# Recommendations on seismic actions on port structures

J. Llorca
*Dirección General de Puertos, Ministerio de Obras Públicas y Transportes (MOPT), Spain*

C. Huerta, J.J. Lopez-Cela & E. Laso
*Sociedad de Investigación, Estudios y Experimentación (SINEX S.A.), Spain*

E. Alarcón
*ETS Ingenieros Industriales, Universidad Politécnica de Madrid (UPM), Spain*

ABSTRACT: The paper describes the main features of a Technical Recommendation first draft on Seismic Action on port structures promoted recently by the Spanish Ministry of Public Works (MOPT).

## 1 INTRODUCTION

The Dirección General de Puertos (DGP) of the Spanish Ministerio de Obras Públicas y Transportes (MOPT) has embarked in the composition of up-to-dape Recommended Actions for the design of coastal works. A first publication concerning general rules, load combination hypothesis, risk assesment etc have already been published (ref.1). Other studies are currently being conducted on special matters: wave action, soil mechanics, etc.

The objective of this paper is to present a summary of the first draft on Seismic Actions to be included in a forthcoming new publication.

The main difficulty of the approach has been the comprehensive nature of the activities developed in a port, i.e.: the seismic action can affect structures with many different objectives so different design methods and safety levels have to be summarized in a modest amount of rules.

On the other hand, during the past 20 years the state of the art has progressed enormously and the same happened with the computational methods available to the designer so that, depending on the importance of the structure, different levels of refinement should be adopted by the analyst. The Recommendations have to be considered as minimal requeriments when specifiying a computational method and as a global field in which to act when they fix the characteristics of the action or treat general problems in which an standard accepted by the profession is not yet available. For instance, when the importance of topographic features in modifying the seismic action has to be assesed.

Although there are lots of subjects where a new reasearch is needed it was felt that a regulatory effort could be useful to help, at least, to establish a common ground of discussion among Administrative Authorities and Designers.

## 2 SEISMIC ACTION

In Spain the current regulation (ref.2) is generally considered out-of-date so that several studies have been performed to analize the sismogenetic regions (ref.3, 4, 5, 6) from an engineering viewpoint. The approaches have been probabilistic following the path shown by Cornell and others in the sixtyes. Finally a map has been recently proposed (ref.7), figure 1, that establishes for a site the maximum acceleration that can be expected for a perior of 50 years with a probability of about 10 % (seismic hazard), that is that basic return period considered to build the map has been 500 years. Also a special effort has been devoted to differentiate the effects of the local seismicity from that originated at long distances (the Azores-Gibraltar fault)(ref.6).

As it is well know the seismic hazard has to be composed with the structure vulnerability and the value of the structure to get the risk. The practical approach is to proceed the other way around i.e.: once the value of the structure and the permissible risk have been established an importance factor is defined as the ratio of the so-called design acceleration to the basic acceleration defined in the map. The idea is to

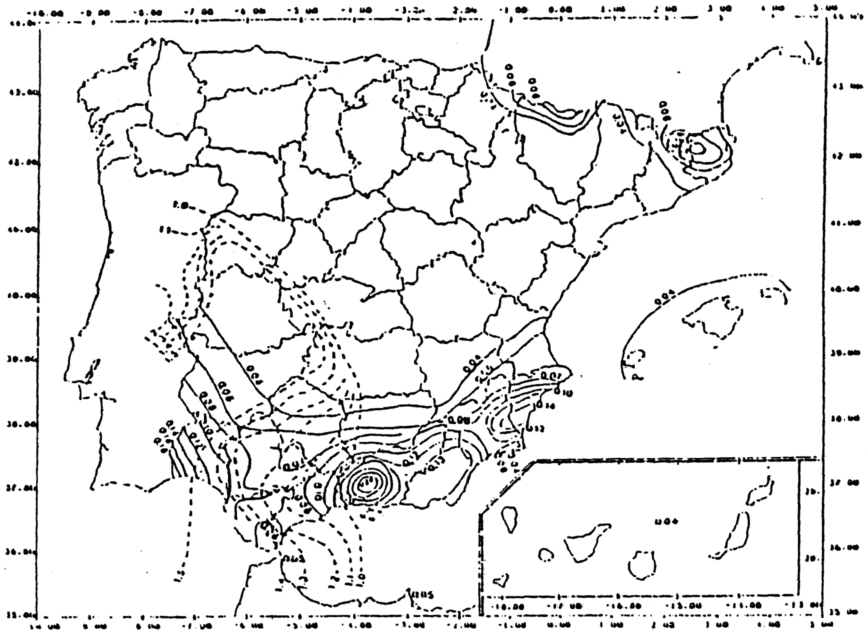

Figure 1

accept a rule of the type (Whitman an Cornell 1976)

$$\frac{1}{T} = \frac{\alpha}{a^{k}} \qquad (1)$$

where T is the return period, a the soil acceleration and $\alpha$ and k parameters related to the site. By applying eq.1 to the specific situation and the basic map definition and using the approximate relation

$$E = \frac{L}{T} \qquad (2)$$

between the risk E, the structure expected life L and the return period T it is easy to show that the importance factor can be defined by the ratio

$$\frac{a}{a_b} = \left(\frac{L}{T_b}E\right)^{\frac{1}{k}} \qquad (3)$$

As above said in the spanish basic map $T_b$ = 500 years and k (ref.9) is taken as 2.7.

L and E can be fixed using tables I and II (ref.1). An application to an specific case can be seen in a companion paper (ref.10).

An important aspect in maritime works where the construction phase may have an important duration is the establishment of a design acceleration for it. Although reference 1, contains several reccomendations a practical rule can be the election of a return period producing during the construction period the same risk than that accepted for the whole life of the structure.

In some occasions other parameters of the soil are needed: velocity to estimate the displacements of retaining walls, displacements to compute the minimum support length for bridge abutments or typical wave lengths to take into account the possibility of multiple support excitations. Those quantities are fixed (table III) according to the spectrum shape and different soil conditions (ref.12). The shape of the elastic spectrum follows the proposal contained in reference 7. The normalized acceleration is given for a damping ratio of 5 % by the rules

$$\alpha(T) = 1 + [\alpha(T_0) - 1]\frac{T}{T_0} \quad if\ 0 < T < T_0$$

$$\alpha(T) = \alpha(T_0) \qquad T_0 \le T \le T_1$$

$$\alpha(T) = \alpha(T_0)\frac{T_1}{T_0} \qquad T > T_1$$

where

$$\alpha(T_0) = (3C - 3,8)\ (K - 1,25) + 2,3$$
$$T_0 = 0,125C + 0,2K - 0,175$$
$$T_1 = 0,215K \frac{(5C - 1)}{\alpha(T_0)} \qquad (5)$$

and C takes the values 1, 1,4 and 1,8 for 3 stadardized soil types that follow the general pattern established in the Eurocode 8 (ref.11) while K takes into account the influence of the events originated in the Azores-Gibraltar fault (dashed lines in figure 1).

MINIMUM DESIGN LIVES FOR WORKS OR STRUCTURES OF DEFINITIVE CHARACTER (in years)

| TYPE OF WORK OR INSTALLATION | REQUIRED SECURITY LEVEL | | |
|---|---|---|---|
| | LEVEL 1 | LEVEL 2 | LEVEL 3 |
| GENERAL USE INFRASTRUCTURE | 25 | 50 | 100 |
| SPECIFIC INDUSTRIAL INFRAESTRUCTURE | 15 | 25 | 50 |

LEGEND:

GENERAL USE INFRASTRUCTURE:
General character works: not associated with the use of an industrial installation or of a deposit.

SPECIFIC INDUSTRIAL INFRASTRUCTURE:
Works in the service of a particular industrial installation or associated with the use of transitory natural deposit) of resources (e.g. industry service port, loading platform of a mineral deposit, petroleum extraction platform; etc.).

LEVEL 1:
Works and installations of local or auxiliary interest. Small risk of loss of human life or environmental damage in case of failure.
(Defense ad coastal regeneration works, works in minor ports of marinas, local outlets, pavements, commercial installations, buildings, etc.).

LEVEL 2:
Works and installations of general interest.
Moderate risk of loss of human life or environmental damage in case of failure.
(Works in large ports, outlets of large cities, etc.).

LEVEL 3:
Works and installations for protection against innundations or international interest. Elevated risk of human loss or environmental damage in case of failure.
(Defense of urban or industrial centers, etc.).

Table I

MAXIMUM ADMISSIBLE RISKS TO DETERMINE CHARACTERISTIC VALUES OF VARIABLE LOADS IN THE SERVICE PHASE AND EXTREME CONDITIONS OBTAINED FROM STATISTICAL DATA

a) DAMAGE INITIATION RISK

| ECONOMIC REPERCUSSION IF WORK IS DISABLED. Index: Cost of losses / investment | | POSSIBILITY OF HUMAN LOSS | |
|---|---|---|---|
| | | REDUCED | EXPECTED |
| | LOW | 0.50 | 0.30 |
| | AVERAGE | 0.30 | 0.20 |
| | HIGH | 0.25 | 0.15 |

b) TOTAL DESTRUCTION RISK

| ECONOMIC REPERCUSSION IF WORK IS DISABLED. Index r: Cost of losses / investment | | POSSIBILITY OF HUMAN LOSS | |
|---|---|---|---|
| | | REDUCED | EXPECTED |
| | LOW | 0.20 | 0.15 |
| | AVERAGE | 0.15 | 0.10 |
| | HIGH | 0.10 | 0.05 |

The damage initiation or total destruction risk shall be adopted as maximum admissible risk according to its deformation characteristics and ease of repair of the resistant structure. For rigid or fragile works without possibility of repair, the total destruction risk shall be adopted.
For flexible, semi-rigid, or generally repairable works, (damages less than a preset level in terms of the structural type) the damage initiation risk shall be adopted. In these types of works, the total destruction risk shall also be adopted, defining the damage level for each structural type to be considered as total destruction. The resulting action shall be considered accidental.

LEGEND:

- POSSIBILITY OF HUMAN LOSS
  - Reduced: When human loss is not expected in case of failure or damage.
  - Expected: When human loss is foreseeable in case of failure or damage.
- ECONOMIC REPERCUSSION IF WORK IS DISABLED

$$\text{Index } r = \frac{\text{Cost of indirect or direct losses}}{\text{investment}}$$

- LOW: $r \leq 5$
- AVERAGE: $5 < r \leq 20$
- HIGH: $r > 20$

Table II

Tabla III. Values normalized to 1 g acceleration.

| | max.soil velocity | max.soil displac. | walk length |
|---|---|---|---|
| SOIL TYPE | V (cm/sec) | $d_0$ (cm) | λ (m) |
| I | 60 | 45 | 600 |
| II | 100 | 70 | 500 |
| III | 140 | 100 | 300 |

The design spectrum contained in the reccomendation takes into account the accepted ductility of the structure through a behaviour factor q dividing $\alpha(T_0)$ and modifies the spectrum for $T > T_1$ in order to correct the inherent uncertainties in the long period range. In addition a limitation is established on the minimum design values. The final reccomendation is

$$\alpha(T) = 1 + \left[\frac{\alpha(T_0)}{q} - 1\right]\frac{T}{T_0} \qquad 0 < T < T_0$$

$$\alpha(T) = \frac{\alpha(T_0)}{q} \qquad T_0 \leq T \leq T_1$$

$$\alpha(T) = \frac{\alpha(T_0)}{q}\left(\frac{T_1}{T}\right)^{\frac{2}{3}} \qquad T > T_1$$

$$\alpha(T) \not< 0,2\,\alpha(T_0) \qquad (6)$$

For vertical accelerations a ratio of 2/3 of the above values is taken.

In case of slender structures like towers, chimneys, etc. a rotation spectrum can be derived from the eq.6 expresions following the rules that follow:

Rotation around the two horizontal axis

$$\theta_{x,y}(T) = \frac{1,7\pi}{\lambda}\,\alpha(T)$$

Rotation around the vertical axis

$$\theta_z(T) = \frac{2\pi}{\lambda}\,\alpha(T) \qquad (7)$$

where $\lambda$ is the wave length dominant in the soil type foundation.

An envelope spectrum is recommended when several different soils affect the structure and a multiple support excitacion is not feasible.

Also different reccomendations for power spectral densities and numerical simulation of time histories are defined following the current state of the art (ref.12).

A mention is also given to posible amplifications due to the soil layering or to topographical features at the site.

## 3 COMPUTATIONAL METHODS

The Recommendations establish general rules to analize the structures following the lines of direct integration, frequency response and modal methods. Among the last one the spectral modal methods are given more detailed treatment specially in what refers to the truncation problem and the superposition of partial responses to get the general one.

The truncation process is recommended to be controlled by the computation of the portion of the mobilized mass that is included in the considered approach, i.e., if the motion equations are

$$\underline{m}\,\ddot{\underline{x}} + \underline{c}\,\dot{\underline{x}} + \underline{k}\,\underline{x} = -\,\underline{m}\,\underline{J}\,\ddot{x}_s \qquad (8)$$

where

$\underline{m}$ is the mass matrix
$\underline{c}$ is the damping matrix
$\underline{k}$ is the stiffness matrix
$\underline{x}$ is the displacement vector
$\underline{J}$ is the influence vector

The total mass mobilized by the imposed base motion is

$$M = \underline{J}^T\,\underline{m}\,\underline{J} \qquad (9)$$

and that corresponding to mode j can be written al

$$M_j = \Gamma_j^2\,(\underline{\Phi}_j^T\,\underline{m}\,\underline{\Phi}_j)$$

$$\Gamma_j = \frac{\underline{\Phi}_j^T\,\underline{m}\,\underline{J}}{\underline{\Phi}_j^T\,\underline{m}\,\underline{\Phi}_j} \qquad (10)$$

where $\underline{\Phi}_j$ is the mode-shape j and $\Gamma_j$ the so-called participation factor. The criterion is then to reach at least the condition

$$\sum_1^N M_j \geq 0,90\,M \qquad (11)$$

where N is the number of modes that are included in the analysis. Although the condition can be not sufficient in some problems (see ref.13) it can at least be considered as a minimum requirement to get reasonable answers.

The Recommendations also allow the use of the modal acceleration method to define a "residual mode" to correct the "mised" mass using a static approach to the high frequency contributions along a line similar to that presented in ref.14. Finally an "equivalent static force" analysis is allowed using static deformations $\underline{d}$. The approximation

$$x = A\,d \qquad (12)$$

where A is a parameter to be determined, in equation (9) and the premultiplication by $\underline{d}^T$ produces the estimate of the frequency

$$\omega^2 = \frac{d^T k\, d}{d^T m\, d} \qquad (13)$$

and the 1 d.o.f. system

$$\ddot{A} + 2\,\zeta\,\omega\,\dot{A} + \omega^2 A = -\frac{d^T m\, J}{d^T m\, d}\,\ddot{x}_s$$

where damping ratio has been introduced formally alike with the modal approach. The equivalent force is then, using the acceleration spectrum,

$$F_{eq} = a\,\alpha(\omega,\zeta)\,\frac{d^T m\, J}{d^T m\, d}\,(m\,d) \qquad (15)$$

In some occassions the static displacement $\underline{d}$ is computed as part of routine calculations using properties of the structure "in service" (for instance: non fissured section properties in concrete structures) while the earthquake situation would need the use of damaged properties. From a strength approach the situation is generally conservative and this is why the Recommendations allow the use of the static response to estimate dynamic forces.

## 4 MISCELLANEOUS DETAILS

The Recommendations include simplified procedures to estimate the Densification and Licuefaction properties of soils acording to the state of the art knowledge. For the dynamic earth-pressure on retaining walls an inverted triangular law is selected and the dynamic action of submerged water is considered to be included in the saturated weigth (figure 2).

$$\theta' = \operatorname{arctg}\frac{k_h}{1-k_v}\,\frac{\gamma_{sat}}{\gamma_{sum}}$$

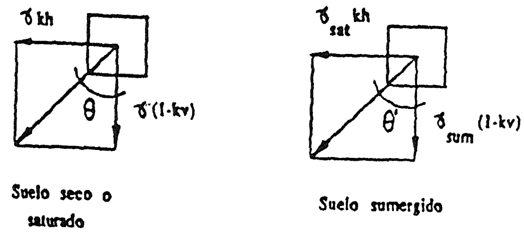


Figure 2

In addition a Westergaard formule can be used to represent the action of free water. A general example can be seen in figure 3 where the following pressures are defined

1. Westergaard hydrodynamic pressure.
2. Hydrostatic pressure.
3. Dynamic passive pressure.
4. Underpressure.
5. Active static pressure.
6. Hydrostatic pressure.
7. Complementary dynamic active pressure.

For slope stability an equivalent linear method is proposed to estimate the displacements following the classical Newmark aaproach: they are obtained through a double integration of the accelerations obtained through a weigthed average inside an assumed mass failure. The limit acceleration (fig. 4) is that value of the horizontal coefficient $k_h$ producing a safety factor of 1. The Yegian formulae (ref.15) is also included as a means to estimate the importance of the effect and the need of more refined estudies.

Soil-structure interaction formula are recommended following the line proposed by Gazetas (ref.16) and for the underground pipes the Kuesel (ref.17) approach has been recommended, while dynamic modelling of tanks follows the classical lines of Housner and Haroum (ref.18,19).

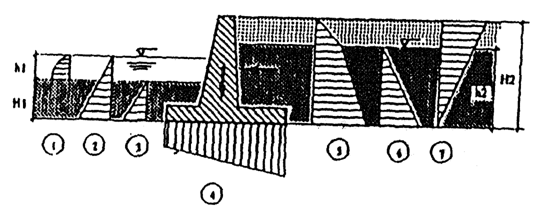

Figure 3

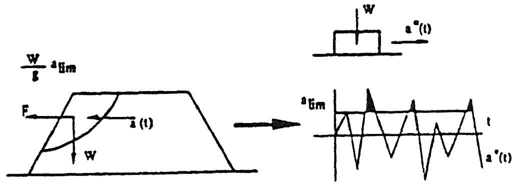

Figure 4

## 5 CONCLUSSIONS

Although much more research is needed to clarify the seismic behaviour of the vast class of problems present in port structures the current state of the art allows at least a classification of subjects and the establishment of minimum requirements to guide the design. Also the use of more refined methods for specially dangerous situations needs some general guidelines that contribute to mantein the design under reasonable safety margins.

The Recommendations of the Spanish MOPT are a first try in those directions.


## 6 ACKNOWLEDGMENTS

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