal to the theoretical angle with the horizontal of the joint under analysis will cut the adjoining segments at different points. The distance between these two points provides the misalignment of the segments.

Once the theoretical centre of the ring is known, it is possible to determine the real centres of each segment. In this case, the position of the ring plays a fundamental role because the theoretical position of each joint depends of the ring's position as shown in Table 4-2.

4.3.3 Comparison with direct field data

In order to check the reliability of the method used for the calculation of the ovalisation described above, the results were checked against measurement using a laser-scanner. As the laser scanner precision is the very similar to the one of the topographical survey (TMS), the differences between the two methods have to be due to the different strategies in data analysis and in the different associated error estimation. The TMS topographical survey did not correct the data while with the laser-scanner data optimization was performed automatically, as explained in Nuttens [126] Han [69], and Yoon [173]. The results of the comparison are shown in Figure 4-19 .

Figure 4-19 Comparison between laser scanner and Global survey

The laser-scanner was used to scan the 12 most deformed rings among the almost 7000 installed. Figure 4-19 shows the differences between the two methods which can vary from approximately 1.2 mm to 25 mm, i.e. 0.2% of the inner diameter. In reality laser-scanner procedure does not seem more accurate than traditional TMS, because of the presence of water leakage and dust in the tunnels which alters laser scanner measurements. In order to achieve better precision, the laser scanner needed more than one survey for the same ring. In summary, both methods have an associated error between 0.09 and 0.20% of the inner diameter which can be considered as acceptable.

Regarding joint rotations, it is possible to compare, for instance, the joint rotations of segments at the tunnel crown for positions 3 and 5 measured by local mapping and using the optimization carried out based on the global survey (TMS). The values show good correlation as can be seen in Figure 4-20.

Figure 4-20 Comparison between global survey and local mapping for joint rotations at tunnel crown (pos 3-5)

The values of the measurements of longitudinal misalignment are presented in Figure 4-21. In 95% of the cases the value of misalignment is greater than the established tolerance (10 mm).

Figure 4-21 Longitudinal Misalignments

Figure 4-22 shows the crack opening for all the observed rings. In 10 % of the cases the value is greater than 0.4 mm. Large cracks were shown to be due to the fact that minimum reinforcement was determined in the project assuming a minimum compression force which is not always present, particularly during the construction phases. Cracks larger than 0.4 mm had to be injected in order to guarantee adequate durability.

Figure 4-22 Crack openings (mm)

The joint opening, expressed in degrees, for plane joints is shown in Figure 4-23. In 98% of the cases the value is greater than that established tolerance (0.5°).

Figure 4-23 Plane joints openings

Figure 4-24 shows the keystone openings, in terms of degrees, for keystone joints. In 80% of the cases the value is greater than the established tolerance (0.5°).

Figure 4-24 Keystone joints openings

Despite the figures given above, regarding the fact that in a large number of the measured sections defects exceeded the tolerances allowed for construction, analysis showed that only in a small number of cases was it necessary to reinforce the structure. The only visible indication of structural distress were the large cracks shown in Figure 4-15. Regarding these cracks it was shown that their magnitude was due to the fact that, in design, minimum reinforcement was determined assuming a minimum compressive force which was not present during the construction phases, so that, in absence of this force, yielding of the reinforcement would occur for the cracking moment, thereby leading to the large cracks. During construction, the forces were redistributed (see reference [78]) thereby not resulting in problems regarding the safety of the structure, even though large cracks (greater than 0.4 mm) had to be injected.

5 METHODOLOGY FOR THE ANALYSIS OF TUNNEL LININGS UNDER THE HYPOTHESIS OF LACK OF BACK-FILLING

5.1 Introduction

The definition of the ovalisation is given in paragraph 3.4 and although different authors ([1] [2]) have already studied the problem (see page 9), their hypothesis did not always capture the complexity of the problem, in particular referring some questionable assumptions are::

- The ring is loaded radially [155],
- The gap between rock and lining is completely filled
- Concrete behaviour is always linear elastic

With these hypothesis, ovalisation plays a negligible role.

These hypotheses are not fulfilled in absence of a significant axial force or when the gap is not yet filled, for example during construction phases. In this case the only bending capacity to support the weight of the segment is provided by bolts placed at segment joints

The main issue in engineering practice is the magnitude of the deformation when the load is not radial, in the absence of a significant axial force acting on the ring, and when the ring, because of the lack of gap-fill is free to expand. In this case, the deformations and the stress distributions are completely out of range when comparison is made between the field data and those values calculated with the models available in technical literature and summarized above because all of them take into consideration that the ring is perfectly confined and immersed into the soil: in other words, the ring displacements because of ovalisation would be very small.

In this chapter, a new approach is presented and based on non-linear behaviour of R.C. and on the constitutive laws of the materials which leads to a redistribution of the internal forces, in

particular of bending moments, and considers provisional bolts which are actually responsible for the achievement of equilibrium in the unfavourable conditions typical of construction phases.

5.2 Field Measurements

Field measures are based on the real case described in chapter 4 and they are related to ovalisation and cracking.

Ovalisation:

The usual approach in technical literature assumes ring ovalisation as a load function of the overburden pressure (see for instance [14] and [23]), which means that for shallow tunnels ovalisation could be observed more often as the surrounding soil is not able to generate enough confined pressure. Other authors (see for instance [3] and [4]) related ovalisation with poor geotechnical conditions which means that a tunnel in fault area, for example, could present higher ovalisation values rather than a tunnel in hard rock.

The observed ovalisation does not depend on cover nor on geotechnical parameters since most of the deformation happens before the soil loads intervene as shown in Figure 5-1, where there is no apparent correlation between ovalisation and cover.

Figure 5-1 Measured ovalisation (mm) versus Tunnel cover (m)

Cracking:

It was observed that the typology of observed ovalisation produced also a general cracking along the ring.

A characterisation of the observed cracks widths has been made for each tunnel as shown in Table 5-1. The maximum values of crack width indicate that yielding of the steel has taken place:

Table 5-1 Cracks widths along the four tunnels

	Average Crack Width	Min crack width	Max crack width	Standard deviation
	mm	mm	mm	
Tunnel 1	0.191	0.050	13.000	0.273
Tunnel 2	0.167	0.010	3.300	0.169
Tunnel 3	0.178	0.005	4.000	0.199
Tunnel 4	0.191	0.005	5.000	0.192
Average	0.182	0.018	6.325	0.208

Referring to the data collected in chapter 4, one of the ring has been chosen, the maximum ovalisation measured for chosen ring is 199 mm and the following cracks have been detected at the crown of the above-mentioned ring. The ring will be used to illustrate the proposed analysis methodology.

Table 5-2 Cracks at crown at the maximum ovalized ring

Crack N°	Width	Crack N°	Width
	mm		mm
1	0.10	7	2.20
2	0.15	8	0.10
3	0.20	9	0.90
4	0.30	10	0.50
5	0.70	11	0.20
6	0.70	12	0.10

5.3 Non-linear analysis

In the deformed rings studied, crack patterns such those collected in Figure 4-15 are typical. The stresses in the steel re-bars, measured with a strain gauge on field, showed values up to 400 MPa, when cutting the steel bar. Indeed, it is worth pointing out that a kinematic model of the structure shows how a small angle offset (0.5%) at one joint during ring installation results in a displacement equal to 45mm, e.g 0.4% of the inner diameter.

In reality, the measured displacements are much larger than 45mm as shown in Figure 5-1. A non-linear analysis was carried out. The segments are modelled as non-linear frame elements, while the back-filling material is modelled by means of linear springs. The joints between segments have been simulated as non-linear frame elements with a section of steel at the middle of the cross section, representing the bolts.

Figure 5-2 Non-linear model

The joint geometry is plane between segments and has combined (convex/concave) geometry at the key-stone elements [6]. In the analysis, the joints have been simulated as shown in the following figure:

Figure 5-3 Joint geometry

The structural elements considered in the analysis have the following geometrical characteristics and uncracked mechanical properties:

Table 5-3 Geometrical characteristics and gross mechanical properties of structural elements

	h (m)	A (m ²)	I (m ⁴)
Segments	0.4	0.4	5.33x10 ⁻³
joints	0.17	0.17	4.09x10 ⁻⁴

In order to limit the deformations, the extension of the saddle (usually measured in degrees) takes on a fundamental importance in the problem definition. In fact, the greater the extension of the saddle area, in phase 1 of Figure 3-4, the smaller the displacements which appear along the ring.

Figure 5-4 Ring saddle (80°, split in in 40°+40°). Bolts are visible between segments

Figure 5-4 shows the positions of bolts whose main function is to “close” and ensure proper ring geometry of the ring from phase 1 to phase 3 of Figure 3-4.

The project, object of this study, provided a theoretical extension of saddle zone of 80° divided in two symmetric portions of 40°, as shown in Figure 5-4.

The first step in the analysis was to recreate the theoretical conditions. The length of the saddle ring zone has been modelled using vertical springs. The springs can be derived from the subgrade modulus. There are different approaches to estimate the subgrade modulus, K, for a tunnel. Among them, it is possible to use the following two formulas:

- Formula based on Galerkin’s approach [28]:

$$K = \frac{E}{R(1 + \nu)} \quad (5-1)$$

where:

K= subgrade modulus

E= Young modulus of the soil.

R= radius of the tunnel (5.57m)

ν = Poisson ratio (0.2)

- Formula By Bowels, based on Vesic's approach [29]

$$K = \frac{E}{B(1-\nu^2)} \quad (5-2)$$

where:

E= Young modulus of the soil

B= equivalent foundation width [see for instance page 509 of reference 29]

ν = Poisson ratio

To evaluate the equivalent subgrade modulus along the saddle area, three different hypotheses were made.

- The subgrade modulus is a function only of the mortar's Young's modulus (evaluated at 30GPa). The thickness of the mortar is 0.125m which is the gap between the rear of the TBM and the surface of the lining.
- The subgrade modulus is a function of the Young's modulus mortar and that of underlying soil. The thickness of the equivalent layer is 0.25m which is two times the gap thickness. Each layer is considered with a depth of 0.125 m
- The subgrade modulus is a function of the Young's modulus of the mortar and that of the underlying soil. The thickness of the equivalent layer considered in this case is 1.5m which is 12 times the gap extension. The mortar layer is 0.125m deep and the underlying soil is 1.375m.

Each of three hypotheses was examined using both Galerkin's and Bowels' approach. In summary 6 calculations were made. Following Bowels [29], the Young modulus for a layered soil can be expressed as:

$$E_{tot} = \frac{E_1 H_1 + \dots E_n H_n}{(H_1 + \dots H_n)} \quad (5-3)$$

where:

E_{tot} = layered Young's modulus

$E_1 \dots E_n$ = Young's modulus of layer 1...n

$H_1 \dots H_n$ = thickness of layer 1...n

Once the subgrade moduli have been obtained for each one of the six possible combinations, each value has been multiplied by the equivalent frame area (0.157m) and the subgrade modulus, for each frame, has been obtained as shown in Figure 5-5 where it is possible to see how the difference between the various subgrade moduli can be six times (model 1 / model 6). Table 5-4 collects all the subgrade moduli employed in non-linear calibrations. Figure 5-5 shows the relation between thickness of the equivalent layer and the subgrade modulus. Model

1, for example, corresponds to subgrade modulus following Galerkin's approach with an equivalent thickness of 0.125 m while model 6 corresponds to Bowles's approach with an equivalent thickness of 1.5m.

Table 5-4 Different Subgrade scenarios

Approach	Model	Thickness m	K MN/m ³	Area m ²	Subgrade modulus MN/m
Galerkin	1	0.125	4488	0.157	704
Galerkin	2	0.25	2598	0.157	408
Galerkin	3	1.5	1023	0.157	161
Bowles	4	0.125	4027	0.157	632
Bowles	5	0.25	2331	0.157	366
Bowles	6	1.5	918	0.157	144

Figure 5-5 Subgrade moduli variation

Figure 5-6 shows the values of the bending moments as function of the tunnel angle (0° at crown and 180° at invert arch) for the two different subgrade moduli (model 1 and model 6). It is possible to see how the difference in terms of bending moments is about 9% at the crown and 21% at invert arch. These differences are rather small, in spite of the significant difference in the value of the subgrade modulus.

Figure 5-6 Bending moment along the tunnel perimeter Subgrade moduli variation

It was finally decided to employ a value of subgrade modulus equal to 250 MN/m, because, within the estimated range, it better simulates the plastic zone around the tunnel. The model is shown in Figure 5-7.

Figure 5-7 Model employed in analysis (at top) vs real ring (at bottom)

As it is visible in Figure 5-7, two cross sections were employed:

- Cross section of the segment with a height of 0.40m of concrete and steel rebars
- Joint section, with a height of 0.17m and a steel bolt exactly at the centre of the section

Each segment, in fact, is connected to the others along the longitudinal joint with two longitudinal bolts (along the same ring) and along the transverse joints also with transverse bolts (with the adjacent ring).

Each bolt is 25 mm diameter with tensile strength of 1040 MPa and a length of 0.520 m. The insertion angle is 30° (Figure 5-8).

The corresponding stress normal to the cross section, $f_{d,n}$ is fixed at 700 MPa. The joint section with the concrete in compression and the bolt in tension works as a rotational spring with elasto-plastic behaviour.

Figure 5-8 Bolt section employed in non-linear analysis

It is important to underline that the main function of the bolts, mainly the one placed along the longitudinal joints, is to connect all the segments within the same ring, providing enough resistance to balance the ring's self-weight in absence of a significant axial force.

This is the main reason why bolts must be taken into account during the verification of the construction phases. The bolts were modelled with a frame element whose cross section is shown in Figure 5-8 (left).

Constitutive laws, with average values for concrete and steel bars have been employed. Average values, instead of characteristic ones have been taken because the present study is a back-analysis of the structure behaviour to explain the excessive cracking.

For this reason, partial factors for materials have been set to 1.0. Concrete class is C45/55 with Young's modulus of 37.000 MPa.

The constitutive laws used area shown in Figure 5-9, Figure 5-10, Figure 5-11 and Figure 5-12.

Figure 5-9 Concrete (joint) constitutive law with no tensile strength

Figure 5-10 Concrete (segments) constitutive law with tensile strength

Figure 5-11 Steel employed in the segments (B-500SD) constitutive law

Figure 5-12 Bolt steel constitutive law

The methodology of the analysis carried out is based on [134] and it consists in the following steps:

- First of all, a linear analysis is carried out to determine the internal force envelopes along the ring.
- For each section of the model, the bending moment is compared to the yielding moment. The yielding moment has been evaluated as 128 KNm (as shown in Figure 5-13)

Figure 5-13 Moment curvature diagram

- When the computed bending moment is greater to than the value established for yielding (128KNm), a hinge is formed at that location, carrying a bending moment equal to this

value. In this way, internal forces redistribute along the ring. This artifice allows to reproducing the response of a structure under bending moments when yielding is reached.

- This process has to be repeated until bending moments within the ring are less than the yielding moment or until the structure has collapsed (formation of 5 plastic hinges).

Three scenarios have been reproduced varying the extension of the saddle areas and are defined in Table 5-5.

Table 5-5 Scenarios analysed depending of the saddle extension

Scenario	saddle extension	
	(°)	
A	40+40
B	40+30
C	40+20

These scenarios are the results of the observations in the field where, in some cases, it was not possible to create the designed saddle as per phase 1 of Figure 3-4. For each scenario, the internal force redistribution was reached in different steps.

5.4 Model Results

The results have been compared with field measurements carried out after the tunnels have been excavated.

The measured cracks are located at the crown, and the internal forces at these sections are given in Table 5-6:

Table 5-6 Summary table among the different scenarios and their evolution after the redistribution steps

	Description	N _{crown}	M _{crown}	Δ _{TS}	σ _{s2}	Crack at crown	Ovalisation
		kN	kNm	mm x10 ⁻⁴	MPa	mm	mm
A1	Model "0" (no hinge)	-1.1	60.1	5.19	272.5	0.3	11.0
A2	Model two hinges	-2.5	65.7	5.69	298.7	0.3	13.0
B1	Model "0" (no hinge)	6.0	86.3	7.68	403.4	0.4	17.0
B2	Model one hinge	11.5	99.5	8.95	470.0	0.5	35.0
B3	Model two hinges	14.9	121.6	3.39	573.6	1.0	56.0
C1	Model "0" (no hinge)	9.0	94.2	8.50	446.4	0.4	33.0
C2	Model one hinge	32.0	152.0	-	-	-	74.0
C3	Model two hinges	9.9	225.0	-	-	-	-
C4	Model three hinges	62.3	466.0	-	-	-	-
C5	Model four hinges	27.0	128.0	4.65	600.0	4.1	151.0

According to [57] it is possible to obtain the crack spacing as:

$$S_{\max} = 2(c + \frac{1}{4} \frac{f_{ctm} \phi_s}{\tau_{bm} \rho_s}) \quad (5-4)$$

where

φ_s= steel bar diameter (10mm)

ρ = effective reinforcement ratio (A's/A_{c,eff}) (0.005585)

f_{ctm} is mean value of axial tensile strength of concrete

τ_{bm} is mean bond strength between steel and concrete

The maximum crack width can be determined from:

$$w_{\max} = S_{\max} (\varepsilon_{steel} - \Delta TS) \quad (5-5)$$

where

ΔTS is the reduction in steel strain due to tension stiffening and defined as:

$$\Delta TS = \beta \frac{\sigma_{sr}}{E_s} \quad (5-6)$$

$$\beta=0.6$$

σ_{s2} is bare steel stress at the crack

$$\sigma_{sr} = \frac{f_{ctm}}{\rho_{eff}} (1 + \alpha_e \rho_{eff}) = 502.3 MPa \quad (5-7)$$

σ_{sr} is the bare steel stress due to the cracking moment

α_e is the modular ratio

By comparing Table 5-2 with the results obtained in Table 5-6 , it is possible to observe how the maximum measured crack, 2.2 mm is something intermediate between models B and C although the measured crack width would lead to steel yielding, and the crack width formulation would not be really applicable.

In fact, it would tend to underestimate the crack width. This can provide an order of magnitude and help to draw conclusions. The second maximum crack, 0.9 mm, collected in table 4 is very similar to Model B. Model A seems not to represent the reality in terms of deformations or cracks widths.

In Table 5-6, it is possible to check the evolution of the internal force redistribution when a hinge is inserted in the model. Model “A” reaches a maximum crack width of 0.3 mm and the ovalisation seems to be constant with a value of 13mm.

Model B reaches a maximum crack value of 1.0 mm at the tunnel crown while the ovalisation increases from 17.0 mm with model “B1” to 56.0 mm with model “B3”.

Model C assumes the formation of a series of hinges, during the redistribution process, where the bending moment is higher than the moment producing yielding of the reinforcement one until “C5” where equilibrium is reached at all sections. For “C5” the ovalisation is of the same order of magnitude as the measured one (199mm).

It is possible, in the following figures, to recreate the redistribution of the internal forces during the various steps whose plastic hinges form in the sectors whose the bending moment is larger than the yielding bending moment. The horizontal lines correspond to the yielding bending moment (128KNm):

Figure 5-14 Model "A" internal forces redistribution

Figure 5-15 Model "B" internal forces redistribution

Figure 5-16 Model "C" internal forces redistribution

6 FORCE TRANSMISSION BETWEEN SEGMENTS: DESIGN OF THE EXPERIMENTAL CAMPAIGN

6.1 Purpose

As stated in the introduction, the first effect of the construction loads is a non-uniform deformation, defined as ovalisation, within the ring. The geometrical offset between segments that, at this stage can expand, changes the contact area at joint sections producing rotations, misalignments and other defects described in the chapter 4.

Moreover, the joints sections are designed to assume high pressures and, in some cases this pressure can lead to a confinement pressure much higher than maximum value established in the Standards, i.e. around $3.0 f_{cd}$.

It is worth remembering that reinforced concrete elements need to be designed to satisfy both Ultimate and Serviceability Limit State criteria. Normally the joints sections are verified only under ULS (ultimate limit state) and no reference is made to SLS (serviceability limit state). In particular the Standards do not clarify the cracking criteria for D-regions. The only references encountered in literature are based on an indirect control of cracking. This control must be carried out limiting the maximum tensile strength of the reinforcement bars. In particular two references can be cited:

- Leonhardt [95] who prescribed a maximum allowable tensile stress in steel reinforcement of about 180 MPa.
- Spanish standard [118] which prescribes a limitation of tensile strength in steel reinforcement of about 400 MPa.

The presence of a defect, in the contact area, can reduce A_{c0} providing an increment of pressure to be transmitted and a possible increment of cracking.

This situation leads to the following questions:

- 1) Is the confinement factor for partially loaded area sufficiently defined?

- 2) Is there a way to define a limitation of cracking for SLS for partially loaded areas?
- 3) Does the bar size affect the crack pattern in partially loaded region?

The purpose of these experimental tests is to attempt to answer these questions.

It is important to underline that several tests have already been carried out on partially loaded areas and some of them have been examined in the section 3.5.2.1. Previous experimental tests were employed to properly design the tests campaign described in the following sections.

6.2 Tests Programme

The aim of the experimental campaign was to recreate the joint geometry shown in Figure 5-3.

For this a concrete block of 50x40x50 cm³ was concreted and placed on a series of independent neoprene bearings, so that there would be no friction at the base which could lead to an unwarranted increase in resistance. On this block, the 40 cm dimension represents the segment height (direction transverse to the tunnel axis).

The ratio between the bearing width in the transversal direction (0.17 m) and the concrete block dimension (0.4 m) is taken from the project and provides a ratio of 0.47/0.4=0.425 which is a typical section reduction at joints for this kind of structure. The target characteristic concrete strength in the tests was 25 MPa.

The choice for this moderate value of the resistance was made to maximize the probability of obtaining compression failure with the available laboratory equipment even for high confinement ratios.

Two sets of tests were carried out. In all cases, the necessary concentrated load reinforcement was determined using a strut & tie model according to Eq.6-1.

$$A_s f_{yd} = 0.25 \times P_{Ed} \left(1 - \frac{a_1}{a} \right) \quad (6-1)$$

where:

A_s	is the reinforcement in the given direction
f_{yd}	is the design yield strength of reinforcing steel
a_1	is the plate dimension in the direction being considered
a	is the block dimension in the direction being considered

Since the aim of the experimental program was to study how much compressive stress could be transmitted by the plate to the concrete before failure and the loading frame had a capacity of 5000 kN, it was decided to design the reinforcement for a factored load of 6750 kN using Eq. 6-1, so that it would be guaranteed that failure would come about by compression failure at the plate.

The plate dimensions used were, 170x500 mm² which would provide no lateral confinement in the longitudinal direction, 170x250 mm² and 170x80 mm².

Another two tests were carried out for the intermediate plate dimensions of 170x250 mm² but varying the distribution of reinforcement by using 8 mm bars and 16 mm bars instead of 12 mm

bars which were used for the reference specimen C-25-25. This was done to try to study the effect of reinforcement detailing on both failure load and cracking.

Figure 6-1 shows the reinforcement detail of specimen C-25-25.

Figure 6-1 Reinforcement detailing

Ten one on one scale tests were performed employing the following materials:

- Mean strength of concrete 23MPa
- Steel bars reinforcements B500SD
- The size of reinforcement bars: Ø8, Ø12 and Ø16 mm
- The support of the blocks is constituted by a series of neoprene blocks of dimensions 15mm x 15mm x 50mm.

The nomenclature employed is collected in table 6-1:

Table 6-1 Tests Definition nomenclature

Test	Objective	Number of Tests
BF_25_8	Cracking	2
BF_25_12 ¹ (C_25_25)	Cracking/confined Concrete	2
BF_25_16	Cracking	2
C_25_8	Confined Concrete	2
C_25_50	Confined Concrete	2

The specimens were coded as follows:

- C for compression or BF for cracking. The BF tests varied the bar diameter while maintaining the reinforcement area to study its influence in cracking. All C tests were detailed using 12mm bars
- 25 represents the target value of concrete resistance in MPa
- In C tests, the third code corresponds to the width of the bearing plate in cm.
- In BF tests, the third code corresponds to the bar diameter used to detail the reinforcement. For the BF tests, the bearing plate width was taken as 250 mm.

The reference test is C25-25 with a plate width of 250 mm and a diameter of 12 mm.

The primary objective of the cracking tests was to study the crack patterns in D regions and the influence of diameter of the steel reinforcement bars, keeping all other features constant. Three different configurations were analysed:

Table 6-2 Reinforced bars for cracking tests

	A Direction (400mm)	B Direction (500mm)	Size bars employed
BF_25_8	170 mm	250 mm	Ø8
BF_25_12 (C25-25)	170 mm	250 mm	Ø12
BF_25_16	170 mm	250 mm	Ø16

Tests BF_25_12 and C25_25 are the same. All the tests were equipped with strain gauges along the tensile reinforcement bars and extensometer bases.

The details of the reinforcement design are given in Table 6-3. The reinforcement has been determined assuming that the stress in steel is 400 MPa at failure.

¹ This test is a reference test and has two denominations depending on the objective of the test.

Table 6-3 Dimensioning of the block reinforcement for concentrated load

Specimen ID	P_{Ed} [kN]	a [mm]	b [mm]	a_1 [mm]	b_1 [mm]	$A_{s,t}$ [cm ²]			$A_{s,l}$ [cm ²]		
C-25-50	6750	400	500	170	500	24.26	24 ϕ 12	27.12	0.00	12 ϕ 12	13.56
C-25-25	6750	400	500	170	250	24.26	24 ϕ 12	27.12	21.09	24 ϕ 12	27.12
C-25-8	6750	400	500	170	80	24.26	24 ϕ 12	27.12	35.44	30 ϕ 12	33.9
BF-25- ϕ 8	6750	400	500	170	250	24.26	48 ϕ 8	24.14	21.09	48 ϕ 8	24.144
BF-25- ϕ 16	6750	400	500	170	250	24.26	12 ϕ 16	24.12	21.09	12 ϕ 16	24.12

6.3 Test analytical design

The introduction of a pressure into a concrete block can be treated analytically considering a uniaxial problem in which the load is applied along a strip and the concentrated force so spreads out until uniform compression within the area of the concrete block, is achieved.

It is important to underline, at this stage that not all the force has to be spread out since a part of it, which is already distributed within the bearing width (A_{c0}) can follow a straight path.

Figure 6-2 Strut and Tie for concentrated load [21]

Referring to Figure 6-2, the part of this force per meter of length that does not need to be deflected, N_{sd}^* , can be determined from the following equation:

$$N_{sd}^* = \sigma_{sd} a_1 \quad (6-2)$$

So, the force per meter of length to be deflected is:

$$N_{sd} - N_{sd}^* = \sigma_{sd}(a - a_1) = N_{sd} \frac{a - a_1}{a} \quad (6-3)$$

Force $N_{sd} - N_{sd}^*$ which located at the centre of the block, has to be transferred to a distance of $a/4$ from the centre of the block. It is assumed that this is accomplished in a distance equal to the width of the block, the centre of gravity of the force being located at $a/2$.

More generally a certain diffusion angle α_{mod} can be assumed. The value of the tangent of this angle, if the diffusion distance is taken as a , as in the case represented in Figure 6-2, would be $\tan \alpha_{mod} = 0,5$.

The tensile force generated by the spreading out of the compression stresses can be expressed as in the following equation:

$$T_{sd} = \frac{N_{sd}}{2} \tan \alpha_{mod} \left(\frac{a - a_1}{a} \right) = \frac{N_{sd}}{2} \tan \alpha_{mod} \left(1 - \frac{a_1}{a} \right) \quad (6-4)$$

For a three-dimensional block and a load limited in the longitudinal direction, with a bearing plate of dimensions $a_1 \times b_1$ and a concrete block of dimensions $a \times b$, the eq. (3-9) can be generalized as:

$$T_{sd} = \frac{N_{sd}}{2} \tan \alpha_{mod} \left(1 - \sqrt{\frac{a_1 b_1}{ab}} \right) \quad (6-5)$$

Given sufficient tensile reinforcement, two failures modes should be checked in a partially loaded area:

- Triaxial compression
- Uniaxial compression

Blowout, i.e. lateral failure, is not checked such this checking is normally carried out for partially loaded areas in headed studs.

Both failure modes are expressed in the following formulas:

$$N_{sd} \leq N_{1d} = \min \left(\sqrt{\frac{ab}{a_1 b_1}}; k_{ct} \right) a_1 b_1 f_{cd} \quad (6-6)$$

The value of k_{ct} is that already defined in the section 3.5.2, and will be evaluated for the results of these tests.

Concrete cover was setup as 0.05 m, i.e. 0.1 times the height of the block as per [95].

6.4 Specimen construction

The execution of the specimens was carried out at the laboratory of structural concrete at the Technical University of Madrid.

6.4.1 Concrete

Concrete pouring was conducted by the laboratory technicians, the author and other Phd students. The concrete batch was received on 14th of March 2016. The slump was 50mm resulting a consistency class S2 in accordance with EN206-1 [45].

Figure 6-3 Abrams con (at left) and vibration of concrete (at right)

The results of concrete strength are presented in Table 6-5:

Table 6-4 Measured concrete compressive strength (uniaxial tests)

Compressive strength (MPa)					
Day	Tests	#1	#2	#3	Mean
28	3	19.8	16.8	19.8	18.8
90	3	23.0	23.0	23.0	23.0

Figure 6-4 shows the evolution of the mean compressive strength of the concrete:

Figure 6-4 Evolution of the concrete compressive strength employed in the tests

The target concrete was class R with cement content was 275 kg/m^3 and water-cement ratio 0.5. Maximum aggregate size was set at 20 mm.

6.4.2 Steel

Steel bars employed in the tests were provided on 10th of march 2016. They were made of standard B500SD and nominal yield strength of 500 MPa.

In order to move the RC concrete block two additional steel bars were installed within the block. To guarantee the concrete cover four sets of spacers were provided before pouring of concrete (see Figure 6-5):

Figure 6-5 Spacers installation before concrete pouring (at left) and moving bars to move the concrete block (at right)

6.4.3 Monitoring

The batches were equipped with monitoring to register the strains and load for each load step.

6.4.3.1 Internal monitoring

The installation of strain gauges was regarded as an appropriate and simple procedure to determine the stress in steel reinforcement during the tests. These data is considered fundamental to study the behaviour of the specimens.

The gauges have a pre-attached vinyl lead wire and an entire coating with epoxy resin, which is transparent and flexible, thus ensuring an easy installation of the gauge, and adequate water tightness.

Furthermore, the coating provides some degree a physical/mechanical protection during the pouring and vibration of concrete.

A preliminary preparation of the surface is needed: the main rib is smoothed with a circular saw and carefully cleaned avoiding the presence of debris.

Perfect waterproofing is achieved by bonding the gauge with CN (Cyanoacrylate) adhesive, obtaining a final configuration as shown in the Figure 6-6:

a) Strain gauge

b) Strain gauge in water

c) gauges locations

Figure 6-6 Strain gauges installation

The strain gauges were attached to the steel bars as shown in Figure 6-6.

The procedure followed for this was:

- 1) test the strain gauge to confirm that the resistance is that of the catalogue
- 2) smoothen the rebar at the zone where the strain gauge is to be installed
- 3) smear a bonding adhesive on the smooth surface
- 4) paste the strain gauge and apply pressure manually during 30 seconds.
- 5) retest the strain gauge to check if the process has not damaged the internal resistance by remeasuring the value of the elastic resistance.

In order to check the buoyancy of the process the steel bars were left 48 hours in water and then tested to recheck the resistance.

This process was repeated during all the ten batches preparation.

Table 6-5 Strain gauge features characteristics.

	Manufacture	Gauge	Gauge Features			
			L	W	Resistance	Gauge Factor
			mm	mm	Ω	
BF_25_8 specimen 1	HBM	1-LE11-3/350Z	3	2.0	350	2.03
BF_25_8 specimen 2	HBM	1-LE11-3/350Z	3	2.0	350	2.03
BF_25_12 (C25_25) specimen 1	Tokyo Sokki Kenyujo Co., Ltd	WFLA-6-11-1L	6	2.2	120	2.12
BF_25_12 (C25_25) specimen 2	Tokyo Sokki Kenyujo Co., Ltd	WFLA-6-11-1L	6	2.2	120	2.12
BF_25_16 specimen 1	Tokyo Sokki Kenyujo Co., Ltd	WFLA-6-11-1L	6	2.2	120	2.12
BF_25_16 specimen 2	Tokyo Sokki Kenyujo Co., Ltd	WFLA-6-11-1L	6	2.2	120	2.12
C8_C specimen 1	HBM	1-LV41-3/120A	3	1.1	120	2.00
C8_C specimen 2	HBM	1-LV41-3/120A	3	1.1	120	2.00
C50_C specimen 1	HBM	1-LV41-3/120A	3	1.1	120	2.00
C50_C specimen 2	HBM	1-LV41-3/120A	3	1.1	120	2.00

Table 6-6 shows the characteristics of the strain gauges used. Different types were used to delivery and schedule constraints.

Strain gauges are identified with the code SG-#. Unfortunately for C25_50 tests, those installed near the bearing load, i.e. the highest ones, were irretrievably damaged in spite of the care taken during the pouring procedure and it was not possible to measure strains within these bars.

6.4.3.2 External instrumentation

After pouring, further instrumentation was provided in order to supply a more complete information about the specimen behaviour.

A series of mechanical strain gauges were attached to the concrete surface. Strains could be then measured at each load stage by means of mechanical extensometer.

The horizontal targets were installed in correspondence with the steel bars levels while two vertical targets were installed on each block face in order to correlate the internal and external deformation of the concrete during the load steps as shown in Figure 6-7:

Figure 6-7 Mechanical strain gauges installed during a test (extensometer base is artificially painted in the figure)

6.5 Test setup

The test was carried out in the Structure's laboratory of the Civil Engineer School of the Technical University of Madrid. Each batch was tested in a hydraulic press capable of developing a nominal ultimate load of 5000 kN.

The bottom connection between the batch and the load press was solved by placing 24 neoprene bearings of 150 mm x 150 mm x 50 mm as shown in Figure 6-8.

The top connection between the batch and the press load was solved with two steel plates 50 mm while their dimensions in place depended on the loaded area considered. The load steps were of 500 kN except for C25_50 series which steps were of 200 kN.

Figure 6-8 Neoprene base (at left) and plate employed as loaded area (at right)

7 EXPERIMENTAL RESULTS

In this chapter, the results of the tests are shown clarifying the principal features that have been studied and compared with non-linear finite element analysis.

7.1 Tests results

The order in which the specimens were tested is given in Table 7-1

Table 7-1 Dates of tests

	Tested date		Tested date
C50_C specimen 1	02/06/2016	BF_25_12 (C25_25) specimen 2	14/06/2016
C50_C specimen 2	07/06/2016	BF_25_8 specimen 1	16/06/2016
C8_C specimen 1	10/06/2016	BF_25_8 specimen 2	16/06/2016
C8_C specimen 2	10/06/2016	BF_25_16 specimen 1	16/06/2016
BF_25_12 (C25_25) specimen 1	14/06/2016	BF_25_16 specimen 2	16/06/2016

The observed failure was brittle with general bursting of concrete. The monitoring system showed that internal cracking occurred at 65%-75% of the failure load.

Concrete surface cracking was in general observed at 85%-90% of the failure load. The results, in terms, of failure pressure are collected in Table 7-2:

Table 7-2 Summary of the tests results

Test	Specimen	N failure	a1	b1	a	b	f _{cm}	σ failure	Experimental $k = \left(\frac{N_{failure}}{A_{co} f_{cm}} \right)$
		kN	m	m	m	m	Mpa	Mpa	
C-25-8	1	1442	0.17	0.08	0.4	0.5	23	106.0	4.6
C-25-8	2	2160	0.17	0.08	0.4	0.5	23	158.8	6.9
C-25-25	1	3073	0.17	0.25	0.4	0.5	23	72.3	3.1
C-25-25	2	2104	0.17	0.25	0.4	0.5	23	49.5	2.1
BF25_8	1	2821	0.17	0.25	0.4	0.5	23	66.4	2.8
BF25_8	2	2635	0.17	0.25	0.4	0.5	23	62.0	2.7
BF25_16	1	1859	0.17	0.25	0.4	0.5	23	43.7	1.9
BF25_16	2	1812	0.17	0.25	0.4	0.5	23	42.6	1.8
C-25-50	1	3340	0.17	0.50	0.4	0.5	23	39.3	1.7
C-25-50	2	3435	0.17	0.50	0.4	0.5	23	40.4	1.7

Factor k is the ratio between the failure stress on the plate and the uniaxial concrete strength. It has a maximum value of 3.0 according to equation 6.63 of EN-1992-1-10 when failure occurs tri-axial compression.

The representation of the ultimate pressure versus the square root of the ratio between the bearing area, A_{co} , and concrete block area, A_{cl} , is given in Figure 7-1. The symbols represent the experimental results obtained in these tests, and the continuous curve represents the prediction of EN 1992-1-1 assuming a concrete strength equal to the mean measured value of 23 MPa.

It can be seen that the specimens with the smallest loading plate (C25-8) which would be failing in tri-axial compression provide resistance pressures which exceed by a wide margin the value of $3.0 \times f_{cm}$ that limits the resistance of the Eurocode model.

On the other hand, the rest of the tests show adequate agreement with the theoretical predictions.

Another interesting aspect of the results regards specimens with square 250x250 mm plates in which the reinforcement detail was varied.

It can be seen that specimens with more closely spaced reinforcement (BF25_8 – with 8 mm bars) behaved significantly better than specimens with more spaced reinforcement (BF25_16 – with 16 mm bars) which actually fell slightly underneath the EN 1992-1-1 prediction.

The reference specimen (C25-25) shows some scatter with a mean value of 61 MPa, slightly below the mean value specimens BF25_8 (64 MPa). This behaviour is attributed to more effective confinement of concrete which is obtained by more closely spaced bars.

Figure 7-1 Failure pressure versus $\sqrt{A_{co}/A_{c1}}$

If the failure pressures obtained in this study are compared with the current EN-1992-1-1 formulation (using measured mean strength values and assuming a K_{ct} factor of 3.0) it is clear that for high geometric confinement (C25_8 specimen) this formulation results in very conservative estimates.

With reference to other studies in literature (see section 3.5.2), the compressive strength of concrete increases if it is confined (either by the geometry of the problem as in the tests carried out for this study or by applying active lateral pressure, or even by using spiral reinforcement around the loaded area).

It is worth comparing the available tests on confined concrete with the EN-1922-1-1. It is possible to see how numerous tests for high confined concrete show high stresses. A regression line, considering the different tests is shown in Figure 7-2. It can be seen that this limit, seems to be overly conservative. It is significant than in Leonhardt's classical text on Structural Concrete, Vorlesungen über Massivbau [97], in paragraph 3.5 of Volume II, he cites Spieth, as limiting the maximum compression force in the plate to 9 times the concrete strength, and then adds that he recommends a safety factor of 3 to be applied to this value.

This approach was based on the method of admissible stresses. It seems that the origin of today's limit on maximum pressure to be transmitted to concrete comes from the above factors: $9/3=3$. It seems to ignore that codes have shifted from admissible stresses to the partial factor method, and seems to be applying safety twice for this verification.

This would explain the patent contradiction between experimental evidence and the code model as shown in Figure 7-2

Figure 7-2 Comparison of tests carried out with EC2 Approach

The failure mechanism was practically the same in all the specimen. The first registered crack in strain gauges agreed with the formation of the typical strut and tie mechanism, see Figure 7.3, normally in the next-to-last loading step.

Figure 7-3 Comparison of tests carried out with EC2 Approach

Once shown first crack developed (at left in Figure 7-3), as loading increased the specimen lost the unconfined outer skin, before reaching the failure. Normally the increment of applied pressure between the next-to-last and the last step was between 25% and 30 %.

7.2 Stress strain response

Regarding the measurement of strains, the load/strain curves of rebars located near the top centre and bottom of specimens C_25_8, C_25_25 and C25_50 are grouped in Figure 7-4 to

Figure 7-9. It is interesting to note that the strain is always largest in the bottom bar and smallest in the top bar, so that, in spite of the dimensions of the block some part of the bending mechanism is activated. It is also interesting to note that most bars yield before failure of the specimen, although yielding of reinforcement is not the cause of failure but a secondary effect of compressive failure which manifest itself in the penetration of loading plate into the concrete block exhibiting a clear plastic behaviour (see Figure 7-10).

Figure 7-4 Load/strain for top, central and lower bars for specimen C25-8-1

Figure 7-5 Load/strain for top, central and lower bars for specimen C25-8-2

Figure 7-6 Load/strain for top, central and lower bars for specimen C25-25-1

Figure 7-7 Load/strain for top, central and lower bars for specimen C25-25-2

Figure 7-8 Load/strain for top, central and lower bars for specimen C25-50-1

Figure 7-9 Load/strain for top, central and lower bars for specimen C25-50-2

Figure 7-10 Penetration of bearing plate after failure for C25-8 specimen 2

In the Figures 7-11 and 7-12, the average vertical strain of extensometers bases is presented along the transversal and longitudinal directions. The tensile stresses that appear at a load level close to failure result from bending of the outer unconfined skin of the reinforcement whose failure is almost a buckling type.

Figure 7-11 Load/strain for vertical extensometers at transversal direction

The longitudinal behaviour seems to follow the same shape.

Figure 7-12 Load/strain for vertical extensometers at longitudinal direction

7.3 Geometrical vs stress confinement

Using both the formulation of Dahl [5] for the Ottosen [6] coefficients and the Extended Leon formulation, a confinement stress was determined so that the Ottosen or Extended Leon formulations would provide the same results as the formula for concentrated loads of EN 1992-1-1 (without upper limit), thereby obtaining a relationship between geometric and stress-type confinements.

The horizontal stress obtained is assumed to be the same in both directions and can be assimilated to the geometric mean of the actual stresses in each direction.

The results are plotted in Figure 7-12, together with the experimental results of this investigation.

For the experimental results, the confining stress was determined from the actually measured values of the failure compressive stress on the plate where load was applied.

It can be seen that both criteria (Ottosen and Extended Leon) provide fairly similar results which are reasonably in agreement with the experimental values (within the error margin already observed in Figure 7-2).

Figure 7-13 Relationship between geometric confinement and stress confinement

In Figure 7-13 analytical approximations to the curves are also plotted in grey. The analytical expressions for these approximations are given in Eq.7-1.

It can be seen that these expressions provide simple and accurate approximations to correlate geometrical and stress confinement.

Ottosen formula with Dahl Coefficients (7-1)

$$f_{c1} = f_{c2} = 4.4 \left(\frac{A_{c0}}{A_{c1}} \right)^{0.65}$$

Extended Leon Formulation

$$f_{c1} = f_{c2} = 2.1 \left(\frac{A_{c0}}{A_{c1}} \right)^{0.85}$$

7.4 Cracking

Extensive research has confirmed that crack width at service loads is directly related to the measured mean steel elongation i.e., in service, the mean steel stress [61].

In common engineering practice, however, in D regions crack spacing and mean steel elongation are difficult to obtain. In such cases, the value of the crack width cannot be derived directly from a Strut-and-Tie model, which is a design tool based solely on equilibrium [145].

In fact, the steel stress deduced from these models is greater than the real one due to the neglected tensile strength of concrete.

This problem is related to another issue: cracking models provide crack widths at the surface of the concrete elements. Conventionally, crack widths are considered only at the tensile face of beams, as the largest cracks are expected to occur there [134]. Actually, the crack opening near the bar is very small: Goto [65] cracks (internal, secondary, non-passing cracks) help distribute the differential strain between steel and concrete and reduce the opening of the passing crack at bar level.

Therefore, the closing of the internal cracks, which do not appear on the surface, implies an increase in the width of the passing cracks [135].

Of course, the limitation of crack width for corrosion protection would be counterproductive if it is achieved by a cover decrease. For this reason, EN-1992-1-1 [33] did not make the cover an explicit parameter of the formulation for crack spacing. Regarding this last aspect, is worth mentioning the controversial role of cracks in the corrosion of reinforcement. Concrete cracks are generally recognized as facilitating chloride ingress and as initiating the active corrosion of embedded reinforcement. However, while some authors (see, for instance, [142]) measured an increase of the chlorides diffusion and a loss of weight of the reinforcement, many others claim there is no correlation between crack width and durability.

The ACI 318 [11] philosophy, for instance, is steeped in this idea. Moreover, routine inspection of structures shows that durability problems arise from joints and bearings rather than from excessive crack widths. Darwin [48], among others [61], has shown that there is no clear correlation between surface crack in the range normally found at service load levels and the effect of crack width on reinforcement corrosion.

Summarizing, there is no globally accepted formulation for crack width or crack spacing available, or even definition of what the critical variables actually are [24]. It is not even clear if there is really the need of controlling crack width, because the results of accelerated tests have not yet been confirmed as representatives of the behaviour of real structures.

This is even truer in the case, as mentioned before, of D regions: the vast differences or inconsistencies between code provisions are an indicator of the lack of consensus among researchers and designers regarding what the minimum serviceability requirements should be.

According to NHCRP [122], more research on this topic needs to be done. In view of the increasing importance of serviceability in design, this work aims to contribute to the discussion on this topic. Firstly, recommendations found in literature and code provisions are presented.

Then, further experimental results are presented. The study was inspired by observations relating to a real case of a TBM ring.

Although the values of registered maximum pressure vary from 44 MPa to 66 MPa for the cracking tests (#BF specimen), the failure mechanism was almost identical in all specimens.

The observed failure mechanism, moreover, was brittle and once first crack appeared, normally perpendicular at bars LS-G1 or TS-G1, it propagated quickly within the specimen. The first crack appeared at a load of 1500 kN, which corresponds to almost 35 MPa. This value is approximately 1.5 times f_{cm} .

Figure 7-13 shows the average failure pressure for each set of 2 specimens. The figure seems to indicate that using more closely spaced bars provides higher resistance.

Figure 7-14 Average failure pressure for the cracking (#BF) specimen.

The first visible crack appeared along the longitudinal face of the specimen after the next-to-last stage while in the transversal direction the first cracks that formed led to failure in the next load stage as shown in Figure 7-3. The specimens, however, could resist between 25 % and 30 % more after the first cracking before reaching failure.

For all specimens, the strain values measured by strain gauges on the rebars has a close correlation to the average strain values measured by superficial extensometer bases as shown in Figures 7-14 to Figure 7-21, where missing curves are due to malfunction of internal strain gauges.

Figure 7-15 BF-25-Ø8- specimen 1: strain stress curves for longitudinal bars

Figure 7-16 BF-25-Ø8- specimen 1: strain stress curves for transversal bars

Figure 7-17 BF-25-Ø8- specimen 2: strain stress curves for longitudinal bars

Figure 7-18 BF-25-Ø8- specimen 2: strain stress curves for transversal bars

Figure 7-19 BF-25-Ø12 specimen 1: strain stress curves for longitudinal bars

Figure 7-20 BF-25-Ø12 specimen 1: strain stress curves for transversal bars

Figure 7-21 BF-25-Ø12 specimen 2: strain stress curves for longitudinal bars

Figure 7-22 BF-25-Ø12 specimen 2: strain stress curves for transversal bars

Crack widths are determined by identifying extensometer bases that are crossed by a crack and multiplying the strain measured at the bases by the reference length (i.e. 200 mm). This calculation neglects the tensile strain of the uncracked concrete which is one order of magnitude smaller.

The crack width is related to the bar stress by using the strain measurement given by the strain gauge of the corresponding bar and assuming linear behaviour of steel. Figure 7-23 shows the behaviour of the specimen BF-25-Ø8-1 in terms of pressure at the plate against crack width and reinforcement stress.

It can be seen that at 35 MPa the superficial cracks are below 0.1 mm while the strain gauges register the first horizontal movement indicating sudden crack propagation. At 47 MPa the superficial crack has grown to 0.3-0.4 mm and the tensile stresses in the reinforcement vary from 200 MPa for the bottom bar, LI-G3, to 592 MPa for LS-G3.

Figure 7-23 Specimen BF-25-Ø8-1 comparison between ext. central base and steel bars

Specimen BF-25-Ø8-2 shows more or less, the same behaviour (see Figure 7-24). At 35 MPa the superficial cracks are below 0.15mm. At 47 MPa the superficial crack is greater than 0.15 mm and the corresponding steel bar stress is higher than 250 MPa. At this point the system becomes unstable.

Figure 7-24 Specimen BF-25-Ø8-2 comparison between ext. central base and steel bars

The stress of the bars exhibits non-linear behaviour with respect to the bearing pressure, in the elastic branch, due to the formation and propagation of cracks, not due to the yielding of steel.

Specimen BF-25-Ø12-1 (see Figure 7-25) shows that under a pressure of about 60 MPa, superficial cracks are under 0.15mm. The corresponding steel bar stress is about 250 MPa. Failure occurred at 72 MPa after an increment of pressure of 20%. The bars show approximately linear behaviour until first cracking.

Figure 7-25 Specimen BF-25-Ø12-1 comparison between ext. central base and steel bars

Specimen BF-25-Ø12-2 (see Figure 7-26) shows the same behaviour described above but for lower bearing pressure.: under a pressure of 35 MPa the superficial crack is less than 0.15 mm. Failure occurs under an equivalent pressure of 49.5 MPa.

Figure 7-26 Specimen BF-25-Ø12-2 comparison between ext. central base and steel bars

Specimen BF25-16-1 (see Figure 7-27) had a quasi-brittle failure in which no visible superficial cracks were detected during the tests until failure, which was sudden. This behaviour is also descriptive of specimen BF-25-16-2 (see Figure 7-28)

Figure 7-27 Specimen BF-25-Ø16-1 comparison between ext. central base and steel bars

Figure 7-28 Specimen BF-25-Ø16-2 comparison between ext. central base and steel bars

As explained before, the results of the extensometer bases have been used to evaluate the maximum crack width during the step loads. Figure 7-29 shows the estimated crack plotted against the reinforcement stress within the bar. The maximum crack width of 0.55 mm was registered for Ø8mm and Ø 12mm specimens for bar LS-G3 and LS-G1 respectively.

The linear correlation provides a high value for the coefficient of determination is with larger data dispersion for cracks smaller than 0.1 mm and corresponding steel stress of about 150 MPa due to tension stiffening effects. From Figure 7-28 it can be seen that the implicit control of tensile stress of 400 MPa at ULS provides a crack width of about 0.45 mm (at ULS).

However, it must be pointed out that this value takes into account tension stiffening effects and that the 400 MPa value for a S&T model would, in reality, correspond to a lesser stress measured in the bar and thereby to a smaller crack width.

It can be seen also that the specimens with Ø8mm and Ø 12mm registered the maximum crack width at the steel bars closest to bearing plate.

The specimens with Ø 16mm showed the lowest crack widths but also the lowest failure pressures because of their brittle behaviour. Moreover, the steel bars did not register yielding during the tests.

Figure 7-29 Crack width versus reinforcement stress for LS-G1 bar

7.5 Finite element approach

The tests results have been checked against finite elements analysis to study if it will possible to carry out a reliable prediction of the behaviour of the concrete block based on numerical approach.

7.5.1 Software employed

The main application field of the Finite Element program ATENA, created by Červenka Consulting [38], is the analysis of concrete and reinforced concrete structures, focused on non-linear analysis. The software is currently used in a variety of projects research programs and international prediction competitions. For instance, in 2005 the software helped the company in winning a competition whose target was to predict the shear strength of slabs with inclined reinforcement. This software has been employed for the simulation of the experimental campaign described before.

7.5.1.1 Computational Aspects

The use of NFLFEA allows to consider the redistribution of internal forces due to the nonlinear material behaviour. The resulting stress and deformation fields satisfy all three requirements of mechanics: equilibrium of forces, compatibility of deformations and material laws. The material chosen to simulate concrete for this case is CC3DNonLinCementitious2. It is a combined fracture-plastic model for concrete. Tension is handled by a fracture model, based on the classical orthotropic smeared crack formulation and the crack band approach. It employs the Rankine failure criterion, exponential softening, and it can be used for both rotating and

fixed crack model. In this study, the fixed crack model was adopted. The plasticity model for concrete in compression is based on the Menétrey-Willam [115] failure surface.

The position of the Menétrey-William failure surface is not fixed but it can expand and move along the hydrostatic axis, simulating the hardening and softening stages by using factor β which is a model calibration factor. The model can be used to simulate concrete cracking, crushing under high confinement and crack closure due to compression in other directions.

Moreover, the concrete constitutive law is based on a damage concept, in which the stress-strain law covers the whole range of behaviour including the post-peak softening in tension as well as in compression.

For 2-D behaviour, the peak stresses in compression and tension are defined by the strength envelope according to Kupfer [91]. Figure 7-29 shows the basic input needed to cover the whole range of behaviour including the post-peak softening in tension as well as in compression. The peak stresses in compression and tension are defined by the strength envelope according to [35].

Figure 7-30 ATENA's screenshot to define 3D Nonlinear Cementitious 2

The fictitious crack model developed by Hillerborg et al.[73] is considered, in which the tensile strength is controlled by the crack opening. The softening behaviour of concrete in tension adopted is the one derived experimentally by Hordjik [74], and an enhanced crack model that was originally proposed by Bazant et al. [20] also is used. Aggregate interlock and dowel action of the reinforcement is taken into account by a reduction of the shear modulus with growing strain normal to the crack, according to the law derived by Kolmar [90]. The model can be used to simulate concrete cracking, crushing under high confinement, and crack closure due to crushing in other material directions.

The reinforcement can be included in two ways, either as smeared, or as discrete bar elements. In this study, discrete bar elements were used. Their behaviour was modelled by a uniaxial ideally bilinear elasto-plastic stress-strain relationship. Moreover, the bars were included in the model with the original shapes including vertical arms and not, as often done in the engineering practice, as different horizontal layers.

7.5.1.2 Model parameters

7.5.1.2.1 Confined Concrete

As stated before, the material model employed for the analysis is able to reproduce and control during the load process, the cracking and relative softening after cracking. The mean concrete strength, f_{cm} , was taken as 23 MPa. The material parameters employed were the default ones with the following modifications:

- the tensile strength of concrete was reduced by 10%
- the critical compressive displacement, w_d , which was established as 0.005 m
- multiplier for the plastic flow direction, β was taken as 0.2. This parameter is used to define the Menetrey-Williams failure criterion

The tensile strength was reduced by the 10% to take into consideration the influence of the shrinkage in the specimen as per the point 2.2.8 of [38].

This relation is used for stress response in two orthogonal material axes within the concept of orthotropic damage model.

The fixed crack model was adopted. The strain-localization due to softening is controlled by the crack band, which is related to the element size. Strain localizes into a displacement, which represents a failure discontinuity in tension, or compression. Within the crack band model these displacements can be calculated as:

$$w_c = L_t \varepsilon_t \quad (7-2)$$

$$w_d = L_d \varepsilon_d$$

where in tension w_c is the crack width, ε_t is a tensile strain within the crack band L_t . Similarly, w_d is the compression slip of the crushing zone, ε_d is compressive strain (or in case of strain decomposition a plastic strain), L_d is the localization band in compression. The values of bands L_t (tension), L_d (compression), measured in perpendicular directions, are the element projections shown in Figure 7-30.

Figure 7-31 Localization bands in tension and compression [37]

The constitutive laws in the softening ranges, are formulated in terms of stress-displacement variables. In tension the exponential softening law according to Hordijk is accepted, which is fully defined by two parameters, the fracture energy G_f and tensile strength f_t . In compression, the linear approximation of Van Mier softening is used, defined by parameter w_d . Details of these formulations can be found in the references [36] and [37].

The critical displacement has been set up as ten times smaller than the mesh element size (0.04 m) based on the study of low mesh sensitivity of the concrete models carried out in [36].

Finally, the β factor was set up at 0.2 in accordance with [36] as the default value, zero, seems to be very conservative. However, its value should not exceed 0.5 in order to not generate convergence problems during the computational steps.

Finally, the load was increased by deformation control in steps according to the standard Newton-Raphson iteration method.

The load was simulated as a vertical prescribed deformation applied on the bearing plate. The displacement was the ratio between the maximum bearing pressure and the bearing plate's Young's modulus.

7.5.1.2.2 Steel

Reinforcing steel was modelled as a discrete material with hardening behaviour with yield stress, σ_y , of 550 MPa ultimate stress, σ_t , of 650 MPa and a maximum strain ϵ_{lim} of 0.067. The Young's Modulus was taken as 200 GPa. The steel employed for the bearing plate, was simulated with as an elastic material with a Young's modulus of 200 GPa.

7.5.1.2.3 Neoprene

The neoprene employed in the tests was modelled as a series of vertical springs.

7.5.2 FEM Results

In this paragraph, a comparison between the results obtained using NLFEA (ATENA) and those measured in the tests is carried out. The results are focused on particular issues which are very important for the design:

- The evaluation of the steel stress within the bar measured in one point
- The evaluation of cracking and the cracking patterns to better investigate and prevent possible durability problems under the serviceability conditions.
- The ability of the software to reproduce the high pressures measured within the concrete blocks

It is possible to see in Figure 7-32 and Figure 7-33 which present a comparison between stresses in reinforcement obtained from strains measured in tests against the stresses in the bearing plate in specimen C25-8 in the transversal (left) and longitudinal (right) directions, and how the values taken from NLFEA that, the values of the tensile stresses measured and predicted are of the same order of magnitude although the patterns are quite different. This could be due to the high compressive stress measured in concrete (more than 100 MPa) which is approximately 4 times the values of f_{cm} .

This is a reflection on the difficulties encountered by NLFEA to closely model the behaviour in more extreme case. Graphs 7-32 and 7-33 correspond to different strain gauges as there no possibility to find a pair of working strain gauges to compare the results.

Figure 7-32 Bar reinforcement stress comparison between ATENA and measured tests for C25-8-1

However, the “elastic” branches of the curves show reasonable agreement between NFLEA and experimental tests.

Figure 7-33 Bar reinforcement stress comparison between ATENA and measured tests for C25-8-2

For specimen BF25-12 (see Figure 7-34), the correspondence between theoretical calculations and tests is closer.

Figure 7-34 Bar reinforcement stress comparison between ATENA and measured tests for BF-25-12 (C25-25)

For specimen BF25-8 (see Figure 7-35) the NLFEA is also able to reproduce reasonably well the measured behaviour for lower pressure values. However, the stiffness and strength are somewhat overestimated for higher pressures.

Figure 7-35 Bar reinforcement stress comparison between ATENA and measured tests for BF-25-8

For specimen BF25-16 (see Figure 7-36) good agreement is observed within the full range of the behaviour.

Figure 7-36 Bar reinforcement stress comparison between ATENA and measured tests for BF-25-16

For specimen C25-50 (see Figure 7-37) the elastic branch of the reinforcement bar is adequately reproduced in this case too.

Figure 7-37 Bar reinforcement stress comparison between ATENA and measured tests for C25-50-1

Finally, a comparison of the measured and predicted (NLFEA) the crack widths is presented in Figure 7-37. The corresponding average value are measured at the extensometers bases installed in correspondence of the steel bars. The agreement between the finite elements and the experiments is quite good for the step prior to failure for which significant cracking is first observed.

Figure 7-38 Comparison between cracking width predicted by NLFEA and average measured crack width

8 CONCLUSIONS AND FUTURE RESEARCH

This chapter summarizes the main conclusions derived from the work and analysis presented in the previous chapters and provides guidelines for future research.

8.1 Conclusion regarding the characterization and measurement of tunnel lining defects

Characterization of the defects is of special importance in order to decide on their treatment and the need or not to evaluate their possible bearing on structural safety.

A new specifically tailored method of measurement was proposed for the measuring of misalignments and rotations of joints corresponding to keystone segments. Also a simple measurement procedure for the determination of the contact surface at flat joint was also introduced. The quality of the measurements was shown to be adequate by correlation to other measurements involving global mapping. For this a procedure was outlined in order to obtain comparable measurements.

It is significant that a large number of measured segments were shown to be out of tolerance:

- 153 rings (2.2% of the analysed rings) showed an ovalisation greater than 1.0%, the acceptable allowable limit for tunnel deformation,
- In 95% of the cases, segment misalignments were shown to be larger than the established tolerance of 10 mm.
- In 98% of the cases measured joint openings in flat joints were shown to be larger than the tolerance of 0.5°
- In 80% of the cases joint openings in keystone segments were shown to be larger than the tolerance of 0.5°

In spite of these figures, the tunnels showed neither structural nor serviceability distress (except excessive cracking due insufficient minimum reinforcement). This fact is related to the high redundancy and robustness of this type of structure, and shows, that while having strict

tolerances may be a good manner of improving construction quality, when the established values are not fulfilled the consequences are not dramatic, and should not be exaggerated.

In any case, when defects do occur, it is important to have a clear methodology to investigate, classify and measure them, in order to be able to carry out a solid analysis of consequences and define the corresponding repair procedures when needed.

8.2 Conclusion regarding the behaviour of a tunnel lining under lack of back-fill

Non-linear analysis of a tunnel ring with poor support conditions carried out in Chapter 5 has shown that:

- The approach of tunnel design based on elastic behaviour of reinforced concrete, while generally acceptable for service conditions, is not always adequate for the initial phase of ring installation, since the axial forces at the joints are very low.
- For the construction phase the usual models (Jansenn, Blom) are not applicable due to lack of axial force. For this case, the analysis must take into account provisional bolts which guarantee the structural integrity during this phase.
- The values of deformations and cracks widths observed on site can be simulated assuming an asymmetrical saddle based on field observations. This configuration, never studied in detail, explains the severe ovalisation within the ring and the presence of large cracks at the tunnel crown observed on site.

The model employed, to simulate the joints between the segments, allows to model hinges within the ring which lead to redistribution of internal forces. This redistribution capacity is the reason why the formation of a hinge within the ring does not lead to collapse.

8.3 Conclusions regarding geometrical confinement

The study of joints between tunnel lining segments shows that:

- TBM joints are designed to transmit high pressures. This is due to the geometrical reduction between segment height and joint section, which is close to 50 %. This reduction is needed to facilitate the ring assembly and the installation of the gasket. The stresses are further increased due to load eccentricity and construction defects.
- Even without construction defects, a rigorous application of design standards would result in non-compliance in many practical cases.

In order to further investigate this circumstance, an experimental campaign was carried out, aiming to model a specific case. In the experimental campaign the degree of geometric confinement was varied by varying the dimensions of the bearing plate.

The results of the experimental campaign have shown that standards are overly conservative, a fact that can also be established from the experimental results provided by other researchers. The factor that seems to explain this situation is the limit of the maximum compressive stress on the bearing plate to about 3 times the uniaxial compressive strength. In this study values of up to 7 were registered before failure. If this limitation is excluded, or heavily increased, the predictions become much more realistic. Without this limitation, the current tunnelling design practice can be fully justified.

Confinement of concrete can be established by means of an external actively compressive pressure applied perpendicularly to the main loading direction, as it is done in triaxial tests, as

a passive confinement provided by reinforcement or as a geometric confinement which is obtained when the loaded area is smaller than the dimensions of the concrete block. The relationship between geometrical confinement and stress confinement has been studied by establishing a relationship between equivalent external confining pressures and the ratio between loaded area and the dimensions of the concrete block. For this, the expression of EN 1992-1-1 (without the limiting the maximum compressive stress) has been compared to the general failure criteria of Ottosen and to the extended Leon formulation. By comparison between these formulations, simple expressions relating to geometric and stress confinement have been obtained.

8.4 Conclusions regarding cracking

The results of the experimental test show that:

- The standards' approach to indirectly control the cracking within D-regions by limiting the maximum stress within the steel in ULS seems to be adequate
- No cracking was detected under the failure in BF-25-Ø16 specimens which exhibited the smallest failure pressures.
- The behaviour of the tests carried out was relatively brittle due to the high compressive stresses achieved. Spalling of the corner was observed in all specimens at failure. Visible cracking occurred only in the next-to-last loading step for Ø8 and Ø12 mm.
- All the specimens showed the first crack at bearing pressures from 35 MPa to 40 MPa which means from 1.5 to 2.0 times f_{cm} .
- The higher efficiency provided by small bars Ø8mm, compared with Ø12mm and Ø16mm, allowed to reach higher values of the failure pressure
- Crack width was better controlled in Ø8mm specimen rather than Ø12mm or Ø16mm because of the smaller spacing between bars.

It is therefore recommended to use smaller diameters to improve the performance of D-Regions involving concentrated loads producing high pressure in C-C-C nodes.

8.5 Conclusions regarding NLFEA analysis

The results of the comparison between NLFEA analysis and tests results show that:

- The analysis of D-regions should begin with the Stru&Tie approach based on plasticity theory. However, this approach cannot be considered, in some cases, as the final design point as it is not able to reproduce the effective triaxial behaviour of concrete.
- When the concrete in a D-region is subjected to high compressive stresses, its modelling should implement a concrete damage model
- The software used (ATENA) was able to reproduce the high pressures measured in concrete blocks during the tests.
- The concrete parameters were set up as the default ones except for those connected with plastic flow. The justification of the parameters employed are available in technical literature and in the software's manual
- The NLFEA results have been compared to the results of an experimental campaign carried out and described in paragraph 2.
- The results and comparison show that the software used (ATENA in this case) with an adequate choice of parameters was able to reproduce the elastic branch of steel stress

curves and, the non-linear branch for concrete stresses for up to three times the value of f_{cm} . For higher stresses, larger deformations were observed in tests.

- The crack width estimates obtained from the model compared very well with the average values measured using extensometer bases on the specimens, for the stage prior to failure.

8.6 Future research

Future research should focus on:

- Normative implementation of the TBM analysis stage construction in the design conditions shown in chapter 5.
- Evaluation of the ovalisation of the TBM rings as a design approach to be systematically analyzed in design
- A deeper analysis of the geometrical D-regions should be made in order to revise the Standards limitations to maximum compressive stress.
- Cracking tests clearly show that the diameter of the rebars affect the failure pressure and, implicitly, the cracking. Further investigations should focus on the experimental tests to provide a direct crack width estimation in SLS rather than an implicit control as assumed in the major Standards.

APPENDIX A: LABORATORY TEST DATA

This appendix collects all the results of the tests carried out during the campaign described in the chapter 6.

A.1 C25 50: specimen 1

The first specimen to be tested was the C25_50 specimen 1. This test took place 78 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 200 kN incremental steps until reaching the failure load.

A.1.1 Instrumentation nomenclature

The instrumentation nomenclature is collected in the following figure:

Figure A 1 C25-50 specimen 1 3D representation

The geometrical definitions are:

Table A 1 C25-50 specimen 1 geometrical definitions

Transversal [m]	longitudinal [m]	Specimen		
		Height [m]	Trans. bearing plate [m]	Longit. bearing plate [m]
0.4	0.5	0.5	0.17	0.50

Steel reinforcement is collected in the following table:

Table A 2 C25-50 specimen 1 reinforcement definition

Transversal reinforcement		Longitudinal Reinforcement		Supports
reinforcement	Required steel	Reinforcement	Required steel Area	reinforcement
	Area [mm ²]		[mm ²]	
24Ø12	2400	12Ø12	1200	2Ø12

A.1.2 Instrumentation definition

As stated before each specimen was equipped with strain gauges within the internal steel bars and extensometer bases along the concrete surface. The definition of the instrumentation is collected in the following figures where the positive and negative faces are defined in Figure A 1:

Figure A 2 C25-50 specimen 1: instrumentation nomenclature

A.1.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 3 C25-50 specimen 1 load displacement curve L-H-1

Figure A 4 C25-50 specimen 1 load displacement curve L-H-2

Figure A 5 C25-50 specimen 1 load displacement curve L-H-3

Figure A 6 C25-50 specimen 1 load displacement curve T-H-1

Figure A 7 C25-50 specimen 1 load displacement curve T-H-2

Figure A 8 C25-50 specimen 1 load displacement curve T-H-3

A.1.4 Strain Gauges curves

The strain gauges were installed in correspondence along the central bars beneath the bearing plate (see Figure A 1). The results are shown in the following figures:

Figure A 9 C25-50 specimen 1 C25-50 specimen 1 bar: LS-G2

Figure A 10 C25-50 specimen 1 C25-50 specimen 1 bar: LS-G3

Figure A 11 C25-50 specimen 1 C25-50 specimen 1 bar: LI-G3

Figure A 12 C25-50 specimen 1 C25-50 specimen 1 bar: TS-G1

A.1.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 3340 kN which corresponds with a vertical pressure of 39.3 MPa approximately. The failure was fragile. The first superficial crack appeared under a vertical load of 2600 kN which corresponds with a vertical pressure of 30.5 MPa.

Figure A 13 Failure mechanism at penultimate phase (at left) and collapse (at right).

Strain Gauges Ls-G1, LI-G2 and TS-G2 did not work during the testing. After second step a neoprene support below the specimen was introduced to nullify the bottom confining.

A.2 C25 50: specimen 2

This test took place 83 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental step till reaching the failure load at 3430 kN approximately which corresponds with a vertical pressure of 40.3 MPa.

A.2.1 Instrumentation nomenclature

The instrumentation nomenclature and all the elements are the same defined for the C25-50 specimen 1).

A.2.2 Instrumentation definition

The instrumentation definition is the same defined for C25-50 specimen 1.

A.2.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 14 C25-50 specimen 2 load displacement curve. L-H-1

Figure A 15 C25-50 specimen 2 load displacement curve. L-H-2

Figure A 16 C25-50 specimen 2 load displacement curve. L-H-3

Figure A 17 C25-50 specimen 2 load displacement curve. T-H-1

Figure A 18 C25-50 specimen 2 load displacement curve. T-H-2

Figure A 19 C25-50 specimen 2 load displacement curve. T-H-3

Figure A 20 C25-50 specimen 2 Vertical displacement curves along transversal and longitudinal directions

A.2.4 Strain Gauges curves

The strain gauges were installed in correspondence along the central bars beneath the bearing plate (see Figure A 1). The results are shown in the following figures:

Figure A 21 C25-50 specimen 2 bar: LS-G3

Figure A 22 C25-50 specimen 2 bar: LI-G1

Figure A 23 C25-50 specimen 2 bar: LI-G3

Figure A 24 C25-50 specimen 2 bar: TS-G3

Figure A 25 C25-50 specimen 2 bar: TI-G1

Figure A 26 C25-50 specimen 2 bar: TI-G3

A.2.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of approximately 3400 kN which corresponds with a vertical pressure of 40.0 MPa. The failure was fragile. The first superficial crack appeared under a load of 2500 kN which corresponds with a pressure 29.4 MPa.

Figure A 27 C25-50 specimen 2 Failure Mechanism

A.3 C25 8: specimen 1

This test took place 86 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental steps until reaching the failure load.

A.3.1 Instrumentation nomenclature

The instrumentation nomenclature is collected in the following figure:

Figure A 28 C25-8 specimen 1 3D representation

The geometrical definitions are:

Table A 3 C25-8 specimen 1 geometrical definitions

Transversal	longitudinal	Specimen Height	Trans. bearing plate	Longit. bearing plate
[m]	[m]	[m]	[m]	[m]
0.4	0.5	0.5	0.17	0.08

Steel reinforcement is collected in the following table:

Table A 4 C25-8 specimen 1 reinforcement definition

Transversal reinforcement	Longitudinal Reinforcement	Supports
reinforcement	Reinforcement	reinforcement
Required steel Area [mm ²]	Required steel Area [mm ²]	
24Ø12	30Ø12	2Ø12
2400	3100	

A.3.2 Instrumentation definition

As stated before each specimen was equipped with strain gauges within the internal steel bars and extensometer bases along the concrete surface. The definition of the instrumentation is collected in the following figure where the nomenclature depends of positive and negative face (see the Figure A 28):

Figure A 29 C25-8 specimen 1 Instrumentation nomenclature

A.3.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 30 C25-8 specimen 1 load displacement curve: L-H-1

Figure A 31 C25-8 specimen 1 load displacement curve: L-H-2

Figure A 32 C25-8 specimen 1 load displacement curve: L-H-3

Figure A 33 C25-8 specimen 1 load displacement curve: T-H-1

Figure A 34 C25-8 specimen 1 load displacement curve: T-H-2

Figure A 35 C25-8 specimen 1 load displacement curve: T-H-3

Figure A 36 C25-8 specimen 1 vertical load displacement curves in transversal and longitudinal directions

A.3.4 Strain Gauges curves

The strain gauges were installed in correspondence along the central bars beneath the bearing plate (see Figure A 1). The results are shown in the following figures:

Figure A 37 C25-8 specimen 1 Strain Gauge LS-G1

Figure A 38 C25-8 specimen 1 Strain Gauge LS-G2

Figure A 39 C25-8 specimen 1 Strain Gauge LI-G1

Figure A 40 C25-8 specimen 1 Strain Gauge LI-G2

Figure A 41 C25-8 specimen 1 Strain Gauge TS-G1

Figure A 42 C25-8 specimen 1 Strain Gauge TS-G2

Figure A 43 C25-8 specimen 1 Strain Gauge TI-G1

Figure A 44 C25-8 specimen 1 Strain Gauge TI-G3

A.3.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 1442 kN which corresponds with a vertical pressure of 106 Mpa approximately. The rupture was fragile and the once the first superficial crack appeared the specimen lost its integrity without any load increment.

Figure A 45 C25-8 specimen 1 Failure for concrete splitting (at left) and vertical crack (at right)

A.4 C25 8: specimen 2

This test took place 86 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental step till reaching the failure load.

A.4.1 Instrumentation nomenclature

The instrumentation nomenclature is the same employed for the specimen C25-8 1 (see Figure A28)

A.4.2 Instrumentation definition

The instrumentation definition is the same employed for the specimen C25-8-1 (see Figure A29)

A.4.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 46 C25-8 specimen 2 Load displacement curve. L-H-1

Figure A 47 C25-8 specimen 2 Load displacement curve. L-H-2

Figure A 48 C25-8 specimen 2 Load displacement curve. L-H-3

Figure A 49 C25-8 specimen 2 Load displacement curve. T-H-1

Figure A 50 C25-8 specimen 2 Load displacement curve. T-H-2

Figure A 51 C25-8 specimen 2 Load displacement curve. T-H-3

Figure A 52 C25-8 specimen 2 Load displacement curve. T-H-3

Figure A 53 C25-8 specimen 2 Vertical Load displacement curves in transversal and longitudinal directions.

A.4.4 Strain Gauges curves

The strain gauges were located at the middle bars just beneath the bearing plate. The load displacement curves are collected below:

Figure A 54 C25-8 specimen 2 Strain gauge LS-G2

Figure A 55 C25-8 specimen 2 Strain gauge LS-G3

Figure A 56 C25-8 specimen 2 Strain gauge LI-G1

Figure A 57 C25-8 specimen 2 Strain gauge LI-G2

Figure A 58 C25-8 specimen 2 Strain gauge LI-G3

Figure A 59 C25-8 specimen 2 Strain gauge TS-G3

Figure A 60 C25-8 specimen 2 Strain gauge TI-G1

Figure A 61 C25-8 specimen 2 Strain gauge TI-G2

Figure A 62 C25-8 specimen 2 Strain gauge TI-G3

A.4.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 2160 kN which corresponds with a vertical pressure of 158.8 Mpa. The first superficial crack appeared at longitudinal face under a load of 1500 kN, while the specimen could completely resist another step load till 2000 kN. The failure was by concrete splitting.

Figure A 63 C25-8 Failure mechanism: superficial crack at left and concrete splitting at right.

A.5 C25 25 (BF25-12): specimen 1

This test took place 90 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental steps until reaching the failure load.

A.5.1 Instrumentation nomenclature

The instrumentation nomenclature is collected in the following figure:

Figure A 64 C25-25 specimen 1 3D representation

The geometrical definitions are:

Table A 5 C25-25 specimen 1 geometrical definitions

Transversal	longitudinal	Specimen Height	Trans. bearing plate	Longit. bearing plate
[m]	[m]	[m]	[m]	[m]
0.4	0.5	0.5	0.17	0.25

Steel reinforcement is collected in the following table:

Table A 6 C25-50 specimen 1 reinforcement definition

Transversal reinforcement		Longitudinal Reinforcement		Supports
reinforcement	Required steel Area [mm ²]	Reinforcement	Required steel Area [mm ²]	reinforcement
24Ø12	2400	24Ø12	2300	2Ø12

A.5.2 Instrumentation definition

As stated before each specimen was equipped with strain gauges within the internal steel bars and extensometer bases along the concrete surface. The definition of the instrumentation is collected in the following figure where the nomenclature depends of positive and negative face (see the Figure A 64):

Figure A 65 C25-25 specimen 1 instrumentation nomenclature

A.5.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 66 C25-25 specimen 1 Load displacement curve: L-H-1

Figure A 67 C25-25 specimen 1 Load displacement curve: L-H-2

Figure A 68 C25-25 specimen 1 Load displacement curve: L-H-3

Figure A 69 C25-25 specimen 1 Load displacement curve: T-H-1

Figure A 70 C25-25 specimen 1 Load displacement curve: T-H-2

Figure A 71 C25-25 specimen 1 Load displacement curve: T-H-3

Figure A 72 C25-25 specimen 1 Vertical load displacement curves in transversal and longitudinal directions

A.5.4 Strain Gauges curves

The strain gauges were installed at the middle bars under the bearing plate. The results are shown in the following figures:

Figure A 73 C25-25 specimen 1 Strain gauge LS-G1

Figure A 74 C25-25 specimen 1 Strain gauge LS-G2

Figure A 75 C25-25 specimen 1 Strain gauge LS-G3

Figure A 76 C25-25 specimen 1 Strain gauge LI-G1

Figure A 77 C25-25 specimen 1 Strain gauge LI-G3

Figure A 78 C25-25 specimen 1 Strain gauge TS-G1

Figure A 79 C25-25 specimen 1 Strain gauge TS-G2

Figure A 80 C25-25 specimen 1 Strain gauge TS-G3

Figure A 81 C25-25 specimen 1 Strain gauge TI-G1

A.5.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 3073 kN which corresponds with a vertical pressure of 72.3 MPa approximately. The specimen did not show any superficial crack during the step loads so the failure was fragile. The bearing plate produced a superficial rupture near.

Figure A 82 C25-25 specimen 1 Failure mechanism

A.6 C25 25 (BF25-12): specimen 2

This test took place 90 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental step till reaching the failure load.

A.6.1 Instrumentation nomenclature

The instrumentation nomenclature is the same employed for C25-25 specimen 1 (see Figure A 64).

A.6.2 Instrumentation definition

The instrumentation definition is the same employed for the specimen C25-25 1 (see Figure A 65).

A.6.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 83 C25-25 specimen 2 Load displacement curve L-H-1

Figure A 84 C25-25 specimen 2 Load displacement curve L-H-2

Figure A 85 C25-25 specimen 2 Load displacement curve L-H-3

Figure A 86 C25-25 specimen 2 Load displacement curve T-H-1

Figure A 87 C25-25 specimen 2 Load displacement curve T-H-2

Figure A 88 C25-25 specimen 2 Load displacement curve T-H-3

Figure A 89 C25-25 specimen 2 Vertical load displacement curves in transversal and longitudinal directions.

A.6.4 Strain Gauges curves

The strain gauges were installed at middle bars just beneath the bearing plate. The results are shown below:

Figure A 90 C25-25 specimen 2 Strain Gauge LS-G1

Figure A 91 C25-25 specimen 2 Strain Gauge LS-G3

Figure A 92 C25-25 specimen 2 Strain Gauge LI-G1

Figure A 93 C25-25 specimen 2 Strain Gauge LI-G2

Figure A 94 C25-25 specimen 2 Strain Gauge LI-G3

Figure A 95 C25-25 specimen 2 Strain Gauge TS-G1

Figure A 96 C25-25 specimen 2 Strain Gauge TS-G3

Figure A 97 C25-25 specimen 2 Strain Gauge TI-G1

A.6.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 2075 kN which corresponds with a vertical pressure of 48.8 Mpa approximately. Strain gauges LS-G2, TS-G2 and LI-G2 did not worked during the testing. No superficial cracks were observed during the testing.

Figure A 98 C25-25 specimen 2 Failure Mechanism

A.7 BF25 8: specimen 1

This test took place 92 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental steps until reaching the failure load.

A.7.1 Instrumentation nomenclature

The instrumentation nomenclature is collected in the following figure:

Figure A 99 BF-25-8 specimen 1 3D representation

The geometrical definitions are:

Table A 7 BF-25-8 specimen 1 geometrical definitions

Transversal	longitudinal	Specimen Height	Trans. bearing plate	Longit. bearing plate
[m]	[m]	[m]	[m]	[m]
0.4	0.5	0.5	0.25	0.50

Steel reinforcement is collected in the following table:

Table A 8 BF-25-8 specimen 1 reinforcement definition

Transversal reinforcement	Longitudinal Reinforcement	Supports
reinforcement	Reinforcement	reinforcement
Required steel Area [mm ²]	Required steel Area [mm ²]	
48Ø8	48Ø8	2Ø12
2400	2400	

A.7.2 Instrumentation definition

As stated before each specimen was equipped with strain gauges within the internal steel bars and extensometer bases along the concrete surface. The definition of the instrumentation is collected in the following figure where the nomenclature depends of positive and negative face (see the Figure A 99):

Figure A 100 BF-25-8 specimen 1 instrumentation nomenclature

A.7.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 101 BF-25-8 specimen 1 Load displacement L-H-1

Figure A 102 BF-25-8 specimen 1 Load displacement L-H-2

Figure A 103 BF-25-8 specimen 1 Load displacement L-I-1

Figure A 104 BF-25-8 specimen 1 Load displacement L-I-2

Figure A 105 BF-25-8 specimen 1 Load displacement T-H-1

Figure A 106 BF-25-8 specimen 1 Load displacement T-H-2

Figure A 107 BF-25-8 specimen 1 Load displacement T-H-3

Figure A 108 BF-25-8 specimen 1 Load displacement T-H-4

Figure A 109 BF-25-8 specimen 1 Vertical load displacement curves in transversal and longitudinal directions

A.7.4 Strain Gauges curves

Strain gauges were installed at the middle bars just beneath the bearing plate. Results are shown in the following figures:

Figure A 110 BF-25-8 specimen 1 Strain gauge LS-G1

Figure A 111 BF-25-8 specimen 1 Strain gauge LS-G2

Figure A 112 BF-25-8 specimen 1 Strain gauge LS-G3

Figure A 113 BF-25-8 specimen 1 Strain gauge LS-G4

Figure A 114 BF-25-8 specimen 1 Strain gauge LI-G1

Figure A 115 BF-25-8 specimen 1 Strain gauge LI-G2

Figure A 116 BF-25-8 specimen 1 Strain gauge LI-G3

Figure A 117 BF-25-8 specimen 1 Strain gauge LI-G4

Figure A 118 BF-25-8 specimen 1 Strain gauge TS-G1

Figure A 119 BF-25-8 specimen 1 Strain gauge TS-G2

Figure A 120 BF-25-8 specimen 1 Strain gauge TS-G3

Figure A 121 BF-25-8 specimen 1 Strain gauge TI-G2

Figure A 122 BF-25-8 specimen 1 Strain gauge TI-G3

A.7.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 2821 kN which corresponds with a vertical pressure of 66.3 Mpa approximately. Superficial cracks appeared under a vertical load of 1500 kN along longitudinal face and, at 2000 kN some horizontal cracks appeared along the longitudinal face. Failure mechanism occurred by concrete splitting.

Figure A 123 BF-25-8 specimen 1 Failure mechanism. Superficial crack at left and concrete splitting at right.

A.8 BF25 8: specimen 2

This test took place 92 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental step till failure load.

A.8.1 Instrumentation nomenclature

The instrumentation nomenclature is the same employed for BF25-8 specimen 1 (see Figure A 99)

A.8.2 Instrumentation definition

The instrument definition is the same employed for BF25-8 specimen 1 (see Figure A 100)

A.8.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 124 BF-25-8 specimen 2 Load displacement curves L-H-1

Figure A 125 BF-25-8 specimen 2 Load displacement curves L-H-2

Figure A 126 BF-25-8 specimen 2 Load displacement curves L-H-3

Figure A 127 BF-25-8 specimen 2 Load displacement curves L-H-4

Figure A 128 BF-25-8 specimen 2 Load displacement curves T-H-1

Figure A 129 BF-25-8 specimen 2 Load displacement curves T-H-2

Figure A 130 BF-25-8 specimen 2 Load displacement curves T-H-3

Figure A 131 BF-25-8 specimen 2 Load displacement curves T-H-4

Figure A 132 BF-25-8 specimen 2 Vertical load displacement curves in transversal and longitudinal directions

A.8.4 Strain Gauges curves

Strain gauges were installed at the middle bars just beneath the bearing plate. The results are shown in the following curves:

Figure A 133 BF-25-8 specimen 2 Strain gauge LS-G1

Figure A 134 BF-25-8 specimen 2 Strain gauge LS-G2

Figure A 135 BF-25-8 specimen 2 Strain gauge LS-G3

Figure A 136 BF-25-8 specimen 2 Strain gauge LI-G1

Figure A 137 BF-25-8 specimen 2 Strain gauge LI-G2

BF25-8 SP.2 LI-G3

Figure A 138 BF-25-8 specimen 2 Strain gauge LI-G3

BF25-8 SP.2 LI-G4

Figure A 139 BF-25-8 specimen 2 Strain gauge LI-G4

Figure A 140 BF-25-8 specimen 2 Strain gauge TS-G2

Figure A 141 BF-25-8 specimen 2 Strain gauge TS-G3

BF25-8 SP.2 TS-G4

Figure A 142 BF-25-8 specimen 2 Strain gauge TS-G4

BF25-8 SP.2 TI-G1

Figure A 143 BF-25-8 specimen 2 Strain gauge TI-G1

Figure A 144 BF-25-8 specimen 2 Strain gauge TI-G2

Figure A 145 BF-25-8 specimen 2 Strain gauge TI-G3

Figure A 146 BF-25-8 specimen 2 Strain gauge TI-G4

A.8.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 2578 kN which corresponds with a vertical pressure of 60.6 Mpa approximately. A vertical superficial crack appeared under a load of 1500 kN along the longitudinal face of the specimen. Different cracking appeared under a load of 2500 kN and then the rupture occurred after starting the 3000 kN step.

Figure A 147 BF-25-8 specimen 2 Failure mechanism. First cracking at left and failure at right.

A.9 BF25 16: specimen 1

This test took place 93 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental steps until reaching the failure load.

A.9.1 Instrumentation nomenclature

The instrumentation nomenclature is collected in the following figure:

Figure A 148 BF25-16 specimen 1 3D representation

The geometrical definitions are:

Table A 9 C25-50 specimen 1 geometrical definitions

Transversal	longitudinal	Specimen Height	Trans. bearing plate	Longit. bearing plate
[m]	[m]	[m]	[m]	[m]
0.4	0.5	0.5	0.17	0.25

Steel reinforcement is collected in the following table:

Table A 10 C25-50 specimen 1 reinforcement definition

Transversal reinforcement	Longitudinal Reinforcement	Supports
reinforcement	Reinforcement	reinforcement
Required steel Area [mm ²]	Required steel Area [mm ²]	
12Ø16	12Ø16	2Ø12

A.9.2 Instrumentation definition

As stated before each specimen was equipped with strain gauges within the internal steel bars and extensometer bases along the concrete surface. The definition of the instrumentation is collected in the following figure where the nomenclature depends of positive and negative face (see the Figure A 148):

Figure A 149 BF25-16 specimen 1 instrumentation nomenclature

A.9.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 150 BF25-16 specimen 1 Load displacement curve L-H-1

Figure A 151 BF25-16 specimen 1 Load displacement curve L-H-2

Figure A 152 BF25-16 specimen 1 Load displacement curve T-H-1

Figure A 153 BF25-16 specimen 1 Load displacement curve T-H-2

Figure A 154 BF25-16 specimen 1 Vertical load displacements in transversal and longitudinal directions

A.9.4 Strain Gauges curves

The strain gauges were installed at the middle bars just beneath the bearing plate. The load displacement curves are collected in the following curves:

Figure A 155 BF25-16 specimen 1 Strain gauges LS-G1

Figure A 156 BF25-16 specimen 1 Strain gauges LS-G2

Figure A 157 BF25-16 specimen 1 Strain gauges LI-G1

Figure A 158 BF25-16 specimen 1 Strain gauges LI-G2

Figure A 159 BF25-16 specimen 1 Strain gauges TS-G1

Figure A 160 BF25-16 specimen 1 Strain gauges TS-G2

Figure A 161 BF25-16 specimen 1 Strain gauges TI-G1

A.9.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 1859 kN which corresponds with a vertical pressure of 43.7 Mpa approximately. The observation of the failed specimen detected the longitudinal steel bars out of original plane. Strain gauge TI-G2 did not worked during the testing.

Figure A 162 BF25-16 specimen 1. Failure mechanism: steel bars out of plan at left and failed specimen at right.

A.10 BF25 16: specimen 2

This test took place 93 days after pouring. Mean concrete compressive strength was 23 MPa. The test was carried out with 500 kN incremental step till reaching the failure load.

A.10.1 Instrumentation nomenclature

The instrumentation nomenclature is the same collected for BF25-16 specimen 1 (see Figure A 148).

A.10.2 Instrumentation definition

The instrumentation definition is the same collected for BF25-16 specimen 1 (see Figure A 149)

A.10.3 Load Displacement curves

The load displacement curves are collected below:

Figure A 163 BF25-16 specimen 2 Load displacement curve L-H-1

Figure A 164 BF25-16 specimen 2 Load displacement curve L-H-2

Figure A 165 BF25-16 specimen 2 Load displacement curve T-H-1

Figure A 166 BF25-16 specimen 2 Load displacement curve T-H-2

Figure A 167 BF25-16 specimen 2 Vertical load displacement curves in transversal and longitudinal directions

A.10.4 Strain Gauges curves

The strain gauges were installed at the middle bars just beneath the bearing plate. The load displacement curves are collected below:

Figure A 168 BF25-16 specimen 2 Strain Gauges LS-G1

Figure A 169 BF25-16 specimen 2 Strain Gauges LS-G2

Figure A 170 BF25-16 specimen 2 Strain Gauges LI-G1

BF25-16 SP.2 LI-G2

Figure A 171 BF25-16 specimen 2 Strain Gauges LI-G2

BF25-16 SP.2 TS-G1

Figure A 172 BF25-16 specimen 2 Strain Gauges TS-G1

Figure A 173 BF25-16 specimen 2 Strain Gauges TS-G2

Figure A 174 BF25-16 specimen 2 Strain Gauges TI-G1

Figure A 175 BF25-16 specimen 2 Strain Gauges TI-G2

A.10.5 Failure Mechanism and remarks

The failure mechanism was by cracking of the specimen with consequent concrete cover splitting under a load of 1812 kN which corresponds with a vertical pressure of 42.6 Mpa approximately. There was no registration of superficial crack during testing till failure. The mechanism of failure produced a translational movement of concrete cover and a general failure along the longitudinal face of the specimen.

Figure A 176 BF25-16 specimen 2 Failure mechanism. Translational movement at left and cracking before failure.

APPENDIX B: FINITE ELEMENT ANALYSIS

In this appendix, the mesh and bars position are collected together with the convergence parameters monitored during the analysis to check the buoyancy of the models and the results discussed in Chapter 7.

B.1 C25 50

Figure B 1 C25-50 Mesh and bars position

Figure B 2 C25-50 Disp.Err

Figure B 3 C25-50 Resid.Err.

Figure B 4 C25-50 Res.Abs.E

Figure B 5 C25-50 Energy E

Figure B 6 C25-50 RDisp. Err

Figure B 7 C25-50 RResid. Err

Figure B 8 C25-50 RRes. Abs. E

Figure B 9 C25-50 REnergy E.

B.2 C25 8

Figure B 10 C25-8 Mesh and bars position

Figure B 11 C25-8 Disp.Err

Figure B 12 C25-8 Resid.Err.

Figure B 13 C25-8 Res.Abs.E

Figure B 14 C25-8 Energy E

Figure B 15 C25-8 RDisp. Err

Figure B 16 C25-8 RResid. Err

Figure B 17 C25-8 RRes. Abs. E

Figure B 18 C25-8 Renery E.

B.3 C25 25 (BF25-12)

Figure B 19 C25-25 (BF-25-12) Mesh and bars position

Figure B 20 C25-25 (BF-25-12) Disp.Err

Figure B 21 C25-25 (BF-25-12) Resid.Err.

Figure B 22 C25-25 (BF-25-12) Res.Abs.E

Figure B 23 C25-25 (BF-25-12) Energy E

Figure B 24 C25-25 (BF-25-12) RDisp. Err

Figure B 25 C25-25 (BF-25-12) RResid. Err

Figure B 26 C25-25 (BF-25-12) RRes. Abs. E

Figure B 27 C25-25 (BF-25-12) REnergy E.

B.4 BF25-8

Figure B 28 BF-25-8Mesh and bars position

Figure B 29 BF-25-8Disp.Err

Figure B 30 BF-25-8 Resid.Err.

Figure B 31 BF-25-8 Res.Abs.E

Figure B 32 BF-25-8Energy E

Figure B 33 BF-25-8RDisp. Err

Figure B 34 BF-25-8RResid. Err

Figure B 35 BF-25-8RRes. Abs. E

Figure B 36 BF-25-8REnergy E.

B.5: BF25-16

Figure B 37 BF-25-16Mesh and bars position

Figure B 38 BF-25-16 Disp.Err

Figure B 39 BF-25-16 Resid.Err.

Figure B 40 BF-25-16 Res.Abs.E

Figure B 41 BF-25-16 Energy E

Figure B 42 BF-25-16 RDisp. Err

Figure B 43 BF-25-16 RResid. Err

Figure B 44 BF-25-16 RRes. Abs. E

Figure B 45 BF-25-16 REnergy E.

BIBLIOGRAPHY

- [7] A. F. T. E. S. (1999). "The Design, Sizing and Construction of Precast Concrete Segments Installed at the Rear of a TBM." Guidelines for Segmental Lining Ver. 1(Paris 1999, pp.35).
- [8] A.F.T.E.S. (1993). "Segmental gasket ".
- [9] A.F.T.E.S. (1997). "The Design, Sizing and Construction of precast concrete segments installed at the rear of Tunnel Boring machine TBM."
- [10] A.F.T.E.S. (1998). "Working group N7 Temporary and permanent support."
- [11] American Concrete Institute (ACI): "ACI-318 Building Code Requirements for Structural Concrete", Farmington Hills, MI, September, 2011
- [12] American Concrete Institute ACI (1998). "Causes, evaluation and repair of cracks in concrete structures" Aci Committee 224 American concrete institute
- [13] American Association of State Highway and Transportation Officials: "AASHTO LRFD Bridge Design Specifications", Seventh Edition, 2014.
- [14] Ardiaca (2009). "Mohr-Coulomb parameters for modelling of concrete structures." Plaxis Bulletin Spring Issue 2009
- [15] Arnau (2012). "Structural response of precast concrete segmental tunnel lining" PHD Thesis University of Barcelona
- [16] Au T, Baird DL. Bearing capacity of concrete blocks. J Am Concrete Inst Proc 1957;56(9):869–79.
- [17] Babu (2005). "Constitutive modelling of concrete: an overview." Asian Journal of civil Engineering (Buildings and Housing) Vol 6, NO.4 (2005) pages 211-246
- [18] Bairán, J., & Marí, A (2006). "Coupled model for the non-linear analysis of anisotropic sections subjected to general 3D loading. Part 1: Theoretical formulation." Computers & Structures, 84, 2254-2263.

- [19] GL Balázás: “Design for SLS according to fib Model Code 2010”. Structural Concrete. Volume 14, Issue 2, pages 99–123, June 2013.
- [20] Bazant, Z.P, Oh, B.H.: Crack Band Theory for Fracture of Concrete, Materials and Structures, RILEM, Vol. 16, 155-177, 1983.
- [21] Belluzzi (1940). “Scienza delle costruzioni ” Zanichelli Bologna
- [22] Baumann (1992). “Tunnelauskleidungen mit Stahlbetontübbingen.” Bautechnik 69 (1992), H.1, S.8.
- [23] Bowels (1997) Foundation design and analysis, 5th edition McGraw Hill.
- [24] A. Borosnyói and G. L. Balázs. Models for flexural cracking in concrete: the state of the art. Structural Concrete, Volume 6, Issue 2, 01 June 2005, pp. 53 –62.
- [25] Blaber, B Adair, and A Antoniou, "Ncorr: Open-Source 2D Digital Image Correlation Matlab Software." Experimental Mechanics (2015).
- [26] Blom C.B.M. (2003). “The structural (un)safety of tunnels explained by analytical approach.” In Heron, vol. 48, núm. 1, pp. 17-32.
- [27] Blom, C. B. M. (2002). “Design philosophy for concrete linings of tunnels in soft soils.” Ph. D. Thesis, TU Delft University of Technology, Holand.
- [28] Braestrup (1981). “Structural concrete as a plastic material.” Ein Dienst der ETH-Bibliothek Rämistrasse 101, 8092 Zürich, Schweiz.
- [29] BTS (2000). “Specification for Tunneling.” British Tunneling Society,: T.Telford, London 2000, pp.2024-2028.
- [30]BTS (2004). "Tunnel Lining Design Guide." British Tunnelling Society – Institution of Civil Engineers, Thomas Telford 2004
- [31] Busari (2012). “Coupled Stress-Seepage Numerical Design of Pressure Tunnels.” IAHR-HK Student Research Forum, November 17, 2012.
- [32] Cavalaro (2009). “Aspectos tecnológicos de tuneles construidos con tuneladora y dovelas prefabricada de 228oncrete.” PHD Thesis University of Barcelona
- [33] CEN (2004). “ENV 1992: Eurocode-2: Design of concrete structures – Part 1-1: General rules and rules for buildings, .” European Committee for Standardization
- [34] Cervenka, Pukl (1992). “Computer models of concrete structures.” Iabse Structural engineering International Vol2 103-107.

- [35] Červenka, “V. simulating a response”. *Concrete Engineering International*, 4 (4): 45-49, 2000.
- [36] Červenka, V. Computer simulation of failure of concrete structures for practice. 1st fib Congress 2002 Concrete Structures in 21 Century, Osaka, Japan, Keynote lecture, 289-304, 2002.
- [37] Červenka V., Pukl R., Ožbolt, R. & Eligehausen R. Mesh Sensitivity Effects In Smeared Finite Element Analysis Of Concrete Fracture, *FraMCoS-2*, Second International Conference on Fracture Mechanics of Concrete and Concrete Structures, Zürich, Switzerland, 1387-1396, 1995.
- [38] Červenka Vladimír, Jan Červenka, and Libor Jendele. *ATENA Program Documentation – Part 1 – Theory*. Červenka Consulting, Prague, October 2013.
- [39] Chawalit, Thinvongpituk and Somya Poonaya (2008). “A Study of the Ovalisation of Circular Tubes Subjected to Pure Bending.” *World Congress on Engineering*.
- [40] Chen, W. F. and Drucker, D. C., Bearing capacity of concrete blocks or rock, *J. Eng. Mech. Div., Proc. ASCE*, Vol. 95, No. EM4, Aug. 1969, pp. 955–78.
- [41] Chen, W.F.: “Limit Analysis and Soil Plasticity”, Elsevier Scientific Publication Co., Amsterdam, 1975
- [42] Cheehan, L. (2009). “An Investigation of the Effect of Erosion Voids on Existing Tunnels.” Department of Civil Engineering & Applied Mechanics McGill University, Montreal, Quebec, Canada.
- [43] Cividini (2012). “Investigation on the Cause of Damages of a Deep Tunnel.” *International journal of geomechanics Asce* November/December 2012.
- [44] Corradi Dall’acqua (2010). “Meccanica delle strutture Vol III. La valutazione della capacità portante”. McGraw Hill editore.
- [45] Comité Européen de Normalisation (CEN): “EN 206-1: Concrete Part 1: Specification, performance, production and conformity“, 2000.
- [46] DAI (1998). "Convergence properties of nonlinear conjugate gradient methods ".
- [47] Dahl, K.B.:” A failure criterion for normal and high strength concrete”. Technical Report 286, Department of structural Engineering, Technical University of Denmark, 1992
- [48] David Darwin et al. Debate: Crack Width, Cover, and Corrosion. *Concrete International*, Volume 7, No. 5, pp. 20–35, May 1985.
- [49] De Souza Neto, E. A., Peric D., & , Owen , D. R. J. (2008). “Computational Methods for Plasticity.” West Sussex, Wiley.

- [50] Do (2013). "Three-dimensional numerical simulation for mechanized tunnelling in soft ground: the influence of the joint pattern." DOI 10.1007/s11440-013-0279-7.
- [51] Doran (1998). "The use of "Drucker-Prager criterion" in the analysis of reinforced concrete members by finite elements" Digest 1998.
- [52] Doran, S. R., Wood T., Tham S. K., Copsey J. N., Wen D. (2000). "The assessment of limits for the movement of subway tunnels and trackwork due to adjacent construction." Tunnel and underground structures, Balkema, Holand, pp. 495-500.
- [53] Drucker, D. C., Prager, W., (1952). "Soil mechanics and plastic analysis or limit design." Quaterly of Applied Mathematics 10, 157-165.
- [54] ERMCO (2006). "Guidance to the engineering properties of concrete."
- [55] Este G., William, K. "Fracture energy formulation for inelastic behaviour of plain concrete", J. Eng. Mech. 120 (9), 1983-2011, 1994
- [56] European Committee for Standardization (CEN), ENV 1992: Eurocode-2: "Design of concrete structures - Part 1-1: General rules and rules for buildings", 2004.
- [57] fib, buletin No 65 (2012). "Model code 2010 final draft." Vol I Technical report, fib Lausanne.
- [58] Frosch, R. J. Another Look at Cracking and Crack Control in Reinforced Concrete. ACI, Structural Journal, vol. 96 n°3, p. 437-442. Farmington Hills, USA. May-June 1999.
- [59] Georg (2006). "Selection of segmental lining system and repair classification" ITA concrete proceedings 2006.
- [60] Gander, et al. (1994). "Least-Squares Fitting of Circles and Ellipses." This paper appeared in similar form in BIT 34(1994), 558-578.
- [61] Gergely, P. and Lutz, L.A. Maximum Crack Width in Reinforced Concrete Flexural Members. Causes, Mechanism, and Control of Cracking in Concrete, SP-20. American Concrete Institute. Farmington Hills, USA. 1968. pp. 87-117.
- [62] Grippo and S. Lucidi (1997). "A globally convergent version of the Polak-Ribiere conjugate gradient method." Math. Prog., 78 pp. 375-391.
- [63] Gladwell (1980). "Contact Problems in the Classical Theory of Elasticity." Sijthoff & Noordhoff (14.16)
- [64] González, C., and Sagaseta, C. (2001). "Patterns of soil deformations around tunnels. Application to the extension of Madrid Metro." Computers and Geotechnics, 28, 445–468.
- [65] Goto, Y.: Cracks Formed in Concrete Around Deformed Tension Bars. ACI Journal. vol. 68, No. 4. April 1971.

- [66] Gruebl (2005). "Ring Coupling for segmental Linings – old Hat or Necessity?" Tunnel special edition IUT 05.
- [67] Gruebl (2006). "Segmental Rings (Critical Loads and Damage Prevention)." International Symposium on Underground Excavation and Tunnelling 2-4 February 2006, Bangkok, Thailand
- [68] Guerra (2008). "Quais os criterios a ter em conta para a selecção de uma tuneladora". Conferencia presentada en la 3ª Edición Master-Course del Seminario Especializado por el Institute for Internacional Research IIR Portugal de título: "“Selecção, 231oncrete tunnel e gestao de operações com tuneladoras”. Lisboa, 15 y 16 de Mayo 2.008.
- [69] Han (2012). "Monitoring tunnel profile by means of multi-epoch dispersed 3-D LiDAR point clouds." Tunnelling and Underground Space Technology Available online at www.sciencedirect.com.
- [70] Hawkins NM. The bearing strength of concrete loaded through rigid plates. Mag Concrete Res 1968;20:31–40.
- [71] Hefny A.M.; Tan F.C.; Macalevey N.F. (2004). "Numerical study on the behaviour of jointed tunnel lining." Journal of the Institution of Engineers, Singapur, vol. 44, núm. 1, pp. 108-118.
- [72] Hestens, Stiefel (1952). "Methods of conjugate gradients for solving linear systems " Journal of Research of the National Bureau of Standards Vol 49 N. 6.
- [73] Hillerborg, A.; Modéer, M.; Petersson, P-E: Analysis of crack formation and crack growth in concrete by means of fracture mechanics and finite elements, Cement and Concrete Research, Vol 6, pp. 773-782, 1976.
- [74] Hordijk, D.A.: Local Approach to Fatigue of Concrete, Doctor dissertation, Delft University of Technology, The Netherlands, 1991.
- [75] Hsieh , S. S., Ting , E. C., & , Chen, W. F. (1982). "A plasticity-fracture model for concrete." International Journal of Solids and Structures, 113, 577-593.
- [76] Hsu (2010). "Unified Theory of concrete structures." University of Houston, USA.
- [77] Hu , H. T., & , Schnobrich , W. (1989). "Constitutive Modeling of Concrete by Using Nonassociated Plasticity." Journal of Material of Civil Engineering, 1.
- [78] Iasiello, C. Pérez Caldentey, A., Groli, G. Analysis of TBM lining rings under lack of gap fill: ovalisation and crack patterns. A practical case. Journal of Performance of Constructed Facilities (DOI: 10.1061/(ASCE)CF.1943-5509.0001010 Vol. 31, Issue 4 (August 2017)

- [79] Ince and Arici: "Size effect in bearing strength of concrete cubes", Construction and Building materials, October 2004.
- [80] Imran, I. and S.J. Pantazopoulou. "Experimental Study of Plain Concrete under Triaxial Stress." ACI Materials Journal 93 (6) (1996): 589-601.
- [81] ITA (1999). "Guidelines for Design of Shield Tunnel Lining." Working Group Research.
- [82] ITAtech (2014). "ITAtech Guidelines on Best Practices For Segment Backfilling." N° ISBN : 978-2-9700858-5-0.
- [83] Janssen, P. (1983). "Tragverhalten von Tunnelausbauten mit Gelenktübbings." Report-No. 83-41 University of Braunschweig, Department of civil engineering, Institute for structural analysis.
- [84] JSCE (2006). "Standard Specifications for tunnelling: shield tunnels ".
- [85] Kasper (2007). "Lining design for the district heating tunnel in Copenhagen with steel fibre reinforced concrete segments." Tunnelling and Underground Space Technology Available online at www.sciencedirect.com.
- [86] Kawashina, Obinata, Kano Shiba (1998). Experimental study on seismic resistance of reinforced concrete lining of shield tunnels. Proceedings of the 9th world conference on earthquake engineering Tokyo-Kyoto Japan Vol II.
- [87] Kolic D., A., Mayerhofer (2009). "Segmental lining tolerances and imperfections" ITA WTC 2009 Symposium Budapest May 23-28, pp8.
- [88] Kolic (2000). "Quality recommendations on segmental lining for shield drive in hard rock." Int. conference on tunnels and underground structures (eds), Proceedings: 355-362. Singapore: Balkema.
- [89] D. Kolic , A., Mayerhofer (2009). "Segmental lining tolerances and imperfections" ITA WTC 2009 Symposium Budapest May 23-28, pp8.
- [90] Kolmar, W.: Beschreibung der Kraftübertragung über Risse in nichtlinearen Finite-Element-Berechnungen von Stahlbetontragwerken", Dissertation, T.H. Darmstadt, p. 94, 1986.
- [91] Kupfer J., Hilsdorf, H.K. and Rüsch H.: "Behavior of concrete under biaxial stresses". Journal of the American Concrete Institute, 66(8):656-666, 1969
- [92] Lee K.M.; Ge X.W. (2001). "The equivalence of a jointed shield-driven tunnel lining to a continuous ring structure." Canadian Geotechnical Journal, vol. 38, núm. 3, pp. 461-483.
- [93] Lee K.M.; Hou X.Y.; Ge X.W.; Tang Y. (2001). "An analytical solution for a jointed shield-driven tunnel lining." International Journal for Numerical and Analytical Methods in Geomechanics, vol. 25, pp. 365-39

- [94] Leone, Corradi dell'acqua (1994). "Meccanica delle strutture. Vol3: la valutazione della capacità portante." McGraw-Hill.
- [95] Leonhardt, F., Reimann, H., (1966). "Betongelenke." Der Bauingenieur.
- [96] Leonhardt. Fritz Vorlesunger über Massivbau. 1973. Springer Verlag, Berlin/heidelberg.
- [97] Leonhardt Fritz and E. Mönning. Vorlesungen über Massivbau: Teil 2 Sonderfälle der Bemessung im Stahlbetonbau. Springer, 1975.
- [98] Liao (2012). "Identifying TBM Tunneling Defects through Recognition of Ground Settlement Patterns." GeoCongress 2012 © ASCE 2012.
- [99] Linero, D. L., & , Garzón-Alvarado , D. A. (2010). "Elementos de la mecánica del medio continuo para cuerpos sólidos. Volumen 1: Temas Básicos." Bogotá, Universidad Nacional de Colombia.
- [100] Lombardi (1973). "Dimensioning of tunnel linings." Tunnels and Tunnelling 1973, pages 340 – 351
- [101] Lombardi , Amberg (1974). "An elasto-plastic analysis of the stress-strain state around an underground opening." Part II, 3rd Congress of ISRM, Denver.
- [102] Lombardi (1974). "Une méthode de calcul élastoplastique de l'état de tension et de oncrete tu autour d'une cavité souterraine." Congresso Internazionale ISRM, Denver.
- [103] Lombardi (2007). "Evoluzione delle costruzione di opere sotterranee." Colloquia doctoralia 12 Luglio 2007.
- [104] Luna, M. (2013). "Simulación del colapso en túneles dovelados mediante la formación de articulaciones en vigas." Concreto y cemento. Investigacion y desarrollo vol . 4, núm. 2, de la pág. 43 a la pág. 55.
- [105] Luttikholt, A. (2007). "Ultimate Limit State Analysis of a Segmented Tunnel Lining." Tu Delft University of Technology, Holand.
- [106] Maidl (1995). "Mechanised Shield Tunnelling." Ernst&Sohn, Berlin.
- [107] Maidl (2014). "Handbook of tunnel engineer (Vol I and II)." Ernst and Sons, Berlin.
- [108] Maisinobe (2007). "Finding the circle that best fits a set of points."
- [109] Marchetto Francesco, "Use of Headed reinforcement bars in construction. A theoretical approach to determine the dimensions of the anchorage plates and experimental tests on knee joints subjected to a closing moment". PhD Thesis, Polytechnic University of Madrid, 2015.
- [110] Marchetto Francesco, Alejandro Pérez Caldentey, Hugo Corres Peiretti: "Structural performance of corner joints subjected to a closing moment using mechanical anchorages: an experimental study", Structural Concrete, Vol. 17, p. 982-1002, 2016.

- [111] Marinos, P., and , Hoek, E. (2001). “Estimating the geotechnical properties of heterogeneous rock masses such as flysch.” *Bull. Enginng Geol. & the Environment (IAEG)*, 60, 85-92.
- [112] Mashimo (2008?). “Numerical modelling of the 234oncrete of shield tunnel lining during assembly of a tunnel ring.”
- [113] Mashimo, H. I., T. (2003). “Valuation of the load on shield tunnel lining in gravel.” *Tunnelling and underground space technology*, Vol.18: 233-241.
- [114] Mattock, A. H., and Hawkins, N. M., 1972, “Shear Transfer in Reinforced Concrete—Recent Research,” *PCI Journal*, V. 17, No. 2, Mar.-Apr., pp. 55-75
- [115] Menetrey , P. H., & , Willam , K. J. (1995). “Triaxial failure criterion for concrete and its generalization.” *ACI Structural Journal*, 92, 311-318.
- [116] Meyerhof GG. The bearing capacity of concrete and rock. *Mag Concrete Res* 1953;4(12):107–16.
- [117] Mindlin, R. D. (1939). “Stress distribution around a tunnel.” *Proceedings of the American Society of Civil Engineers*, Vol. 65, No. 4, pp. 619-642.
- [118] Ministerio de Fomento: “Instrucción de hormigón estructural EHE08”, 4a Edición Centro de Publicaciones, 2010.
- [119] Montoya (2000). “Modeling of confined concrete.” PhD Thesis University of Toronto
- [120] Montoya, Vecchio, Sheik: “Compression Field Modeling of confined concrete: Constitutive models”, *Journal of materials in civil engineer*, Vol. 18, N4, ISSN 0899-1561/2006/4-510-517
- [121] Muir Wood, A. M. (1975). “The circular tunnel in elastic ground.” *Géotechnique* Volume 25, Issue 1, 01.03.1975, pages 115-127, ISSN: 0016-8505
- [122] National Cooperative Highway Research Program (NCHRP). Verification and Implementation of Strut-and-Tie Model in LRFD Bridge Design Specifications. Requested by: American Association of State Highway and Transportation Officials (AASHTO).
- [123] Nielsen, M.P., Hoang L.C. (1984). “Limit Analysis and concrete plasticity”. Third Edition, CRC Press Taylor & Francis Group.
- [124] Niyogi SK. Concrete bearing strength-support, mix, size effect. *J Struct Div-ASCE* 1974;100:1685–701.
- [125] Niyogi SK. Bearing strength of concrete—geometric variation. *J Struct Div-ASCE* 1973;99:1471–90.

- [126] Nuttens (2012). "Application of laser scanning for deformation measurements: a comparison between different types of scanning instruments." FIG Working Week 2012 Rome, Italy, 6-10 May 2012.
- [127] Lorenzen, K. G., Koncentreret last på beton [Concentrated loads on concrete], MSc thesis, Department of Structural Engineering, Technical University of Denmark, 1995.
- [128] Lorenzen, K. G. and Nielsen, M. P., Koncentreret last på beton [Concentrated loads on concrete], Department of Structural Engineering and Materials, Technical University of Denmark, Report R, No. 34, 1997.
- [129] Ottosen, N. S. (1977). "A failure criterion for concrete." *Journal of Engineering Mechanics ASCE*, 103, 527-535.
- [130] Peck, R. B. (1969). "Deep Excavations and Tunnels in Soft Ground". *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State of the Art Volume*, pp. 225-290.
- [131] Pelizza (2010). "Analysis of the Performance of Two Component Back-filling Grout in Tunnel Boring Machines Operating under Face Pressure." *Proceedings of ITA-AITES World Tunnel Congress 2010: "Tunnel vision towards 2020"*, Vancouver, 14-20 May 2010.
- [132] Peña, et al (2012). "Comportamiento estructural de juntas entre dovelas de concreto prefabricado para túneles." *Concreto y cemento. Investigación y desarrollo Vol III Num 2*.
- [133] Pérez, A.; Corres, H.; Giraldo, A; Peset, J. (2013): Cracking of RC members revisited. Influence of cover, ϕ/ρ_s , ρ_s and stirrup spacing. An Experimental and Theoretical Study. *Structural Concrete*, 2013.
- [134] Pérez, A.; Corres, H.; Petschke, T.; Ezeberry, J.; Giraldo, A, (2012) Serviceability design of columns of long jointless structures, *Engineering Structures*, Vol. 44, 2012
- [135] Pérez Caldentey Alejandro, Francesco Marchetto, Hugo Corres Peiretti, Jesús Iglesias Villareal: "Plate-anchored reinforcement bars: A new simple and physical model for practical applications", *Engineering Structures*, Vol. 52, p. 168-178, 2013.
- [136] Rabczuk (2004). "A numerical model for reinforced concrete structures." *International Journal of Solids and Structures* 42 (2005) 1327–1354.
- [137] Rashid, Y. R. (1968). "Analysis of prestressed concrete pressure vessels." *Nuclear Engrg. And Design* 7(4), 334-355.
- [138] Rezanian (2012). "Non-local regularisation of Drucker-Prager Softening model " *Proceedings of the 20th UK Conference of the Association for Computational Mechanics in Engineering* 27 – 28th March 2012, the University of Manchester, Manchester.

- [139] Ribacchi (1991). "Stato di forza e deformazione intorno ad una galleria." L'ingegnere numero unico.
- [140] RiLi 853: Einsenbahntunnel planen, bauen und instand halten. In DB Netz AG, Dezember 2008
- [141] Rizkalla S.H., M.EL Shahawi, and C.K. Kwok. Cracking behavior of reinforced concrete members. Canadian Society for Civil Engineering (CSCE). Proceedings from 1982 Annual Conference.
- [142] Sahmaran, Mustafa; Li, Victor C.; Andrade, Carmen. Corrosion Resistance Performance of Steel-Reinforced Engineered Cementitious Composite Beams. ACI Material Journal, vol. 105 n°3, p. 243. Farmington Hills, USA. May-June 2008.
- [143] Sagaseta (1987). "Analysis of undrained soil deformation due to ground loss." Géotechnique 37, 301±320.
- [144] Sargin M, Handa V (1971). "A general formulation for stress strain properties of concrete". Report No3. S. Solid Mechanics Division, Study, University of Waterloo 1-27.
- [145] Schlaich Jörg, Kurt Schäfer, and Mattias Jennewein. Toward a consistent design of structural concrete. Technical Report Vol. 32 No. 3, PCI Journal, May-June 1987.
- [146] Shelson W. Bearing capacity of concrete. J Am Concrete Inst Proc 1957;54(5):405–14.
- [147] Shewchuk (1994). "An Introduction to the Conjugate Gradient Method Without the Agonizing Pain." School of Computer Science Carnegie Mellon University Pittsburgh, PA.
- [148] Shorland (1987). "Innovative methods of segmental concrete tunnel lining." Ein Dienst der ETH-Bibliothek ETH Zürich, Rämistrasse 101, 8092 Zürich, Schweiz, www.library.ethz.ch
- [149] Spieth (1959), Spieth H.P., Das Verhalten von Beton unter hoher örtlicher Pressung und Teilbelastung unter besonderer Berücksichtigung von Spannbetonverankerungen, Dissertation, Technial University of Stuttgart,
- [150] Schreyer J.; Winselmann D. (1998). "Suitability tests for the segmental lining for the 4th Elbe tunnel tube, Hambrug." Tunnel 2/98, pp. 30-37, 1998.
- [151] Schreyer J.; Winselmann D. (2000). "Suitability tests for the lining for the 4th Elbe tunnel tube – Results of large-scale tests." Tunnel 1/2000, pp. 34-44,.
- [152] STUVA (2001). "Erarbeitung von Kriterien für den Einsatz von maschinellen Vortriebsverfahren im Tunnelbau." Stand September 2001.
- [153] STUVA: "Eignungsprüfungen 4. Röhre Elbtunnel. Versuche zur Verdrehsteifigkeit" September 1996, F4R-KE62.

- [154] Sugimoto (2006). "Causes of shield damages during construction." International Symposium on Underground Excavation and Tunnelling 2-4 February 2006, Bangkok, Thailand.
- [155] Széchy (1970). "The art of tunnelling".
- [156] U.S., Department of Transportation (2009). "Technical Manual for Design and Construction of Road Tunnels —Civil Elements." Publication No. FHWA-NHI-10-034.
- [157] Tomoya (2009). "Excavation management with use of shield machine in rapidly changing soil condition." Proceedings of the ITA-AITES World Tunnel Congress 2009 "Safe Tunnelling for the City and Environment", Budapest, 23-28 May 2009.
- [158] Thinvongpituk and Poonaya (2008). "A Study of the Ovalisation of Circular Tubes Subjected to Pure Bending" American Institute of Physics
- [159] Tiberti (2008). "Concrete tunnel segments with combined traditional and fiber reinforcement." Tailor Made Concrete Structures – Walraven & Stoelhorst (eds) © 2008 Taylor & Francis Group, London, ISBN 978-0-415-47535-8.
- [160] Van der Vliet C. (2006). "Langsvoeggedrag op basis van de elasticiteitstheorie, Een aanscherping van de Janssen-relatie." Bouwdienst Rijkswaterstaat.
- [161] Van der Waart van Gulik, T. G. (2010). "Snap through of large shield driven tunnels." Tu Delft University of Technology, Holand.
- [162] Van Oorsouw, R. S. (2010). "Behaviour of segment joints in immersed tunnels under seismic loading." Final thesis Delft, June 2010.
- [163] Van Mier, J.G.M. "Fracture of Concrete under Complex Stress." Heron 31 (3) 1986
- [164] Vecchio (1990). "Reinforced concrete membrane element formulations." Journal of structural engineering ASCE pag 730-750.
- [165] Vecchio, De , Roe (1995). "Smeared-crack modelling of concrete tension splitting." Journal of engineering mechanics June 1995. .
- [166] Verruijt, A. (1996) "Complex variable solutions of elastic tunnelling problems," Geotechnical Laboratory, Delft University of Technology.
- [167] Verruijt, A. (1997) "A complex variable solution for a deforming tunnel in an elastic half-plane," International Journal for Numerical and Analytical Methods in Geomechanics, 21, 77-89.
- [168] Verruijt, A. & Booker, J. R. (1996) "Surface settlements due to deformation of a tunnel in an elastic half-plane," Géotechnique, 46(4), 753-756
- [169] Verruijt (2008). "Buoyancy of tunnels in soft soils." Géotechnique 58, No. 6, 513–515.

- [170] Willam, K. J., & Warnke, E. P. (1974). "Constitutive model for triaxial behaviour of concrete. Concrete Structures Subjected to Triaxial Stress." Bergamo, Italy, International Association for Bridges and Structural Engineering.
- [171] Wittke (2006). "Stability Analysis and Design for Mechanized Tunnelling." WBI, Felsbau GmbH, Aachen
- [172] Xiaochun Z.; Wei Z.; Zhengrong H.; Yuewang H. (2006). "Effect of joint structure on joint stiffness for shield tunnel lining." Tunnelling and Underground Space Technology, vol. 21, pp. 406-407.
- [173] Yoon (2008). "Feature extraction of a concrete tunnel liner from 3D laser scanning data." NDT&E International.
- [174] Zerpa, and Colmenares (2004). "Optimización para ingenieros " Facultad de Ingeniería Instituto de Cálculo Aplicado.
- [175] Zhang, Zhou Li (2006). "A descent modified Polak–Ribiere–Polak conjugate gradient method and its global convergence." IMA Journal of Numerical Analysis (2006) 26, 629–640.