

## **SEISMIC PERFORMANCE, ANALYSIS AND DESIGN OF WHARF STRUCTURES: A COMPARISON OF WORLDWIDE TYPOLOGIES.**

**Randolph Carl BORG<sup>1</sup> and Carlo Giovanni LAI<sup>2</sup>**

### **ABSTRACT**

This article is a brief overview of some of the typologies of wharf structures used worldwide and of their performance in relation to earthquake loading. The subject is so vast and branched that a comprehensive illustration of the findings of the study would require a state-of-the-art journal paper or even a book. In this paper the focus is concentrated in presenting a few data on the relative economical importance of seaports worldwide and their associated seismic hazard. An attempt was also done to identify some common typologies of wharves adopted in some regions of the world and to classify them in terms of their seismic performance. Attention was devoted to seaports in Greece, Turkey, Italy, U.S.A., Japan and China. It then follows a brief discussion on design methodologies adopted in Japan, North America and Europe illustrated through an example focused on the port of Gioia Tauro in Southern Italy.

Keywords: Wharf structures, Port structures, Retaining walls, Pseudo-static design.

### **HAZARD AND RISK CONSIDERATIONS**

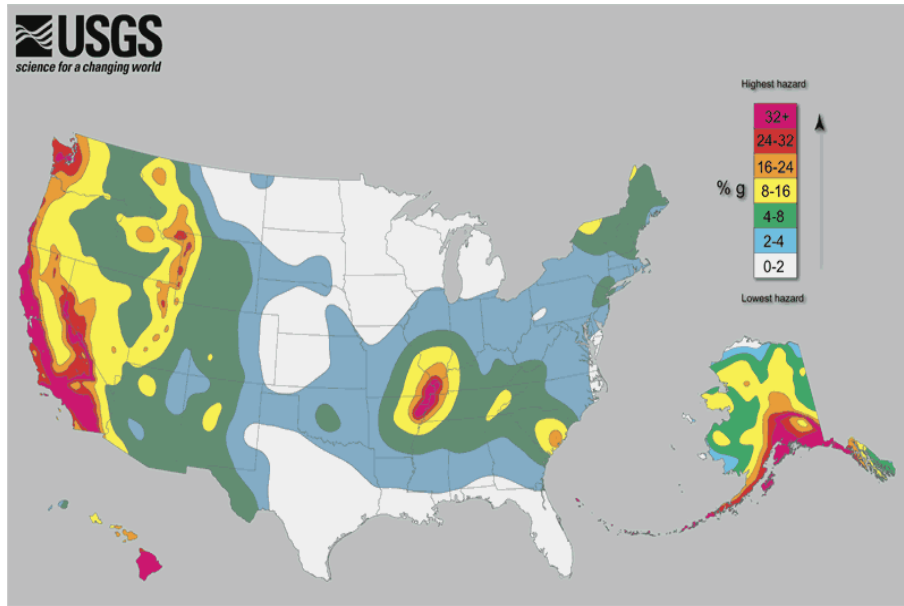
The economies of many industrialised nations are based on the export and import of goods and/or on the flow of travellers in the tourism industry. A relevant portion of this trade takes place and relies on the efficiency of seaports. Many of such ports are located in zones characterised by moderate to high seismicity. In recent history, mooring wharf facilities have been damaged to a varying degree by seismic events to a detriment of the countries' economy. The design PGA for 10% probability of exceedence in 50 years of a number of important ports in USA, Japan, New Zealand and China (PMHW, 1998); and in some seismic European countries (ESPO,2005) namely Turkey, Italy and Greece was determined by comparing the geographical position of the ports with their location in the corresponding hazard maps, as shown in Figure. 1 to 5 (GSHAP, 1999; EAK, 2000; METU, 2006; OPCM-3519, 2006; USGS, 2006).

The cumulative number of ports at a particular PGA normalised by the respective country total number of ports of a given country was then calculated as shown in Figure 6. The graph shows that Japan has its ports uniformly distributed on the hazard spectrum. China ports are spread from low hazard to moderate hazard. New Zealand ports are uniformly spread on the moderate to high seismic hazard bands. U.S. ports are split over the low and high hazard bands, since the West Coast is affected by high seismicity whereas the east coast by has a low seismic hazard (see Figure 1) Turkey has a large percentage of ports located in high hazard zones, while Italian ports are uniformly distributed over the hazard spectrum. The hazard characterising Greek ports lies between these latter two countries.

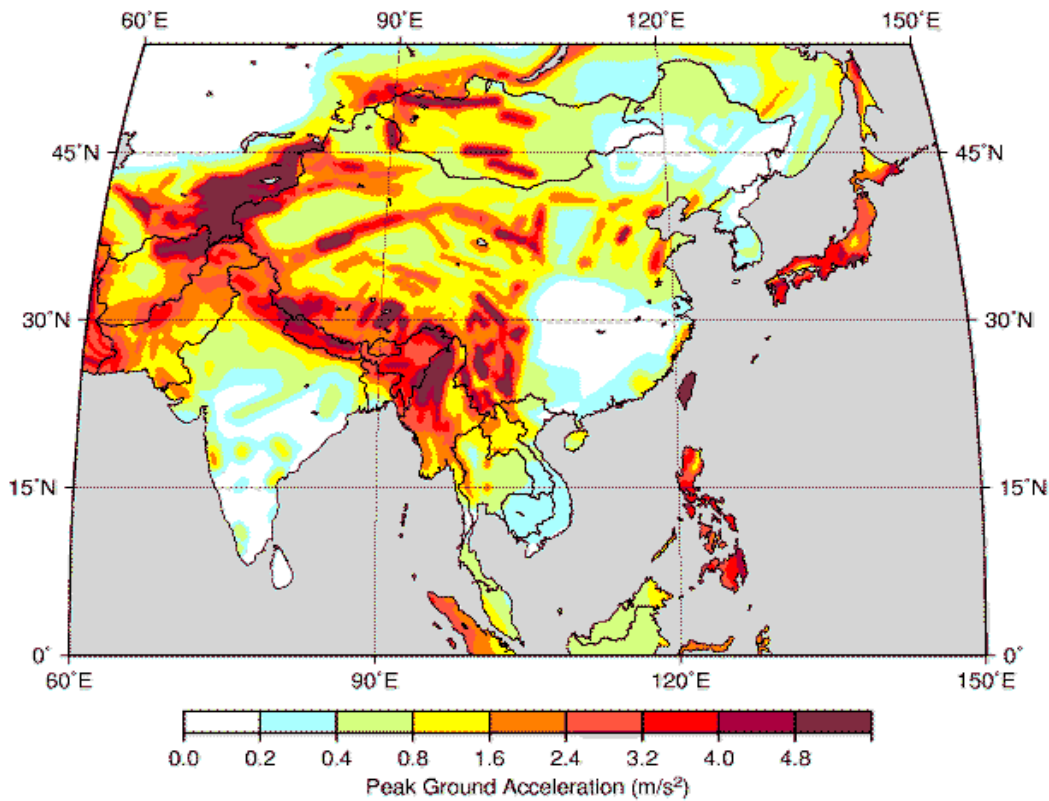
---

<sup>1</sup> European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), Via Ferrata 1, 27100 Pavia, Italy.

<sup>2</sup> European Centre for Training and Research in Earthquake Engineering (EUCENTRE), Via Ferrata 1, 27100 Pavia, Italy.



**Figure. 1 USA seismic hazard map (USGS,2006)**



**Figure 2 Asia seismic Hazard map (GSHAP, 1999)**

Table 2 shows the seismic hazard associated with the top 33 commercial container ports worldwide (ESPO, 2005). For these ports a non-linear regression between seismic hazard and container carriage volume was carried-out using Pearson correlation on SPSS V.12 software. As shown by Table 1, the correlation coefficient of the regression is low. The Pearson model indicates that the level of importance of a port would decrease with the increase of its seismic hazard (see Figure 7), however the reliability of this prediction for the examined data is low as indicated by the correlation coefficient.

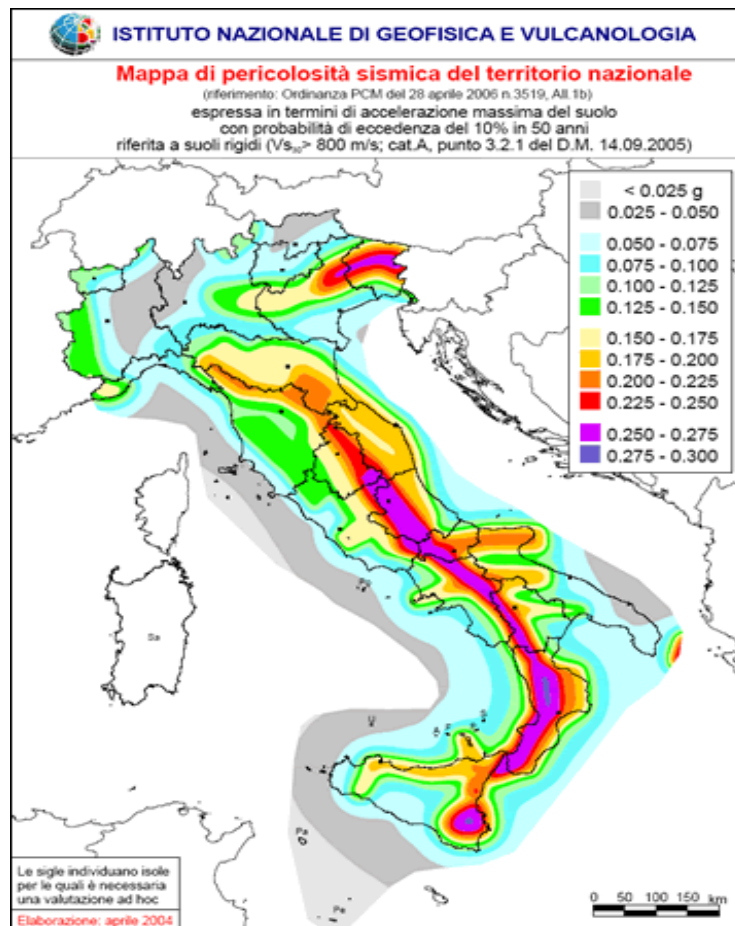


Figure 3. Italian Seismic Hazard Map (OPCM-3519, 2006)

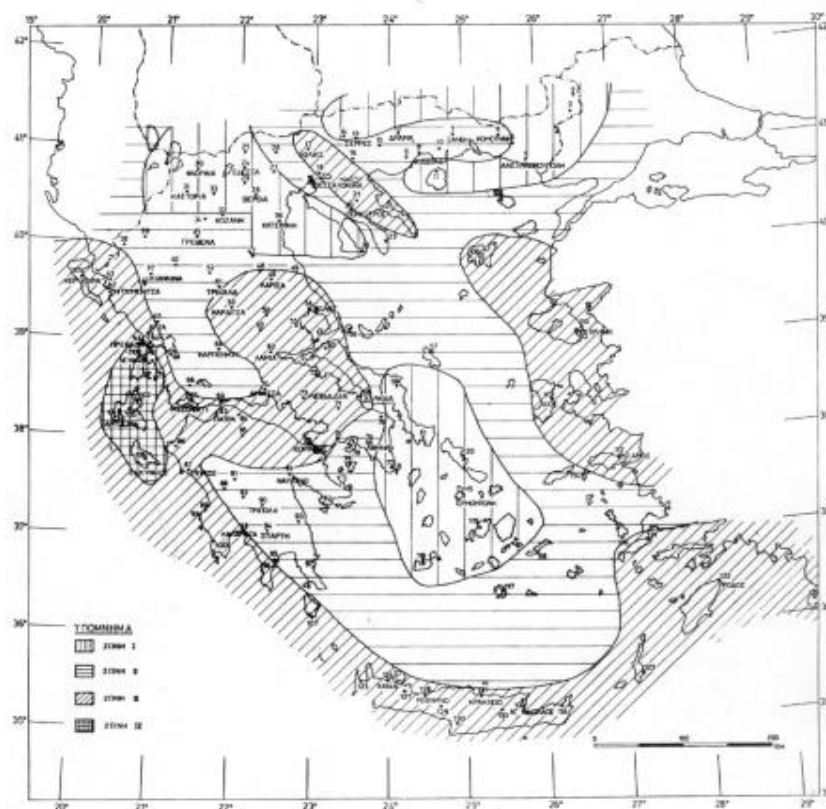


Figure 4. Greek Seismic Hazard Map (EAK, 2000)

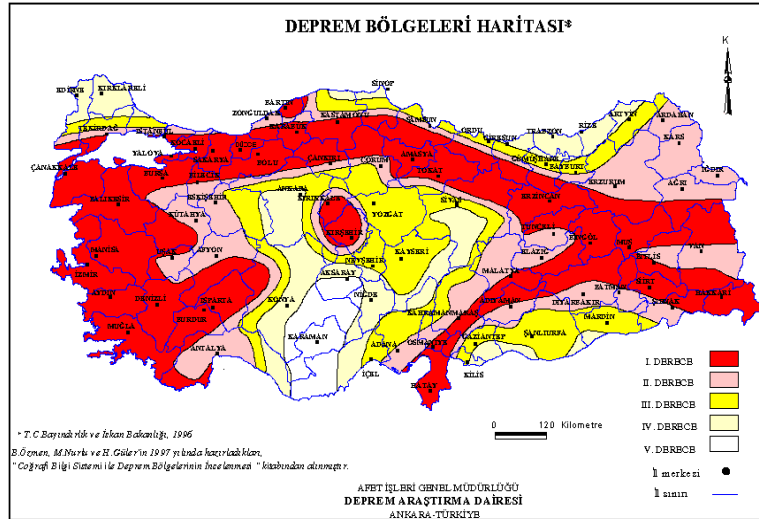


Figure5. Turkish seismic hazard map. (METU, 2006)

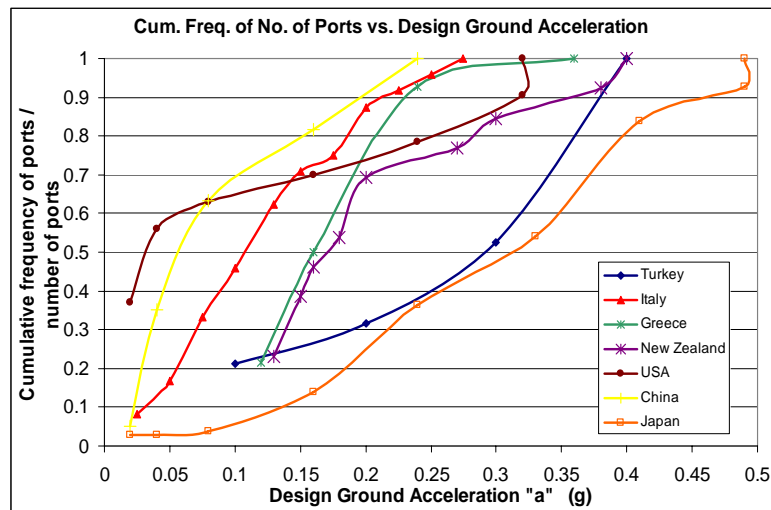


Figure. 6 Cumulative number of ports at a particular hazard value

Table 1. Pearson Correlation of Hazard and container carriage

Correlations		Seismic Hazard Hazard	Million TEU 2003
Seismic Hazard	P.C.	1.000	-0.235
	S	.	0.189
	N	33.000	33.000
Million TEU 2003	P.C.	-0.235	1.000
	S	0.189	.
	N	33.000	33.000
** Correlation is significant at the 0.01 level (2-tailed).			
P.C. = Pearson Correlation; S = 2 Tailed significance; N = Size of sample			

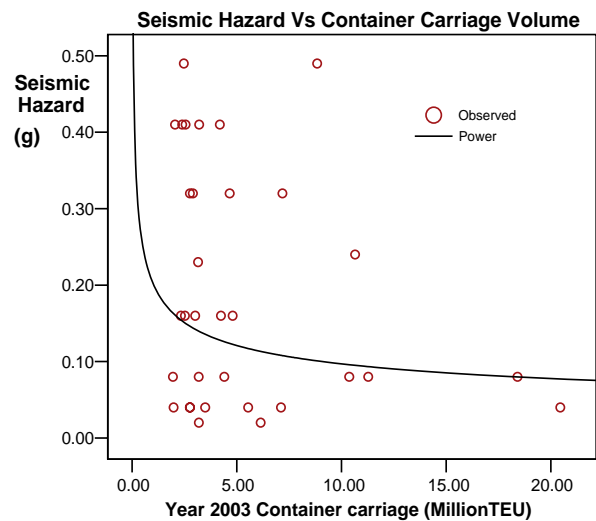


Figure. 7 Pearson Statistical model for hazard and container carriage.

**Table 2. Ranking of important world ports worldwide and their associated seismic hazard (data from ESPO, 2005).**

World Ranking	Port	Country	Hazard	2003	Annual Growth %
			(g)	Million TEU	
1	Hong Kong	China	0.04	20.45	7.8
2	Singapore	Singapore	0.08	18.41	3
3	Busan	Korea	0.08	10.37	16.5
4	Shanghai	China	0.08	11.28	45.2
5	Kaohsiung	Taiwan	0.49+	8.84	8.9
6	Shenzhen	China	0.24	10.65	67.4
7	Rotterdam	Netherlands	0.04	7.11	2.1
8	Los Angeles	USA	0.32+	7.18	20.2
9	Hamburg	Germany	0.02	6.14	12.9
10	Antwerpen	Belgium	0.04	5.54	11.6
11	Port Klang	Malaysia	0.16	4.8	37.3
12	Long Beach	USA	0.32+	4.66	2.6
13	Dubai Ports	Jebel Ali	0.41	4.19*	12.4
14	New York	USA	0.08	4.4	12.7
15	Quingdao	China	0.16	4.24	45.3
16	Tokyo	Japan	0.41	3.2	5.3
17	Bremen	Germany	0.02	3.19	16.2
18	Gioia Tauro	Italy	0.23	3.15	9.7
19	Manila	Philippines	0.41	2.55	8.2
20	Tanjong Priok	Indonesia	0.32	2.9*	13.2
21	Lam Chabang	Thailand	0.08	3.18	17.6
22	Tanjung Pelepas	Malaysia	0.04	3.49	/
23	Jakarta	Indonesia	0.32	2.76	/
24	Tianjin	China	0.16	3.01	34.1
25	Yokohama	Japan	0.49+	2.47	3.7
26	Algeciras	Spain	0.16	2.52	5.5
27	Guangzhou	China	0.04	2.76	39.1
28	Kobe	Japan	0.41	2.39	7
29	Nhava Sheva	India	0.08	1.95*	47.7
30	Nagoya	Japan	0.41	2.05	8.7
31	Ningbo	China	0.04	2.76	106.7
32	Xiamen	China	0.16	2.33	42.1
33	Le Havre	France	0.04	1.98	7.6

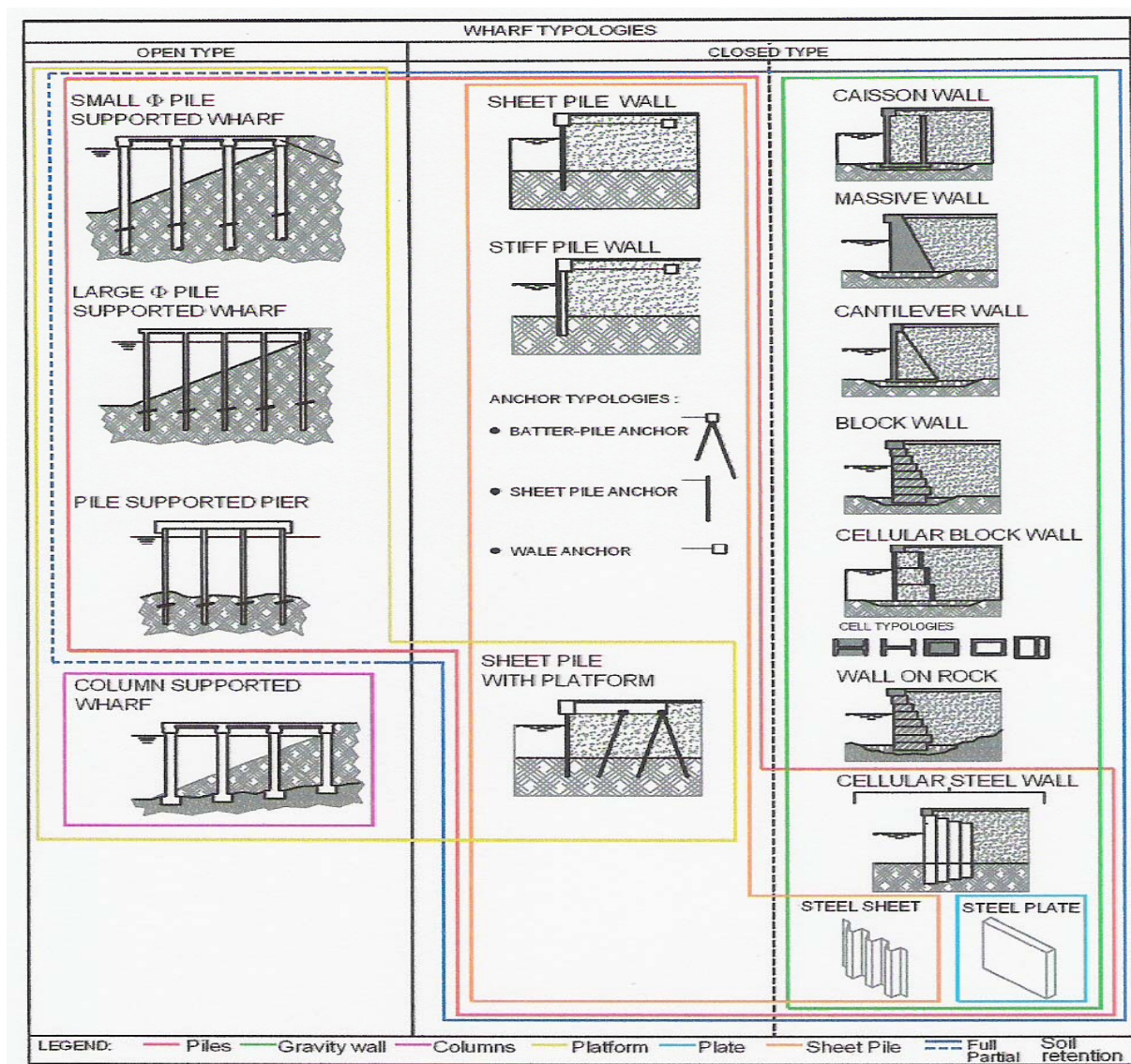
\* 2002 values are used.

## WHARF TYPOLOGIES AND ASSOCIATED EARTHQUAKE DAMAGE.

Wharf structures are traditionally distinguished between *open type* and *closed type* (see Figure 8). Open wharves are platforms supported by a series of vertical or battered piles, whereas closed wharves are composed by a deck supported by soil retained by a wall. Closed type wharves are further subdivided into sheet pile walls and cellular walls. An attempt was made to relate different typologies of wharf structures with various countries worldwide and their associated seismic hazard. The task is not straightforward due to the diversity of wharf typologies adopted even within the same country, and due to the numerous changes and upgrades that a particular seaport undergo throughout time.

However one can observe that closed type wharves is the preferred choice in Asian countries. Open-type wharves are more common in the American continent (OCDI, 2002, PIANC, 2001, Werner, 1999). In Europe both main typologies are commonly used. Failure modes and earthquake induced damages characterizing the identified typologies are short listed in Table 3. The data was retrieved from past case studies (Gazetas, 2005, OCDI, 2002, PIANC, 2001, Werner, 1999). Liquefaction appears to be the most common type of failure. Only column-supported wharves seem not susceptible to this failure mode. Based on the degree of structural redundancy and variety of failure possibility modes, wharf typologies were classified in Table 3. Closed-type wharves have a lower degree of structural redundancy since the lateral resisting structure is generally constituted in a single member element and its seismic performance rely strongly on the participation of other structural elements Structural redundancy was defined as the ability of system to resist deformation after one of it elements fails.





**Figure 8. Schematic classification of Ports and their constituting elements, (OCDI, 2002; PIANC, 2001; Werner, 1999; Scarpelli, 2005)**

Gravity walls usually fail due to seaward large displacements, settlement and tilt. Liquefaction of backfill cause the failure of sheet pile wall as large pressures are experienced. Often, such walls have also anchorages, and hence their efficiency depends on the efficiency of the soil-anchor interaction. Pile supported wharves have the tendency to fail due to large kinematically-induced bending moments in the piles as the dike liquefies. Failure at the pile deck joints is also common failure mode causing total collision. Hooks at the dowels should be bent inwards. Spiral reinforcement should be adopted. It is more a rule than an exception that battered piles fail during seismic loading (Example: Port of Oakland, USA during the Loma Prieta Earthquake). Table 3 shows that the sheet-pile structure typology is the most prone to failure. This is followed by cellular walls. Other gravity walls and open type wharves seem less vulnerable to fail when subjected to earthquake loading.

## DESIGN AND ANALYSIS

The seismic performance expected for a structure for a specified severity of design earthquake depends, on its class of importance as shown in Table 4. Usually in most worldwide codes or guidelines are considered. Level 1 earthquake represents an event whose intensity has either 40% or

**Table 3. Failure and damage identification for wharf typologies and classification of critical typologies, data from worldwide case studies found in Gazetas, 2005; OCDI, 2002; PIANC, 2001; Werner, 1999. A weighting score from “1” (small redundancy) to “4” (large redundancy) is given for each typology. A weighting of “1” is given for any possible presence of failure mode or damage.**

Wharf Type	Failures and damages																						Redundancy				Structure				
	Liquefaction backfill	Liquefaction foundation	Settlement of backfill	Settlement of structure	Pile Deck Hinge	In-ground hinge	Slope instability	Lateral Ground deformation	Seaward displacement	Opening of wall cracks	Deformation of pile	Tensile failure of batter piles	Deck cracks	Platform deformation/cracks	Pile cap failure	Buckling of steel piles	Tie-rod-to-anchor failure	Anchor pull-out	Kick-out of toe	Bulging of ground at walls	Sliding blocks	Wall collapse	Total Failure and Damage	4 - Very High	3 - High	2 - Low	1 - Very Low	Total Redundancy Value	Failure % Weighting	Redundancy % Weighting	Total % bWeighting
Small Diameter Pile Supported Wharf	1	1			1	1	1	1	1		1	1	1	1	1	1		1					14	1				1	0.08	0.02	0.19
Large Diameter Pile Supported Wharf	1	1			1	1	1	1	1		1	1	1	1	1	1		1					15		2			2	0.08	0.05	0.41
Pile supported Pier	1	1			1	1	1	1	1		1	1	1	1	1	1		1					15	1				1	0.08	0.02	0.20
Column Supported Wharf	1				1		1	1	1		1	1	1	1	1	1		1					11	1				1	0.06	0.02	0.15
Sheet Pile Wall	1	1	1			1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	18			3		3	0.10	0.07	0.74
Stiff Pile Wall	1	1	1			1	1	1	1		1	1	1	1	1	1	1	1	1	1	1	1	16			3		3	0.09	0.07	0.65
Sheet Pile with Platform	1	1	1			1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	19			3		3	0.11	0.07	0.78
Caisson Wall	1	1	1	1			1	1	1				1							1	1	1	10				4	4	0.06	0.10	0.55
Massive Wall	1	1	1	1			1	1	1				1							1	1	1	10				4	4	0.06	0.10	0.55
Cantilever Wall	1	1	1	1			1	1	1				1							1	1	1	10				4	4	0.06	0.10	0.55
Block Wall	1	1	1	1			1	1	1				1							1	1	1	11				4	4	0.06	0.10	0.60
Cellular Block Wall	1	1	1	1			1	1	1				1							1	1	1	11				4	4	0.06	0.10	0.60
Cellular Steel Wall	1	1	1	1			1	1	1				1			1				1	1	1	11				4	4	0.06	0.10	0.60
Wall On Rock	1		1				1	1	1				1							1	1	1	8			3		3	0.04	0.07	0.33
	Possible Failure or Damage Type																					Total	1/9			41	1.00	1.00			
	Possible Failure or Damage Type depending on presence of structural elements.																														
Legend:	Possible failure or damage not as critical as for other typologies																										Structural redundancy				

50% probability of being exceeded in 50 years (corresponding to a return period of 95 or 70 years respectively) and Level 2 earthquake representing an event whose intensity has either 5% or 10% probability of being exceeded in 50 years (corresponding to a 975 or 475 years return period respectively).

The methods of analysis are generally divided into three categories: simplified static methods, simplified dynamic and full dynamic approaches. Table 5 is a descriptive summary of the three methodologies that could be used when analyzing wharf structures. In Japanese seaport design guidelines (OCDI, 2002) a dual earthquake intensity approach (Level 1 and 2) is considered together with the adoption of displacement ductility factors in the damage criteria evaluation when using the simplified method of analysis. Strain limits are used in pushover type of analysis. In the U.S., based on the California Marine Oil Terminal Guidelines (PIANC, 2001), a dual earthquake intensity approach is also used together with damage criteria evaluation established on the basis of strain limits. Eurocode 8 (EN 1998-1:2005; EN 1998-5:2005) is also based on the adoption of a dual level earthquake severity approach, however no specific reference or prescriptions are made to seaport structures

In the last decade there have been important developments in the design approach of wharf structures, particularly pile-supported wharves. In the traditional Force-Based Design (FBD) approach, stress limits are obtained by reducing the ultimate elastic stress by a lumped factor. Performance criteria factors are only dealt with at the end of the process and hence a number of iterations are required. Moreover, since usually the supporting piles have different heights along the dike, inelastic deformation and thus ductility demand varies with the piles and this is poorly and inadequately represented by FBD (Priestley, 2003). In the recently developed Direct Displacement-Based Design (DDBD), the capacity of the structure (assessed through moment-curvature and pushover analysis),



and the allowable displacement-related damage are evaluated jointly using a coupled approach based on finding an equivalent single degree of freedom of the structure (Priestley,2003).

**Table 4. Performance levels and methods of analysis prescribed by PIANC.**

Structure Description	Design Earthquake	Performance Grade	Simplified Analysis	Simplified Dynamic Analysis	Dynamic Analysis
Critical structure; handles hazardous materials; economical and social importance; used by many people	Level 1 Level 2	Grade S			Serviceable Serviceable
Slightly less critical than "Grade S"; Structures not easily retrofitted.	Level 1 Level 2	Grade A			Serviceable Repairable
Ordinary structures, of ordinary importance	Level 1 Level 2	Grade B		Serviceable Near Collapse	
Easily restorable ordinary structures	Level 1 Level 2	Grade C	Repairable Collapse		

**Table 5. Description of the methods of Analyses associated to different wharf typologies (data from PIANC,2001; Werner, 1998;OCDI, 2002)**

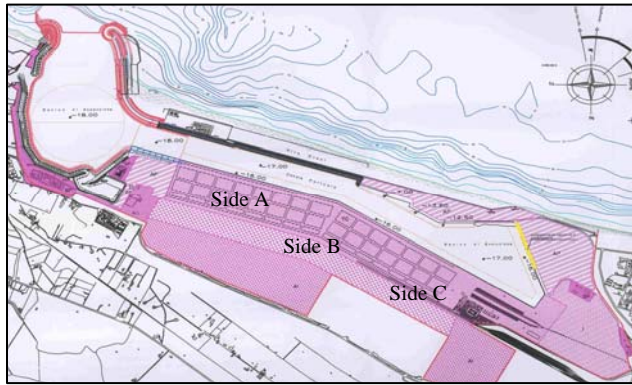
Wharf Type	Analysis and Design Methods								
	Simplified Analysis			Simplified Dynamic Analysis			Dynamic Analysis		
	Analysis	Input	Output	Analysis	Input	Output	Analysis	Input	Output
Small Diameter Pile Supported Wharf	Response Spectrum	Site response analysis (or code)/ stress levels or p-y curves/ Structure dimensions and material properties/Other parameters same as retaining wall	Displacement threshold of deck and dike	Response spectrum / Pushover	Same as for pseudo-static analysis + limit states such as yield and ultimate stresses, and corresponding material parameters.	Deck and dike: displacement; ductility; stress	Same as Gravity walls + pile and platform properties and dimensions. Yield stress for non-linear analysis. Yield stress and M-φ curves for linear analysis.		
Large Diameter Pile Supported Wharf									
Pile supported Pier									
Column Supported Wharf									
Sheet Pile Wall	Pseudo-static /empirical			Newmark Type	Same as gravity walls + as simplified analysis + yield and ultimate limit states.	Wall displacement	Same as Gravity and pile supported wharves + properties and dimensions of sheet piles, tie-rod and anchor.	Residual + Peak displacements; failure + response modes; stress; ductility.	
Stiff Pile Wall									
Sheet Pile with Platform									
Caisson Wall	Pseudo-static / empirical	Seismic coefficients/ site response analysis results (or code)/ geometric extent of soil for liquefaction/ geotechnical parameters/ liquefaction potential/ geometric and strength properties of anchor and tie rod for anchored sheet walls.	Displacement Threshold limit	Newmark Type + simplified charts	Time Histories/ v <sub>max</sub> ; a <sub>max</sub> ; using equations/ PGA at bedrock/ soil properties including SPT-N values, cohesion, angles of friction/ site response analysis.	Wall displacement; ductility; stress.	F.E.M. or F.D.M.: Linear or non linear 2D/3D analysis for (a) structural and, (b) geotechnical modelling (equivalent linear).	Time histories / wall section / bulk and shear moduli and cyclic parameters for non-linear analysis/ G/Go-g and D-g curves for linear analysis.	
Massive Wall									
Cantilever Wall									
Block Wall									
Cellular Block Wall	Pseudo-static			Newmark Type			Same as other gravity walls pile supported wharves + cell properties and dimensions.	Residual + Peak displacements; failure + response modes; stress; ductility.	
Cellular Steel Wall									
Wall On Rock									

## DESIGN EXAMPLE: ANCHORED T-SHAPED PILE WALL

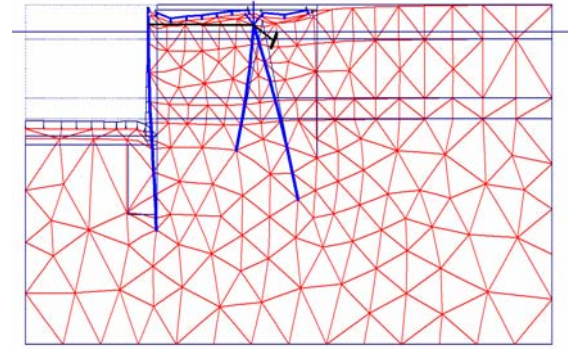
Table 1 shows that the most important seaports are located in Asia where the seismic hazard varies from moderate to high. In this region of the world, closed-type wharves appear to be the most common typology of wharf structure which Table 3 identified as one of the most vulnerable to damage under earthquake loading. Hence analysis and design of a closed type wharf was carried out using independently North American, Japanese and European standard practices. The results of this comparative study were then also compared to the information available from the dynamic analysis of the original design (Figure 10).

The T-shaped R.C. piles anchored wall (a closed type structure) constructed at Gioia Tauro Port in Southern Italy (Geographical co-ordinates: latitude 38° 26' North; longitude 15° 53' East) was considered for the analysis. The site is characterized by a design PGA for stiff ground of 0.223g which value is taken from seismic hazard data compiled by INGV. This corresponds to a ground





**Figure. 9 Plan view of Gioia Tauro seaport in Southern Italy (from Scarpelli, 2005)**



**Figure 10 Finite element mesh of Gioia Tauro cross-section of the wharf structure under study, (from Scarpelli, 2005)**

motion having 10% probability of exceedence in 50 years (i.e. 475 years return period). The R.C. T-shaped piles (see Figure.11a) have tie-rods separated 1.5m apart which connect to a battered pile system of anchors. “Side A” in Figure 9 was considered for the analyses. The retained soil is divided in 4 layers of sand some of which are blended with lime. Soil properties are shown in Table 6. Jet grouting was used in front of the wall for 5.5 m width. An equivalent unit weight of the jet-grouted soil was assumed to be  $14.36 \text{ kNm}^{-3}$  using Eq.[1] (Neville, 1997) given that the compression strength is 19MPa and an operative strain elastic modulus  $E' = 10500 \text{ MPa}$  (Scarpelli, 2005). “R” is the density of the material and “f’c” the compression strength.

**Table 6. Soil parameters adopted for the design of Gioia Tauro wharf (“Side A”) in Southern Italy (data from Scarpelli,2005)**

Layer	$\gamma_{\text{sat-front}}$ kNm-3	$\gamma_{\text{sat-behind}}$ kNm-3	$\gamma_{\text{b-front}}$ kNm-3	$\gamma_{\text{b-behind}}$ kNm-3	$\gamma_{\text{wet}}^*$ kNm-3	c'	$\phi^k$ o	E' kN/m2
U1a	/	17.00	/	7.00	14.16	0	30	30000
U1a	/	17.00	/	7.00	/	0	30	30000
U2	/	19.00	/	9.00	/	0	30	50000
U3	/	18.00	/	8.00	/	0	36	50000
U4	24.46	19.00	14.36	9.00	/	0	36	80000

$$E' = 43p^{1.5}(f'_c)0.5 \times 10^{-6} \quad (1)$$

The pile wall was analysed using the simplified, pseudo-static North American (PIANC, 2001 and Ebeding, 1992), Japanese (OCDI, 2002) and European approaches (EN 1998-5: 2005; EN 1997-1: 2005; EN 1998-1:2005; OPCM-3274:2005). The pseudo-static methods provided by these regulations are based on using modified Mononobe-Okabe relations to compute the seismic earth-pressure. From the site investigations and analysis available, the retained soil resulted not to be susceptible to liquefaction (Scarpelli, 2005).

The same friction angles considered in the F.E.M. were also assumed in the three pseudo-static analysis since the purpose was to compare the results of different methods of analyses. In factual pseudo-static calculations, the adopted value of the friction angle should be lower than that measured in the laboratory or that employed in F.E.M. analysis by factors given in respective codes.

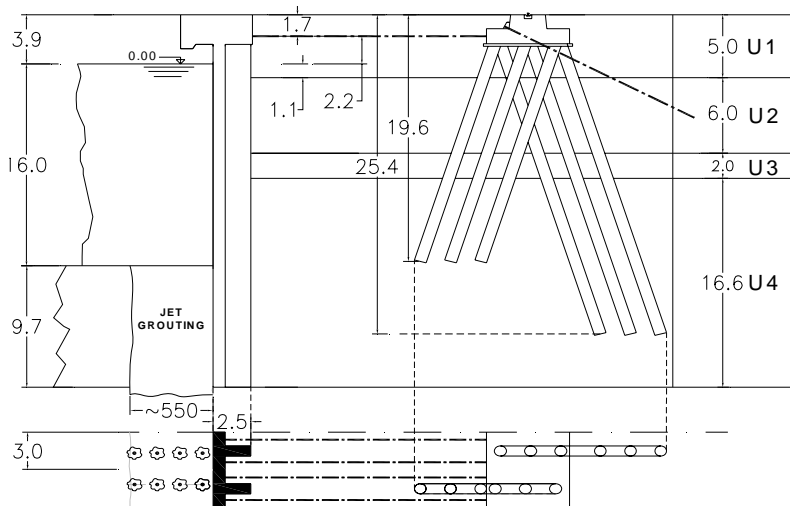
A surcharge  $q=10$  kPa and a vertical crane action of 667 kN were applied. A coefficient of topographic amplification  $S_T = 1.1$  to account for sloping effects and a coefficient  $S = 1.25$  of soil amplification (which would correspond to Soil “Type C” according to EC 8 classification) were considered in the definition of the design seismic action. A factor of importance  $\gamma_I=1.1$  was used to magnify the design acceleration. Concerning the effect of vertical acceleration, the Japanese norms (OCDI, 2002) recommend to neglect them by setting  $k_v=0$ . On the contrary Ebeding et. al. (1992) and EC-8 suggest the structure to be analysed using the most conservative choice among  $1-k_v$ ,  $k_v=0$  and  $1+k_v$ .

Equivalent seismic coefficients were computed for the North-American and Japanese approaches. Using Japanese practice (OCDI, 2002),  $k'_h$  varies with depth between 0.26-0.35 while using the PIANC (2002) guide a value of 0.41 was obtained. For the PIANC (2002) manual a second value of  $k'_h$  and  $k'_v$  was found for the passive earth pressure with saturated values of bulk density. The value adopted for the soil angle of friction was divided by a safety factor of 1.2 as suggested by Ebeding et. al. (1992). For simplification, the water level in front and behind the wall was assumed to be the same even though OCDI (2002) recommends to take into account the possible difference. Thus the hydrostatic pressure distribution at each side of the wall was assumed to cancel out, whereas the hydrodynamic pressure gave a resultant force of  $P_{wd-N.amer}= 357$  kN/m,  $P_{wd-Japan}= 380$  kN/m and  $P_{wd-Euro}= 460$  kN/m. The variation in these values is mainly due to the different values assumed for the horizontal seismic coefficient  $k'_h$ . For the North American approach (PIANC, 2001) the total active earth pressure ( $P_{ae}$ ) was found to be 3196 kN/m, yielding a moment balance about tie rod of 25749 kNm and a tie-rod resultant force of 679 kN/m. The location along the wall where the shear force is zero yielded a maximum bending moment of 7677 kNm. With the Japanese approach (OCDI, 2002) the total passive pressure resulted to be 3687 kN/m while the active pressure 2261 kN/m. Using a simple Euler-Bernoulli beam on Winkler foundation model a maximum bending moment of 3785 kNm was found at the location where the shear force is zero. The tension in the tie rod resulted to be 575 kN.

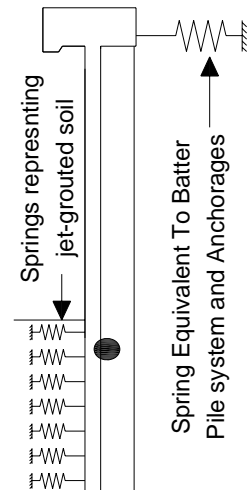
Eurocode 8 (EN 1998-5, 2005) suggests that, when the height of the wall is larger than 10 m (Fardis et. al., 2004), the values of the horizontal seismic coefficient  $k_h$  should be determined from a local site response analysis i.e. considering the variation of PGA with depth. In this example a linear variation of PGA was assumed with depth. For the active pressure a value of  $P_{ae} = 5919$  kN/m was obtained using  $(1-k_v)$ , whereas values of  $P_{ae} = 1805$  kN/m and  $P_{ae} = 231$  kN/m were obtained respectively for  $(1+k_v)$  and  $k_v = 0$ . A finite element model was built using Ruaomoko2D (Carr, 2005), where the wall was modelled as an Euler-Bernoulli beam on a Winkler foundation with elastic-perfectly plastic springs (Fardis et. al, 2004, EN 1998-5: 2005) as shown in Figure 11b. The springs were modelled (Finn, 2005) using non linear stiffness obtained by bilinear approximations of P-Y curves (see Figure 13) of the soil along the passive side of the embedded depth of the wall (Martin, 2005). The tie-rod anchor was modelled by a spring with stiffness value of 38,500 kN/m, equivalent to that of the battered piles-soil system. By varying this stiffness from infinity to zero (see Figure 15), the maximum bending moment outside the embedded depth was observed to increase as the stiffness increases and displacement decreases.

As shown by Figure 12, the loads on the system were applied as distributed loads. The beam was also analysed using North-American and Japanese prescriptions and corresponding seismic loads. Figures 16 and 17 show the bending moment and displacement distributions along the depth of the wall that were obtained.

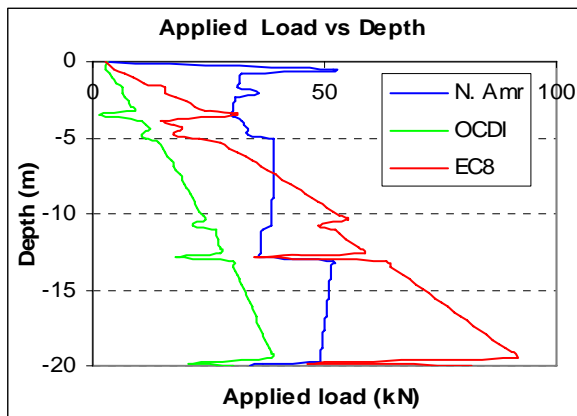
The finite element design of the soil-pile wall carried out by Scarpelli (2005) using PLAXIS, yielded a maximum bending moment  $M_{max} = 8920$  kNm and a maximum displacement  $D_{max} = 8.8$  cm. In the simplified approaches we obtained  $M_{max}= 4230$  kNm, 6100 kNm, 8760 kNm and  $D_{max} = 4.6$  cm, 4.1 cm, 11.1 cm using Japanese, North American and European practices respectively.



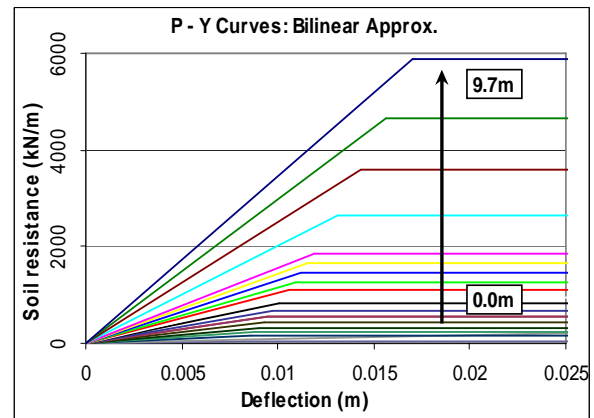
**Figure 11a. Sectional (top) and plan (bottom) view of the earth-retaining wall at Gioia Tauro seaport in Southern Italy.**



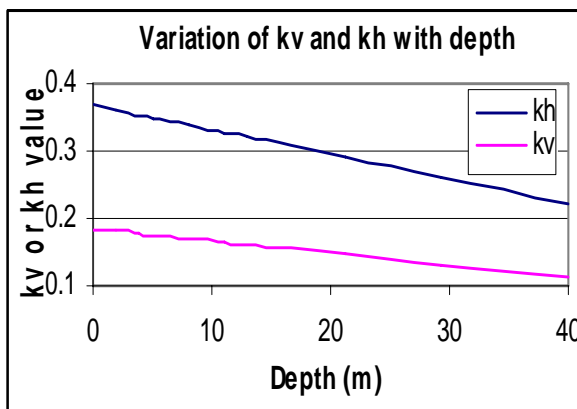
**Figure.11b Equivalent beam on Winkler foundation model of the pile-wall and soil system.**



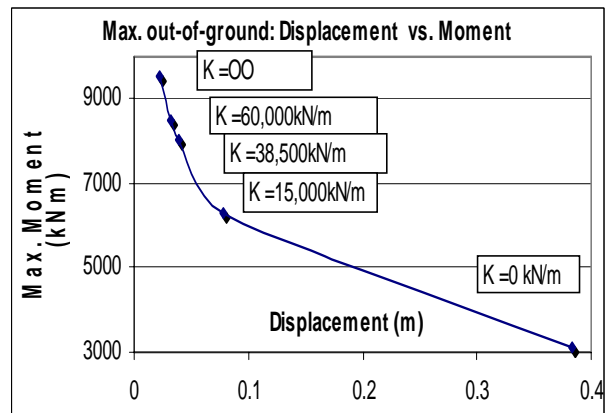
**Figure 12 Spring Model of the soil-pile wall: Applied Loads**



**Figure 13 Spring model of the spring pile wall: P-Y curves of the embedded depth**



**Figure 14 EC8: variation of  $k_h$  and  $k_v$  with depth**

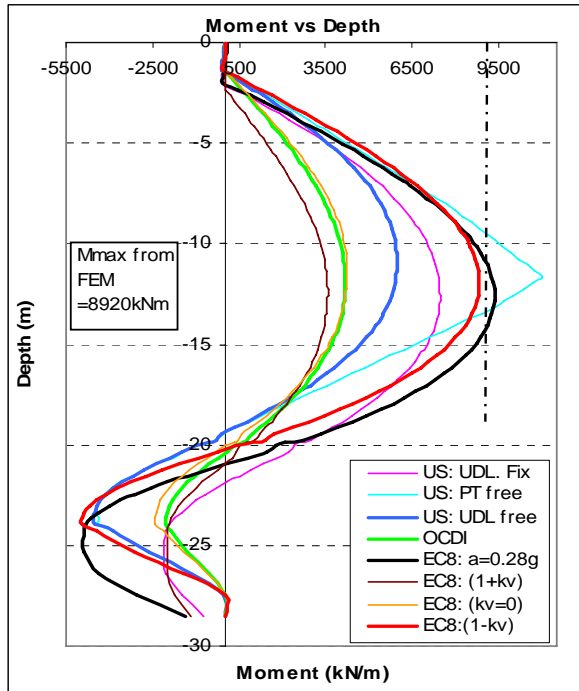


**Figure 15 Spring model of the soil-pile wall: Varying stiffness of anchor**

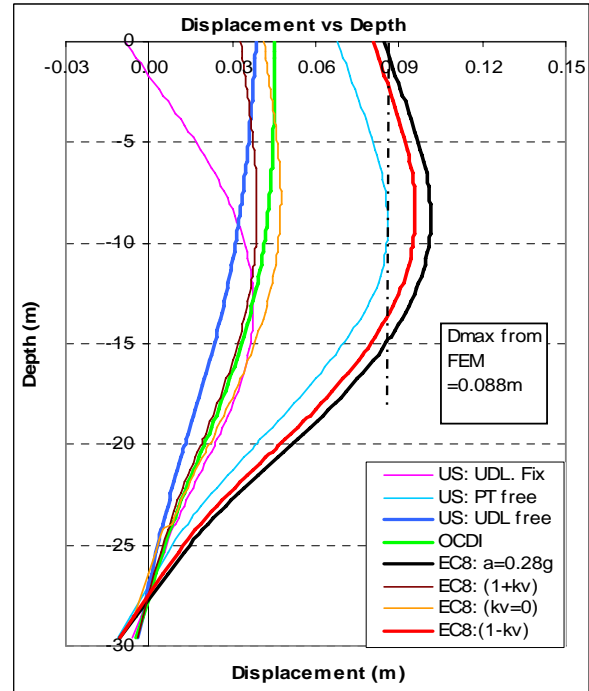


## CONCLUSIONS

This paper attempted to briefly illustrate a few data about the typologies of wharf structures adopted in worldwide seaports, their expected level of damage and seismic vulnerability inferred from observations of past case studies. An attempt was made to correlate the level of importance of a given seaport and its associated seismic hazard information retrieved from standard macro-zonation studies available for a few countries including US, Japan, China and in Europe; Greece, Turkey and Italy. A design example of a closed-type wharf, which appear to be as one of the most vulnerable to earthquake-related damage, has been carried out and the results compared. Standard recommendations from Japanese, North-American and European practices were used. The main outcome of the study can be summarised as follows:



**Figure 16 Spring model of the soil-pile wall: Bending moment vs. depth**



**Figure 17 Spring model of the soil-pile wall: displacement vs. depth**

- If the seismic hazard at a seaport is not high compared to other seaports, its seismic risk is high if its economical importance is large. Although a worldwide correlation between the level of importance of a seaport and seismic hazard is low, port importance tend to decrease as the seismic hazard increases.
- Closed type wharves, particularly sheet pile wharves are the most prone to suffer damage amongst different wharf typologies. Such structures are more common in Asia where seismicity in coastal region varies mostly from moderate to high.
- Figures 16 and 17 show that the pseudo-static method adopted by Eurocode 8 (EC8) describes well the maximum bending moment obtained from advanced finite element analysis. The displacements are slightly over predicted. The seismic loads computed according to Japanese practice (OCDI, 2002) were too low and the bending moment and displacement patterns compare well with the corresponding charts from EC8 for the case  $kv=0$ . For the Japanese practice,  $kh$  increases with depth, however being a function of  $PGA$   $kh$  is expected to decrease with depth. Finally concerning the North American practice (PIANC, 2001), it is observed that the predicted maximum bending moment is higher than that obtained by following Japanese practice, probably because the factor  $(1-kv)$  as taken into account. Its loading pattern however do not reach the values predicted by EC8.

## ACKNOWLEDGEMENTS

The authors would like to express their appreciation to Dr. Renata Gentile, from the University of Genoa, Italy for providing the data relative to Gioia Tauro seaport, in Southern Italy. This study was made possible thanks to a research grant provided by the Department of Civil Protection of Italian Government (*Progetto Esecutivo 2005-2008, punto f dell'articolo 3, progetto n. 5*) whose support and constant encouragement is greatly acknowledged.

## REFERENCES

- Carr, A.J., (2005). "Ruaumoko 2D: Dynamic Analysis of 2-D In-elastic Structures", *University of Canterbury, Christchurch, New Zealand*.
- Carvalho, E., Elnashai, A., Faccioli, E., Fardis, M., Pinto, P. & Plumier, A. (2004). "Designers' Guide to EN 1998-1 and EN 1998-5 Eurocode 8: Design of Structures for Earthquake Resistance." *Thomas Telford, U.K.*
- EAK (2000), "Greek Antiseismic Code." *OASP/SPME*.
- Ebeding, R.M. & Morison, E.E., (1992), "The Seismic Design of Waterfront Retaining Structures." *US Army Corps of Engineers, Technical Report ITL-92-11, Washington D.C.*
- EN 1998-1:2005. "Eurocode 8- Design of Structures for Earthquake Resistance- Part 1: General rules, seismic actions and rules for buildings." *European Communities Commission..*
- EN 1998-5:2005. "Eurocode 8- Design of Structures for Earthquake Resistance-Part 5: Foundations, Retaining structures and Geotechnical aspects." *European Communities Commission..*
- EN 1997-1: 2005. "Eurocode 7 – Geotechnical design – Part 1: general Rules." *Commission of the European Communities.*
- ESPO, (2005). "Factual Report on the European Port Sector." *European Seaports Organisation, Brussels.*
- Finn W.D.L., (2005). "A Study of Piles during Earthquakes." *Issues of Design and Analysis, Bulletin of Earthquake Engineering, 3: 141-234.*
- Gazetas, G., Anastasopoulos, I., Dakoulas, P., (2005). "Failure of Harbour Quay wall in the Lefkada 2003 Earthquake."
- GSHAP, (1999). "Global Seismic Hazard Assessment Program (GSHAP) in Continental Asia", <http://www.seismo.ethz.ch/GSHAP/>
- INGV, 2006 "Italian Seismic Hazard Data", <http://www.zonesismiche.mi.ingv.it>
- Martin G., (2005). "POLA Code: Presentation on Geotechnical Aspects." *Container Wharf Seismic Code Workshop, Port of Los Angeles, California.*
- Middle-East Technical University, (2006) "Turkish Seismic Hazard Map", [www.deprem.gov.tr](http://www.deprem.gov.tr) ; [www.metu.edu.tr](http://www.metu.edu.tr)
- Neville, A.M., (1997). "Properties of Concrete." *Longman Series, London.*
- NTZS1170.5:2004. "Structural Design Actions, Earthquake Actions", *Council of Standards, New Zealand.*
- OPCM-3519, (2006). "Criteri Generali per l'Individuazione Delle Zone Sismiche e per la Formazione e l'aggiornamento degli elenchi delle medesime zone", *Italian Gov.Gazette.*
- OPCM-3274, (2005). "Norme Tecniche per il Progetto, La Valutazione e l'Adeguamento Sismico Degli Edifici", *Italian Government Gazette.*
- OCDI (2002). "Technical Standards and Commentaries for Port And Harbor Facilities in Japan." *Overseas Coastal Area Development Institute, Japan.*
- PIANC (2001). "Seismic Design Guidelines for Port Structures." *International Navigation Association, A.A. Balkema Publishers, Tokyo.*
- Priestley, M.J.N., (2003). "Myths and Fallacies in Earthquake Engineering, Revisited" *IUSS Press, Pavia, Italy.*
- PMHW, (1998), "World Ports Harbours and Marinas", <http://www.portfocus.com>, (2006)
- Scarpelli, G. (2005). "Lavori Di Escavo Del Canale e Del Bacino Di Espansione del Porto." *Design document of Gioia Tauro Port, Gioia Tauro Port Authority.*

SPSS V.12, (2003), "SPSS: Predictive Analytical Software", *SPSS inc., Illinois*.  
USGS, (2006). "National Seismic hazard Maps", <http://earthquake.usgs.gov/research/>  
Werner, S.T. (1998), "Seismic Guidelines for Ports", *Technical Council on Lifeline Earthquake Engineering, Monograph No.12, ASCE*