A STUDY FOR THE DEVELOPMENT OF SEISMIC DESIGN SPECIFICATIONS FOR COASTAL STRUCTURES

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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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ABSTRACT

A STUDY FOR THE DEVELOPMENT OF SEISMIC DESIGN SPECIFICATIONS FOR COASTAL STRUCTURES

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An evolving design philosophy for port structures in many seismically active regions reflects the observations that:

-The deformations in ground and foundation soils and the corresponding structural deformation and stress states are key design parameters.

-Conventional limit equilibrium-based methods are not well suited to evaluating these parameters.

-Some residual deformation may be acceptable.

Performance-based design is an emerging methodology whose goal is to overcome the limitations present in conventional seismic design. Conventional building code seismic design is based on providing capacity to resist a design seismic force, but it does not provide information on the performance of structure when the limit of the force-balance is exceeded. If we demand that limit equilibrium not be exceeded for the relatively high intensity ground motions associated with a rare seismic event, the construction cost will most likely be too high. If force-balance design is based on amore frequent seismic event, then it is difficult to estimate the seismic performance of the structure when subjected to ground motions that are greater than those used in design.

In this thesis a case study will be carried out on a typical port structure to show the performance evolution aspects and its comparison with damage criteria and performance grade in performance-based methodology.

Keywords: Port Structures, Design Methodology

ÖZ

KIYI YAPILARI İÇİN BİR SİSMİK TASARIM ŞARTNAMESİ GELİŞTİRME ÇALIŞMASI

Gözpınar, Erdem
Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Sismik olarak aktif bölgelerde, gelişmekte olan dizayn felsefesi göstermektedir ki:

-Zemindeki ve temel toprağındaki deformasyonlar ve bunlara karşılık oluşan yapısal deformasyon ve stres durumları anahtar dizayn parametrelerini oluşturmaktadır.

-Geleneksel limit denge esaslı dizayn metotları bu parametrelerin takdir edilmesine uygun değildir.

-Bazı kalıcı deformasyonlar kabul edilebilir.

Gelişmekte olan performans-esaslı metot, geleneksel metottaki kısıtlamaların önüne geçmektedir. Geleneksel sismik dizayn belli bir sismik yüklemeye dayanacak kapasite sunmaktadır, fakat yük dengesi aşıldığında yapının performansı hakkında bilgi sunamamaktadır.Eğer nadiren olan çok kuvvetli yer hareketi için limit dengenin aşılmamasını istersek, büyük ihtimalle yapı maliyeti çok fazla olacaktır. Eğer daha sık olan yer hareketi için limit dengenin aşılmaması istenirse bu kez dizaynda kullanılan yer hareketinden daha büyük bir yer hareketinde yapının performansını belirlemek güç olacaktır.

Bu tez çalışmasında performans takdiri ve bunun zarar kriterleri ve performans notuyla karşılaştırması için tipik bir liman yapısı üzerinde durum çalışması yapılacaktır.

Anahtar Kelimeler: Kıyı Yapıları, Tasarım Yöntemi

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CHAPTER I

INTRODUCTION

The occurrence of a large earthquake near a major city may be a rare event, but its societal and economic impact can be so devastating that it is a matter of national interest. Although seismicity varies regionally, earthquake disasters have repeatedly occurred not only in the seismically active regions in the world but also in areas within low seismicity regions. Mitigating the outcome of earthquake disaster is a matter of worldwide interest.

In order to mitigate hazards and losses due to earthquakes, seismic design methodologies have been developed and implemented in design practice in many regions since the early twentieth century, often in the form of codes and standards. Most of these methodologies are based on a force-balance approach, in which structures are designed to resist a prescribed level of seismic force specified as a fraction of gravity. These methodologies have contributed to the acceptable seismic performance of port structures, particularly when the earthquake motions are more or less within the prescribed design level.

Although the damaging effects of earthquakes have been known for centuries, it is only since the mid-twentieth century that seismic provisions for port structures have been adopted in design practice. In 1997, the International Navigation Association formed a working group that focuses international attention on devastating of earthquakes on port facilities. This group also published a document named as 'Seismic Design Guidelines for Port Structures'. The provisions reflect the diverse nature of port facilities. Although constructed in marine environment, the port facilities are associated with extensive waterfront development, and provide multiple land-sea transport connections. The port must accommodate small to very large vessels, as well as special facilities for handling potentially hazardous materials and critical emergency facilities that must be operational immediately after a devastating earthquake.

The primary goal of this study is the development of a consistent set of seismic design guideline in the steps of this working group to mitigate hazards and losses due to earthquakes. The diverse characteristics of port structures led the study to adopt an evolutionary design strategy based on seismic response and performance requirements. Performance-based methodology was studied as a new approach. It is important that the deformations in ground and foundation soils and the corresponding structural deformation and stress states are key design parameters. Conventional building code seismic design is based on providing capacity to resist a design seismic force, but it does not provide information on the performance of structure when the limit of the force-balance is exceeded. Therefore, it is not applicable to evaluate key design parameters.

The seismic design guidelines for port structures in this study address the limitations inherent in conventional design, and establish the framework for a new design approach. In particular, the guidelines intended to be:

- performance-based, allowing a certain degree of damage depending on the specific functions and response characteristics of a port structure and probability of earthquake occurrence in the region;
- user-friendly, offering design engineers a choice of analysis methods, which range from simple to sophisticated, for evaluating the seismic performance of structures;
- general enough to be useful throughout the world, where the required functions of port structures, economic and social environment, and seismic activities may differ from region to region.

CHAPTER II

EARTHQUAKES AND PORT STRUCTURES

Seismic waves are generated along a crustal fault and they propagate through upper crustal rock, traveling to the surface of the bedrock at a site of interest as illustrated in Fig 2.1. The ground motions then propagate through the local soil deposits, reaching to the ground surface and impacting structures. If the intensity of shaking is significant and depending on the soil conditions, liquefaction of near-surface soils and associated ground failures may occur and may affect the port structures. Tsunamis may be generated if an offshore fault motion involves vertical tectonic displacement of the sea bed. The seismic effects on port structures could be very significantly depending on the collective impact of these phenomena.

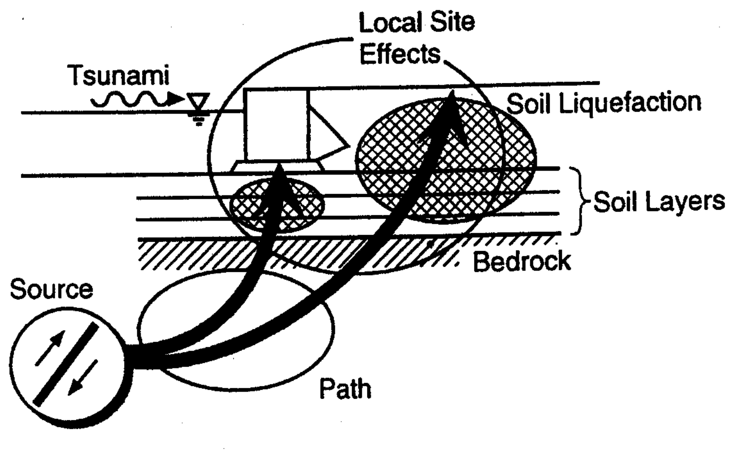


Figure 2.1 Schematic figure of propagation of seismic waves (PIANC, 2001)

2.1 Earthquake Motion

2.1.1 Bedrock Motion

Design at a particular site characterized through seismic hazard analysis where the bedrock motions are used for seismic analysis. If a specific earthquake scenario is assumed in the seismic hazard analysis, the bedrock motion is defined deterministically based on the earthquake source parameters and wave propagation effects along the source-to- site path. In most of the cases, the bedrock motion is defined probabilistically through the seismic hazard analysis, taking into account uncertainties in frequency of occurrence and location of earthquakes. In the engineering design practice, one of the key parameters is the intensity of bedrock motion defined in terms of peak ground acceleration (PGA), or in some cases peak ground velocity (PGV). This parameter is used either by it self or to scale relevant ground motion characteristics, including response spectra and time histories. In the probabilistic seismic hazard analysis, the level of bedrock motion is defined as a function of a return period, or a probability of exceedance over a prescribed exposure time. For a prescribed return period, the bedrock motion is often specified in codes or standards for a region.

2.1.2 Local Site Effects

The soil deposits at a particular site with their dynamic response characteristics may significantly modify the bedrock ground motion by changing the amplitude, frequency content and duration , and it has been termed ‘ local site effects’. Local site effects depend on the material properties of the subsoil and stratigraphy, as well as the intensity and frequency characteristics of the bed rock motion. As strong ground motion propagates upwards, towards the ground surface, the ground motions are tended to be amplified because of the reduction in the strength and stiffness of soil deposits. In engineering practice, local site effects are evaluated either by using prescribed site amplification factors based on statistical analysis of existing data or a site specific response analysis. The site amplification factors are often specified in codes and standards, and are used to scale the bedrock PGA or PGV to obtain the corresponding values at the ground surface, or are used to scale bedrock response spectra to define the ground surface response spectra.

2.2 Liquefaction

Soil liquefaction can be defined as significant reduction in shear strength and stiffness due to increase in pore pressure.

As saturated soil deposits are shaked rapidly back and forth, (e.g. earthquake) the water pressure in the pores of the soil starts to rise. In loose saturated cohesionless soils, (e.g. sand) the pore water pressure can rise rapidly and may reach such a level that the particles briefly float apart and the strengthened stiffness of the soil is temporarily lost altogether. This is a condition called soil liquefaction, and it is shown diagrammatically in Fig 2.2. The strength of soil is the result of friction and interlocking between the soil particles. At any depth in the ground, before the earthquake, the weight of the soil and other loads above is carried in part by friction+interlocking forces between the soil particles and in part by the pore water. When loose soil is shaken, it tries to densify or compact. The presence of the water, which has to drain away to allow the compaction, prevents this from happening immediately. As a consequence, more and more of the weight above is transferred to the pore water and the forces between the soil particles reduce. Ultimately, the pore water pressures may reach such a level that they cause water spouts to break trough the overlying and the whole weight of the overlying material is transferred to the pore water. In this condition, the liquefied soil behaves as a viscous fluid, and large ground movements can occur. This liquefaction condition will continue until the high pore water pressures can drain again. And the contact between the soil particles is restored. Some layers in the ground will densify as a result of this process, and the ground settlements will be observed. Other layers will remain in a very loose condition, and will be prone to liquefy again in the future earthquakes. In order to adequately asses the liquefaction susceptibility of a soil both the cyclic resistance of the material and the seismic actions on the soil by design-level earthquake motions must be determined.

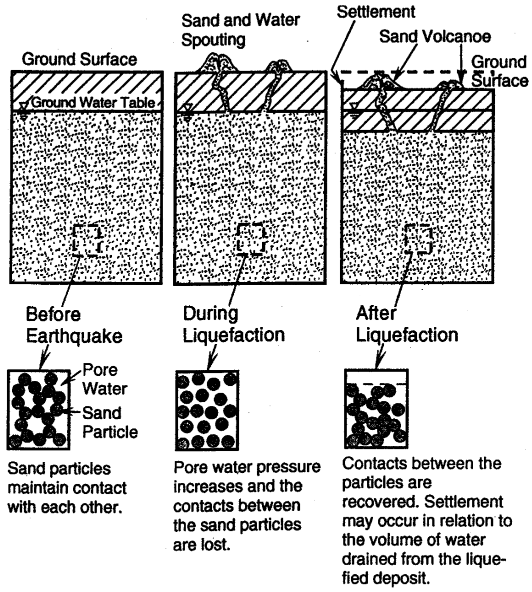


Figure 2.2 Mechanism of liquefaction (PIANC, 2001)

2.3 Tsunamis

Tsunamis are long period sea waves that are generated by seafloor movements. They are associated with seismic fault ruptures, but occasionally with submarine landslides. Although wave amplitudes may be small in the open ocean, the wave height increases as the tsunamis approach shallower depths, occasionally reaching tens of meters at the coast line. The wave height of tsunamis is also amplified toward the end of V-shaped bays. When produced by earthquakes, the predominant wave period of tsunamis ranges from five to ten minutes. Tsunamis can easily propagate long distances, such as across the Pacific Ocean. In this case, the pre dominant wave period typically ranges from forty minutes to two hours. Arrival time ranges from within five minutes for locally generated tsunamis and to one day for distant tsunamis traveling across the Pacific Ocean. Destructive forces by tsunamis can be devastating.

2.4 Port Structures

From an engineering point of view, port structures are soil-structure systems that consist of various combinations of structural and foundation types. Typical port structures are shown in Fig 2.3.

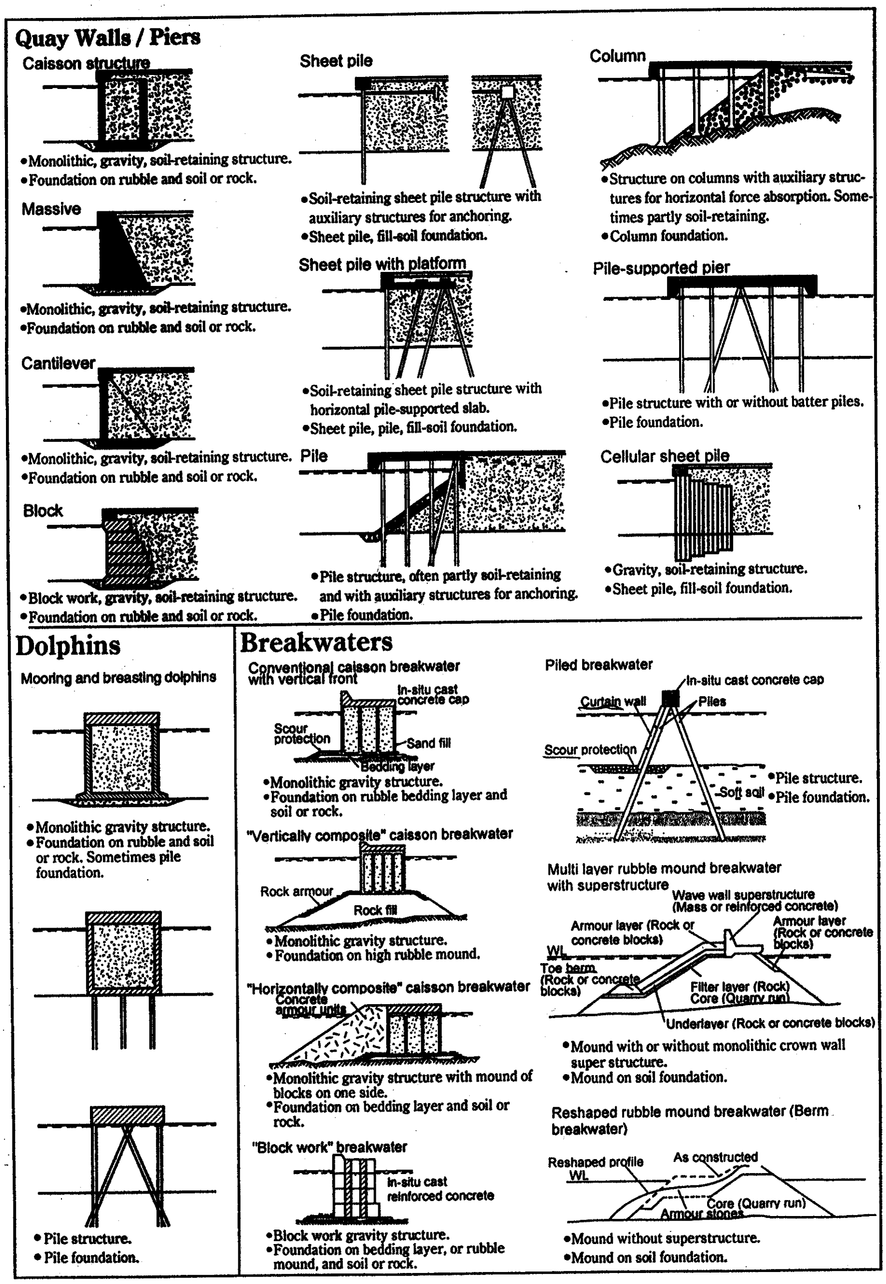


Figure 2.3 Typical port structures (PIANC, 2001)

2.5 Summary of the Paper Written about the Effects of East Marmara Earthquake (EME) (Yüksel et. al, 2003)

The effects of EME and its associated tsunami on marine structures and coastal areas are well investigated by Yuksel et. al, 2003 with consideration of the tectonic setting and geotechnical properties. Common damage modes with gravity type, piled and sheet piled type of marine structures were summarized. In order to understand the damage to the coastal structures, which ranges from small displacements to complete collapses, field observations of ruptures, subsidence or coastal landslides and the tectonic setting under the sea have been discussed.

2.5.1 The Effects of EME on Marine Structures

Block type quays: Serious damage on block type concrete quay walls with a lateral displacement towards the sea and settlement on the backfill behind the quay walls was observed especially at Derince Port. Observations also revealed that the block type quay wall moved seaward without any vertical displacement. Diver reports demonstrated that the blocks slid on their rock foundation without relative vertical movement between blocks. At some quays mid-span deflections and relative corner movements were observed. Also liquefaction was observed on the backfill behind the quay wall. The settlement of backfill caused the tilting of a crane on rails. One of the cranes was overturned while others were derailed due to the rocking response to the earthquake shaking. There was one crane that was fixed to the foundation that did not suffer apparent damage. The most liquefaction occur at a location where near a river basin mainly caused by the complexity of sedimentation of the soil. However, the major problem is sandy backfill material behind the quay walls dredged from a river mount by the sea probably a kind of delta sediment.

Piled and sheet piled quays: Concrete breakage at pile caps, the settlement of the fill area behind the quay probably causing damage to the tie rods, some pile damages, concrete crack along the deck, the settlement of the fill area behind the apron between concrete conveyor belt foundations the tilt of a crane and shearing all of its bolts at the foundation connections of a conveyor belt structure are the damage examples.

Jetties: Damages of jetties is usually related with the damaged piles. Squared concrete piles which have one of the probable disadvantages for the driving into the dense sand and gravel for necessary skin friction and end bearing was used. Steel piles behave better for such kind of soils if the bearing layers contain gravel. The cracks and seaward displacement was observed. One of the jetties had two different structures, one was made of concrete piles but the other was steel pile. The concrete section of the jetty was tilted and displaced away from the steel section. Cracks were observed around conjunctions between the piles and beams where diagonal piles head touched with each other. If there was a distance between pile caps, serious problems had not been observed. The steel piles were wrinkled.

Breakwaters: The breakwater suffered due to settlements. One of the reasons for deformation is liquefaction. Slope stability failure but liquefaction near the toes of slopes may act together and cause the failure of structure. Generally breakwaters did not show serious damage except insufficient foundations.

Tsunami effects: Tsunami effects are mainly on run up ranges and changing water levels. The tsunami behavior affected all small boat harbors by receding the water inside the harbor creating strong currents swept several small boats out to sea. Earthquake damage & failure magnitudes are defined to show the overall devastating effects of EME on coastal structures in a tabulated form (Yalçıner et.al 2001, 2002).

Geotechnical investigations can be summarized as follows:

- Liquefaction and slope failures are important at sandy and silt contained natural ground.
- Big care should be taken for the backfill material.
- Soft clay foundations are problem. Steel piles are more suitable for deep foundations.

Marine structures and their failure classification are summarized in Table 2.1 Fault brake in Izmit Bay is in Fig 2.4. Some damage examples are shown in Figures 2.5, 2.6, 2.7, 2.8, 2.9 and 2.10.

Table 2.1 Marine structures and their failure classification (Yüksel et. al, 2003)

No	Marine Structures	Structure Type	Failure Type	Service Type	Distance to Epicenter (km)
1	TUZLA DOCK PORT	Block type quay, monolithic breakwater	C	▲	48
2	ESKIHISAR FERRY PIER	Block type quay, Ship Ramp	C	□	35
3	ESKIHISAR FISHERY PORT	Rubble mound breakwater	D	□	35
4	ROTA MARINE PIER	Concrete piled pier	D	□	8.5
5	TUPRAS JETTIES AND PİERS	Concrete and steel piled piers	B	▲	5.5
6	DERINCE PORT	Block type and piled quay	B	▲	3
7	PETROL OFISI PIERS	Concrete and steel piled piers	A	▲	4.5
8	SHELL DERINCE PIER	Steel piled pier	A	●	5
9	KORUMA TARIM PIER	Concrete piled pier	A	●	5.5
10	TRANSTURK PIER	Steel piled pier	B	▲	6
11	IZMIT MARINA	Concrete piled pier	C	□	9.5
12	UM MARINE PORT	Steel piled pier	A	●	7.5
13	GOLCUK PORT AND DOCKS	Steel piled pier	B	▲	0.0
14	KARAMURSEL EREGLI FISHERY PORT	Rubble mound breakwater	C	▲	13.5
15	TOPCULAR FERRY PIER	Concrete sheet piled and steel piled piers	D	□	32
16	AKSA PIER AND DOLPHINS	Steel piled pier	B	▲	43
17	YALOVA MARINA	Rubble mound breakwater	D	□	48
18	KORUKOY PIER	Concrete piled pier	D	□	56
19	CINARCIK FISHERY PORT	Rubble mound breakwater	B	▲	65
20	KOCADERE PIER	Concrete piled pier	D	□	71
21	ESENKOY FISHERY PORT	Rubble mound breakwater	C	□	78

Service Type

▲	partial service
□	fully serviceable
●	no service

Failure Type

Level A	Significant failure
Level B	Intermediate failure
Level C	Minor failure

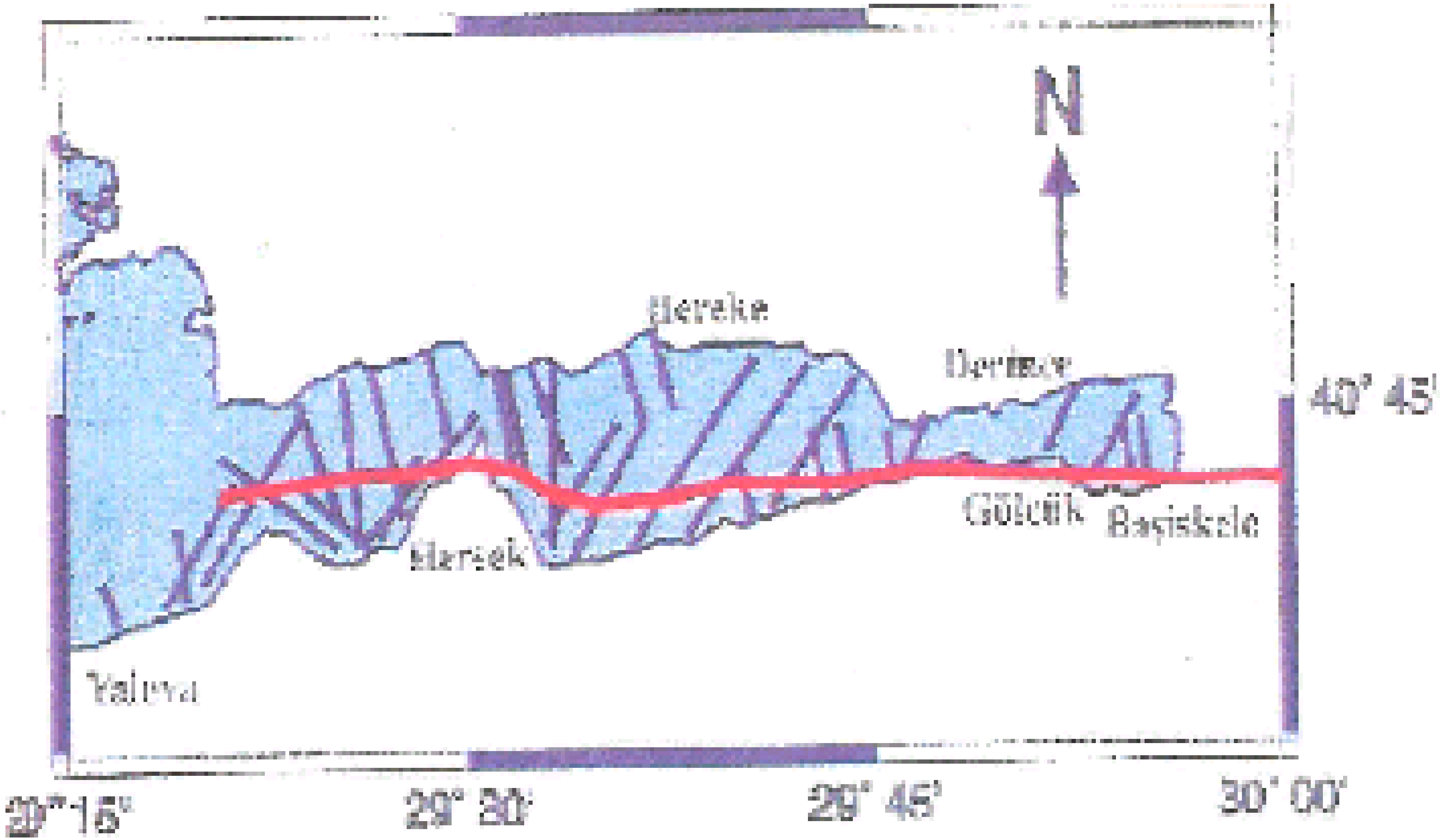


Figure 2.4 Fault break in İzmit Bay (Aug 17, 1999 Earthquake) (Yüksel et. al 2000)

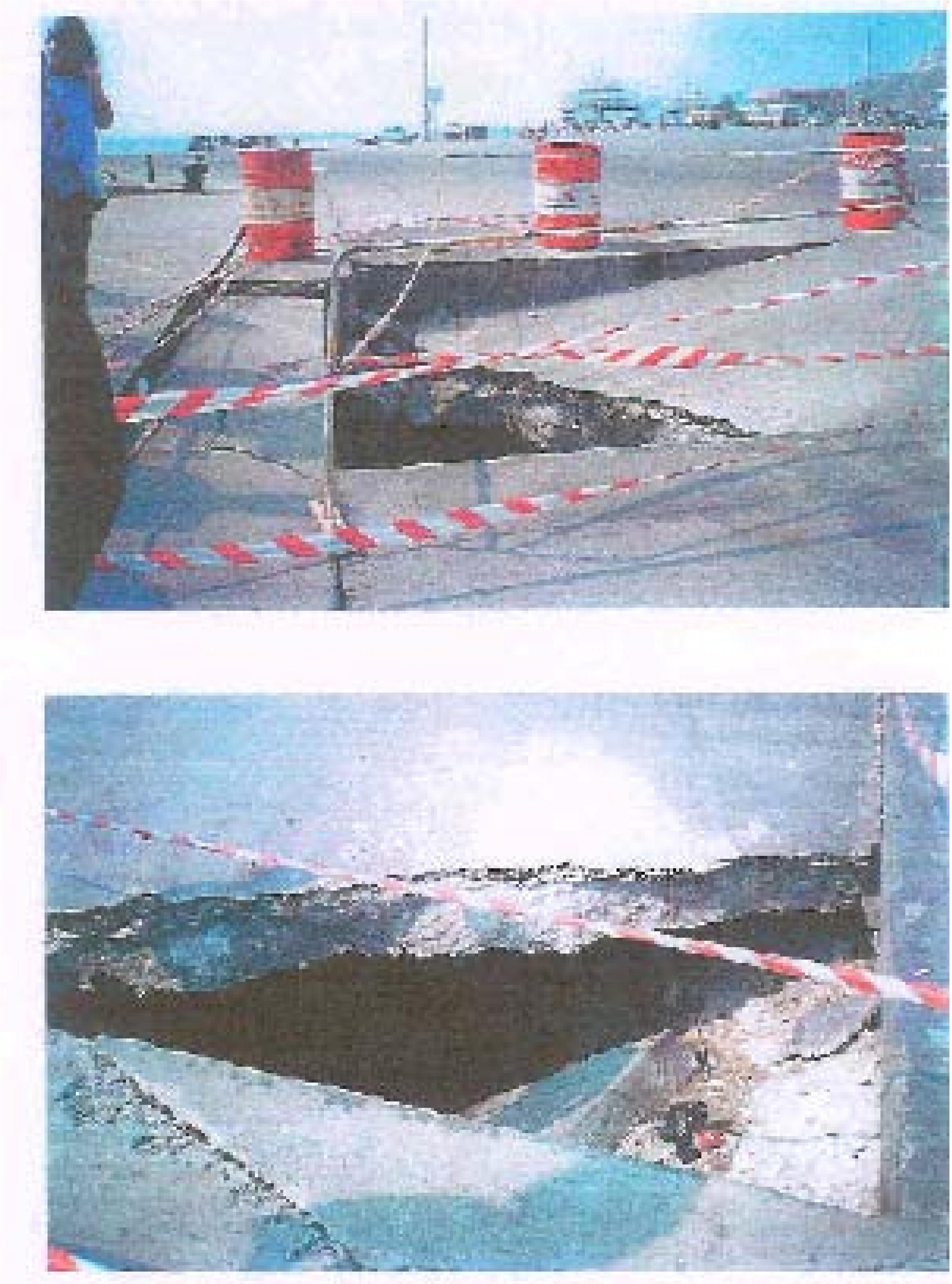


Figure 2.5 Eskihisar Ferry Pier (Yüksel et. al 2000)

Figure 2.6 Tuzla Dock Port (Yüksel et. al 2000)

Figure 2.7 Eskihisar Fishery Port (Yüksel et. al 2000)

Figure 2.8 Petrol Ofisi Piers (Yüksel et. al 2000)

Figure 2.9 Derince Port (Yüksel et. al 2000)

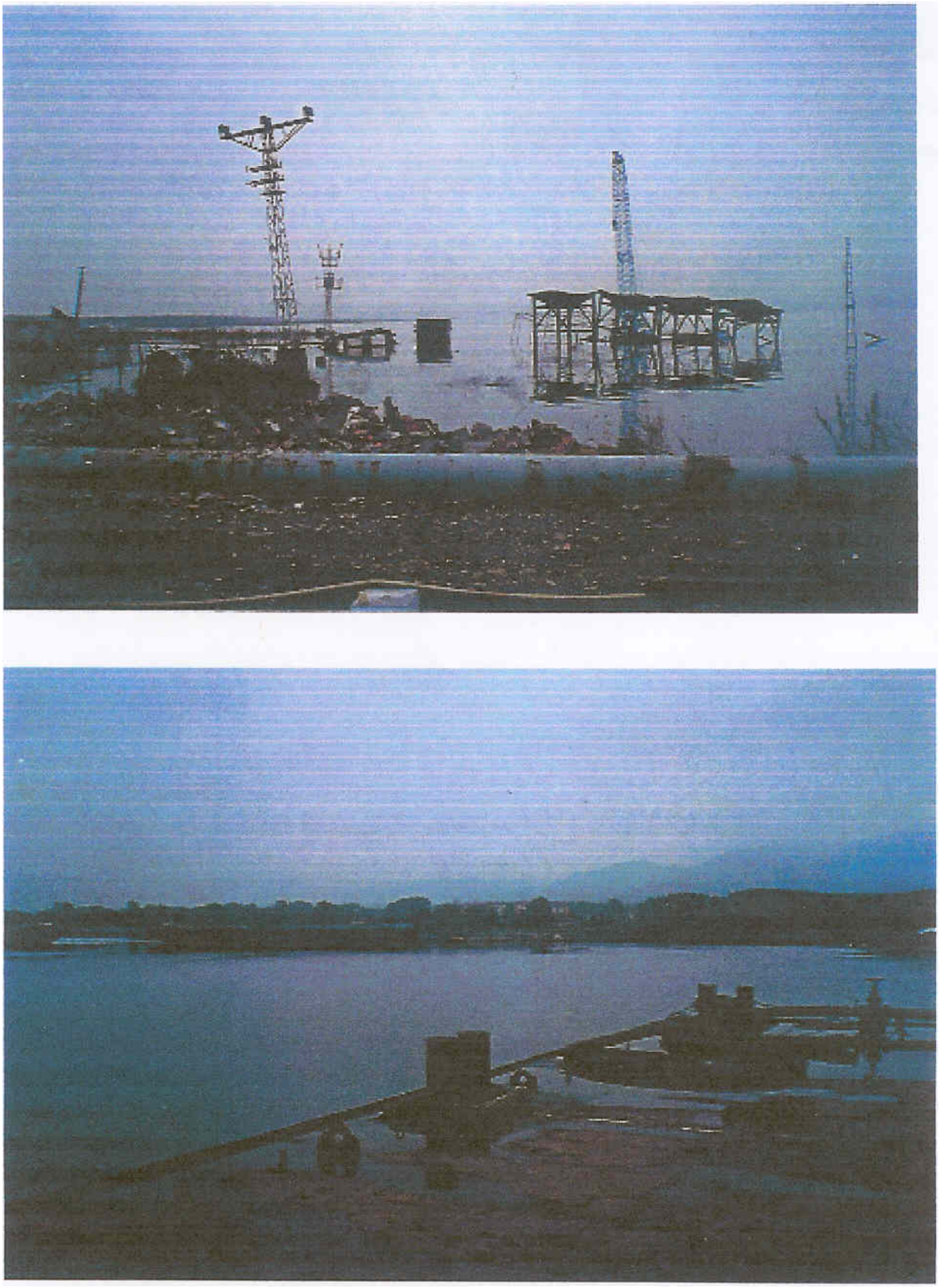


Figure 2.10 U.M Marine Port (Yüksel et. al 2000)

CHAPTER III

DESIGN PHILOSOPHY

In many seismically active regions, the evolving design philosophy and the basic concepts are given below:

- The key design parameters for the performance-based methodology which provides engineers with new design tools are the deformations in ground and foundation soils.
- The corresponding structural deformation and stress states are key design parameters. Deformation/failure modes of gravity quay wall, sheet pile quay wall and pile supported wharf are in Fig 3.1, 3.2 and 3.3 respectively.
- Conventional limit equilibrium-based methods are not well suited to evaluating these parameters.
- Some residual deformation may be acceptable.

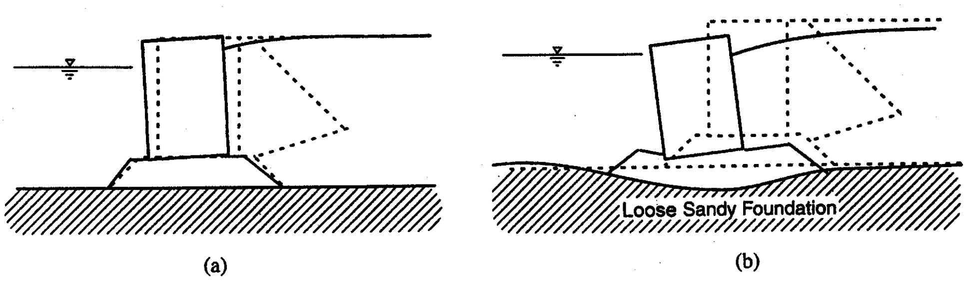


Figure 3.1 Deformation/failure modes of gravity quaywall (PIANC, 2001)

(a) on firm foundation

(b) on loose sandy foundations

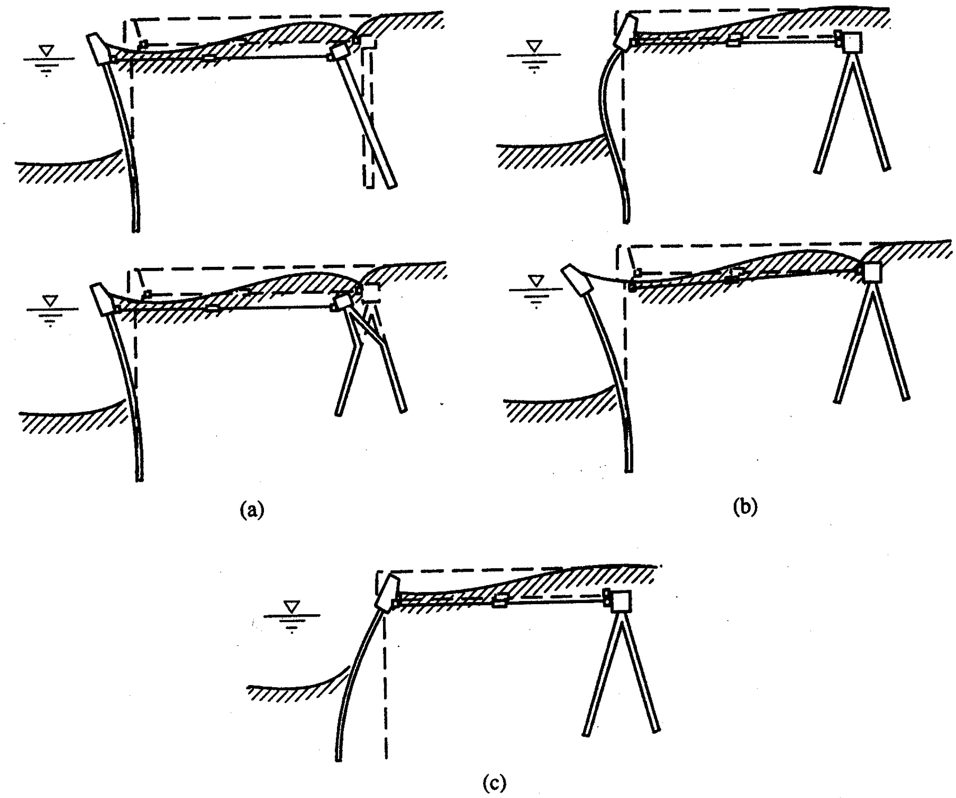


Figure 3.2 Deformation/failure modes of sheet pile quaywall (PIANC, 2001)

(a) Deformation/failure at anchor

(b) Failure at sheet pile wall/tie rod

(c) Failure at embedment

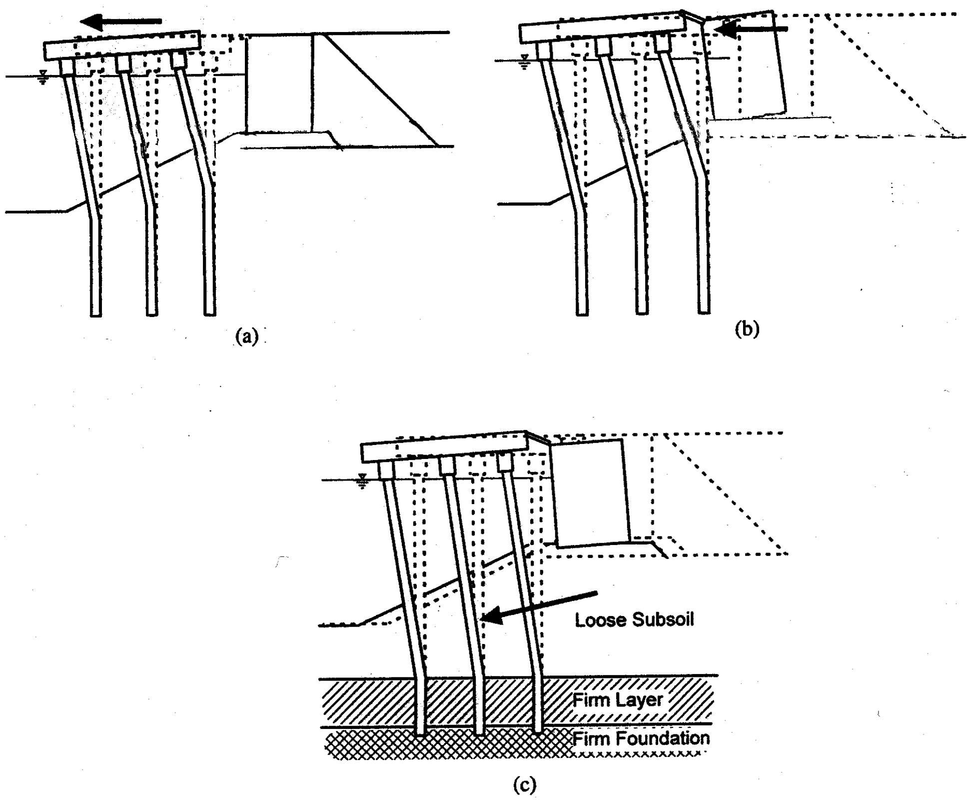


Figure 3.3 Deformation/failure modes of pile-supported wharf (PIANC, 2001)

(a) Deformation due to inertia force at deck

(b) Deformation due to horizontal force from retaining wall

(c) Deformation due to lateral displacement of loose subsoil

3.1 Performance-based Design Methodology

The limitations present in conventional seismic design are overcomed by performance-based design which is an emerging methodology. Conventional building code seismic design is based on providing capacity to resist a design seismic force, but it does not provide information on the performance of structure when the limit of the force-balance is exceeded. If the limit equilibrium is not exceeded for relatively high intensity ground motions associated with a rare seismic event, the construction cost will most likely be too high on the other hand, if force-balance design is based on a more frequent seismic event, then it is difficult to estimate the seismic performance of the structure when subjected to ground motions that are greater than those used in design.

In performance-based design appropriate levels of design earthquake motions must be defined together with the corresponding acceptable levels of structural damage which must be clearly identified. Two levels of earthquake motions are typically used as design reference motions, defined as follows: (PIANC, 2001)

- Level 1 (L1): the level of earthquake motions that are likely to occur during the life-span of the structure;
- Level 2 (L2): the level of earthquake motions associated with infrequent rare events, that are typically involving very strong ground shaking.

The acceptable level of damage specified according to the specific needs of the user/owners of the facilities is defined on the basis of the acceptable level of structural and operational damage given in Table 3.1.The structural damage category in Table 3.1 is directly related to the amount of work needed to restore the full functional capacity of the structure and is often referred to as direct loss due to earthquakes. The operational damage category is related to the length of time and cost associated with the restoration of full or partial serviceability. Economic losses associated with the loss of serviceability are often referred to as indirect losses. In addition to the fundamental functions of servicing sea transport, the functions of port structures may include protection of human life and property, functioning as an emergency base for transportation, and as protection from spilling hazardous materials. If applicable, the effects on these issues should be considered in defining the acceptable level of damage in addition to those shown in Table 3.1.

Table 3.1 Acceptable level of damage in performance-based design.* (PIANC, 2001)

LEVEL OF DAMAGE	STRUCTURAL	OPERATIONAL
Degree 1: Serviceable	Minor or no damage	Little or no loss of serviceability
Degree 2: Repairable	Controlled damage**	Short-term loss of serviceability***
Degree 3: Near collapse	Extensive damage in near collapse	Long-term or complete loss of serviceability
Degree 4: Collapse****	Complete loss of structure	Complete loss of serviceability

* Considerations: Protection of human life and property, functions as an emergency base for transportation, and protection from spilling hazardous materials, if applicable, should be considered in defining the damage criteria in addition to those shown in this table

** With limited inelastic response and/or residual deformation.

*** Structure out of service for short to moderate time for repairs.

**** Without significant effects on surroundings.

Once the design earthquake levels and acceptable damage levels have been properly defined, the required performance of a structure may be specified by the appropriate performance grade S, A, B, C defined in Table 3.2. In performance-based design, a structure is designed to meet these performance grades.

Table 3.2 Performance grades S, A, B and C. (PIANC, 2001)

Performance grade	Design earth quake	
	Level 1 (L1)	Level 2 (L2)
Grade S	Degree 1:Serviceable	Degree 1:Serviceable
Grade A	Degree 1:Serviceable	Degree 2:Repairable
Grade B	Degree 1:Serviceable	Degree 3:Near collapse
Grade C	Degree 2:Repairable	Degree 4:Collapse

The principal steps taken in performance-based design are shown in the following chart in Fig 3.4.

1) Select a performance grade of S, A, B, C: This step is typically done by referring to Tables 3.1 - 3.2 and selecting the damage level consistent with the needs of the users/owners. Another procedure for choosing a performance grade is to base the grade on the importance of the structure. Degrees of importance are defined in most seismic codes and standards. This procedure is presented in Table 3.3. If applicable, other than those of S, A, B; or C may be introduced to meet specific needs of the users/owners.
2) Define damage criteria: Specify the level of acceptable damage in engineering parameters such as displacements, limit stress states, or ductility factors.
3) Evaluate seismic performance of a structure: Evaluation is typically done by comparing the response parameters from a seismic analysis of the structure with the damage criteria. If the results of the analysis do not meet the damage criteria, the proposed design or existing structure should be modified. Soil improvement including remediation measures against liquefaction may be necessary at this stage.

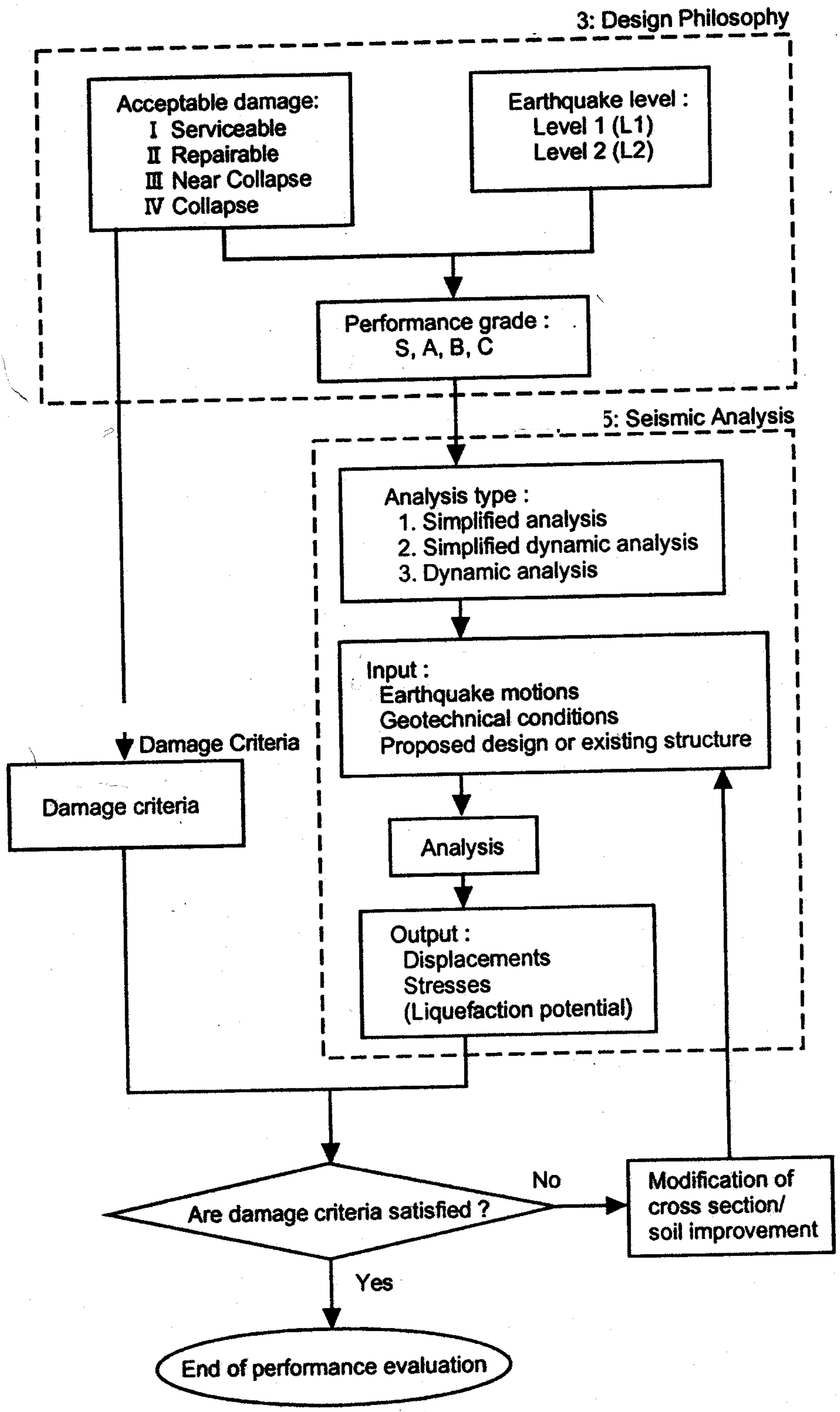


Figure 3.4 Flowchart for seismic performance evaluation (PIANC, 2001)

Table 3.3 Performance grade based on the importance category of port structures (PIANC, 2001)

Performance grade	Definition based on seismic effects on structures
Grade S	1-Critical structures with potential for extensive loss of human life and property upon seismic damage 2-Key structures that are required to be service able for recovery from earthquake disaster 3-Critical structures that handle hazardous materials 4- Critical structures that, if disrupted, devastate economic and social activities in the earthquake damage area
Grade A	Primary structures having less serious effects for 1 through 4 than Grade S structures or 5-structures that, if damaged, are difficult to restore
Grade B	Ordinary structures other than those of Grades S,A and C
Grade C	Small easily restorable structures

3.2 Reference Levels of Earthquake Motions

Level 1 earthquake motion (L1) is likely to occur during the life time of structure and typically defined as motion with a probability of exceedance of 50% during the life-span of a structure. Level 2 earthquake (L2) is infrequent rare event and typically defined as a motion with a probability of exceedance of 10% during the life span. In defining these motions, near field motion from a rare event on an active seismic fault should also be considered if the fault is located nearby. If the life span of a port structure is 50 years, the return periods for L1 and L2 are recommended as 75 and 475 years, respectively.

In regions of low seismicity, L1 may be relatively small and of minor engineering significance. In this case, only L2 is used along with an appropriately specified damage criteria. Here, it is assumed that performance for L2 will implicitly ensure required performance under the anticipated L1 motion. It may be noted that this single level approach is some what similar to conventional design practice; it differs only in that a structure is designed in accordance with a designated acceptable level of damage.

The dual level approach using both L1 and L2 attempts to: 1) ensure a specified level of safety and serviceability for L1, and 2) prescribe the level and modes of seismic damage for L2. This dual level approach is particularly useful in regions of moderate and high seismicity where meeting the specified damage criteria for L2 may not be sufficient to ensure the desired degree of safety and serviceability during L1. Or meeting the performance standard for L1 is not sufficient to ensure the specified performance standard for L2. It should be noted here that stronger L2 excitations will not necessarily solely dictate the final design, which may be highly influenced or even dominated by a high performance standard for L1.

3.3 Performance Evaluation

As a guide for evaluating performance criteria at a specific port, the relationship between degree of damage and the design earthquake motion is illustrated in Fig 3.5. The curves in this figure form the basis for the performance evaluation procedure. This figure is based on the specification of performance grades in Table 2. The curves in Fig 3.5 indicate the upper limits for the acceptable level of damage over a continuously varying level of earthquake motions, including the designated L1 and L2 motions. Each curve in this figure defined by two control points corresponding to the upper limits of the level of damage for L1 and L2 motions defined in Table 2. For example, the curve defining the upper limit for Grade B should go through a point defining the upper limit for damage degree 1for L1 motion, and another defining the upper limit for damage degree 3 for L2 motion. The shape of the curves may be approximated by line segments through the controlling points or may be refined by referring to typical results of non-linear seismic analysis of port structures.

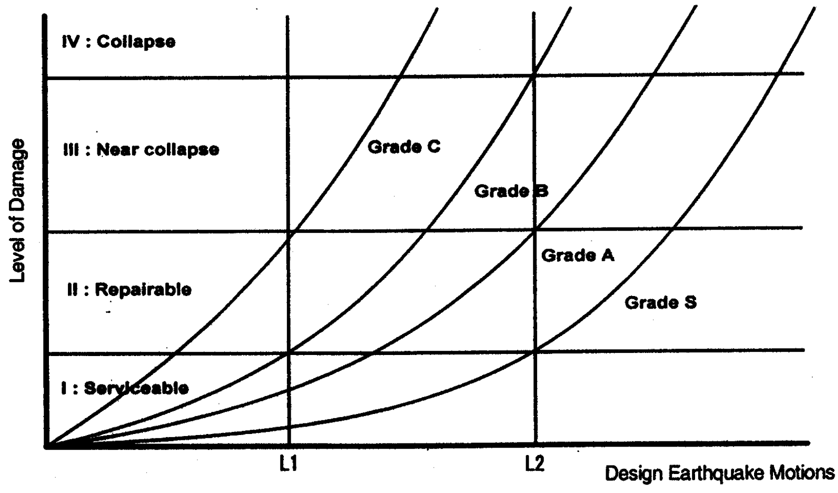


Figure 3.5 Schematic figure of performance grades S, A, B and C (PIANC, 2001)

The vertical coordinates of Fig 3.5 are converted into engineering parameters such as displacements, stress or ductility factors specified by the damage criteria. This conversion allows direct comparison between required performance and seismic response of a structure. The seismic response of a structure is evaluated seismic analysis over L1 and L2 motions and plotted on this figure as 'seismic response curve'. As minimum requirement, analysis should be performed for L1 and L2 earthquake motions. For example, if the structure being evaluated or designed has the seismic response curve 'a' in Fig 3.6 the curve is located below the upper bound curve defining Grade A. Thus this design assures Grade A performance. If an alternative structural configuration yields the seismic performance curve 'b' in Fig 3.6 and a portion of the curve exceeds the upper limit for Grade A, then this design assures only Grade B performance.

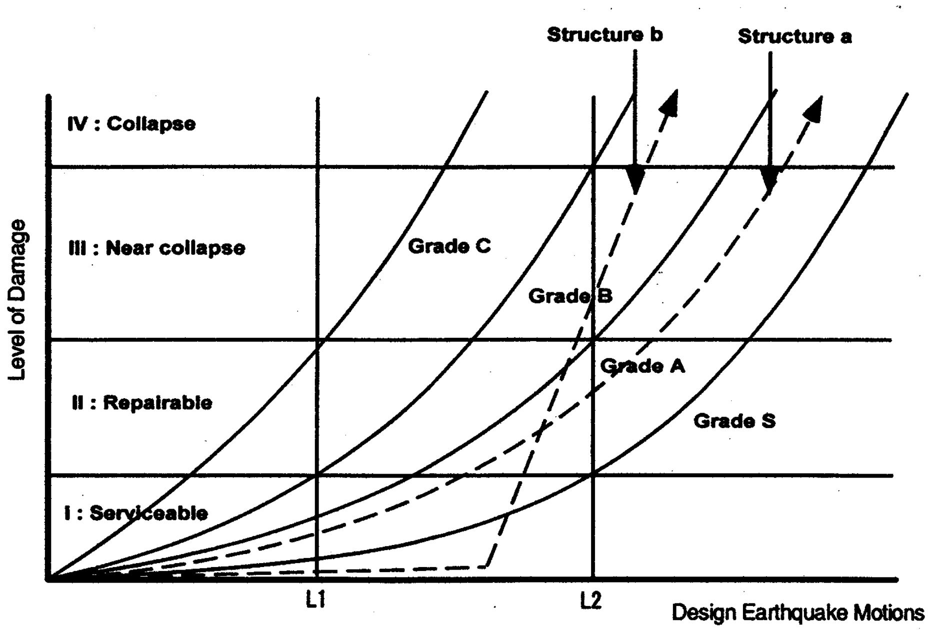


Figure 3.6 Examples of seismic performance evaluation (PIANC, 2001)

CHAPTER IV

SEISMIC ANALYSIS

As in all engineering disciplines, reasonable judgment is required in specifying appropriate methods of analysis and design, as well as the interpretation of the results of the analysis procedures. This is particularly important in seismic design, given the multidisciplinary input that is required for these evaluations, and the influence of this input on the final design recommendations.

4.1 Types of Analysis

The objective of analysis in performance-based design is to evaluate the seismic response of the port structure with respect to allowable limits. Higher capability in analysis is generally required for a higher performance grade facility. The selected analysis methods should reflect the analytical capability required in the seismic performance evaluation.

A variety of analysis methods are available for evaluating the local site effects, liquefaction potential and the seismic response of port structures. These analysis methods are broadly categorized based on a level of sophistication and capability as follows:

- Simplified analysis: Appropriate for evaluating approximate threshold limit for displacements and/or elastic response limit and an order-of-magnitude estimate for permanent displacements due to seismic loading.
- Simplified dynamic analysis: Possible to evaluate extent of displacement/stress/ductility/strain based on assumed failure modes.

- Dynamic analysis: Possible to evaluate both failure modes and the extent of the displacement/stress/ductility/strain.

Table 4.1 shows the type of analysis that maybe most appropriate for each performance grade. The principal applied here is that the structures of higher performance grade should be evaluated using more sophisticated methods.

Table-4.1 Types of analysis related to performance grades (PIANC, 2001)

Type of analysis	Performance grade			
	Grade C	Grade B	Grade A	Grade S
Simplified analysis: Appropriate for evaluating approximate threshold limit for displacements and/or elastic response limit and an order-of-magnitude displacements	□	◇	◇	◇
Simplified dynamic analysis: Of broader scope and more reliable, possible to evaluate extent of displacement/stress/ductility/strain based on assumed failure modes		□	◇	◇
Dynamic analysis: Most sophisticated. Possible to evaluate both failure modes and extent of displacement/stress/ductility/strain			□	□

Index:

□ Standard/final design

◇ Preliminary design or low level of excitations

4.2 Steps of Seismic Analysis

Seismic analysis of port structures accomplished in three steps that include assessment of regional seismicity, the geotechnical hazards, and soil structure analysis. The first step is to define the earthquake motions at the bedrock in Fig 2.1. This is typically accomplished seismic hazard analysis based on geologic, tectonic and historical seismicity data available for the region of interest. One of the key parameters in engineering design practice is the intensity of bedrock motion defined in terms of peak ground acceleration (PGA), or in some cases peak ground velocity (PGV). This parameter is used either by itself or to scale relevant ground motion characteristics, including response spectra and time histories. In the probabilistic seismic hazard analysis, the level of bedrock motion is defined as a function of a return period, or a probability of exceedance over a prescribed exposure time. The bedrock motion for a prescribed return period is often specified in codes or standards for a region. As a deterministic study, the map of Earthquake zones in Turkey as shown in Fig 4.1. Using Fig 4.1, the distribution of earthquake zones along the coasts of Turkey are shown in Table 4.2.

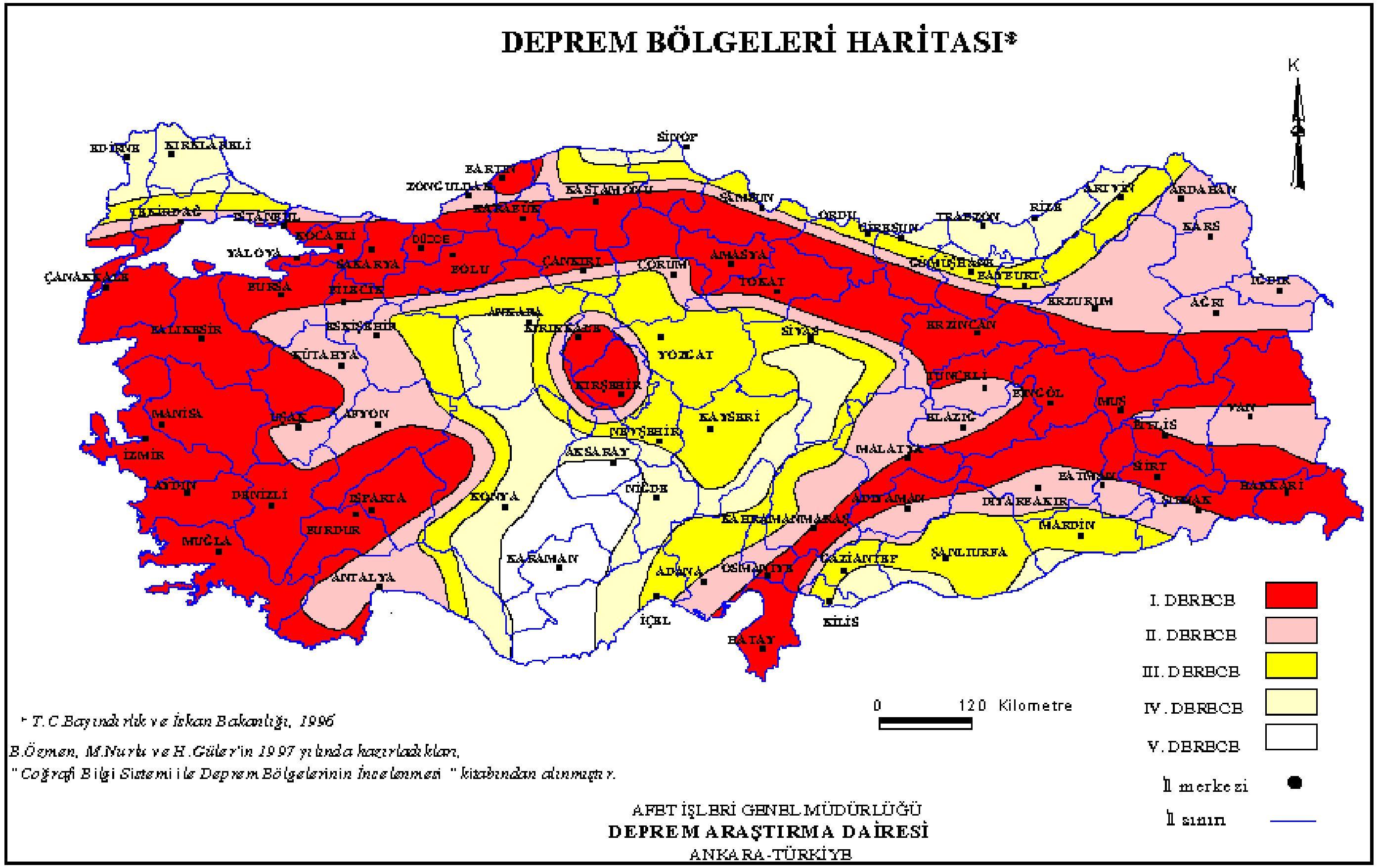


Fig 4.1 Earthquake zones in Turkey

Table 4.2 Earthquake Zones of Coastal Regions

Sea	Coast	Zone
Black Sea	Sarp-Giresun	IV
	Ordu	III
	Ordu-Samsun	II-III*
	Sinop	IV
	Kastamonu	II-III*
	Bartın	I
	Zonguldak	I-II*
	İstanbul	I-II*
	Kırklareli	III-Iv
Marmara Sea	North Coast	I-II*
	South Coast	I
Aegean Sea	Edirne-Muğla	I
Mediterranean Sea	Muğla	I
	Antalya	I-II*
	Alanya-Gazipaşa	II-IV*
	Anamur	V
	Mersin	III-IV*
	Adana	I-II*
	Antakya	I

* Refer to the map of Earthquake zones in Turkey as shown in Fig 4.1. for the exact location of the structure.

4.2.1 First Step of Seismic Analysis

The first step of seismic analysis is to define the Level 1 (L1) and Level 2 (L2) of earthquake motions.

4.2.1.1 Earthquake Motion

Earthquakes are complex natural phenomena, with their origin in the release of tectonic stress which has accumulated in the earth's crust. Their principal effects on port structures are caused by oscillatory ground movements, which depend on such factors as seismic source, travel path, and local site effects.

Each coastal structure, and to a certain extent each structure, requires a specific evaluation of the design parameters of ground motion. The definitions of primary parameters, the recommendations pertaining to the basic data to be collected and the analytical procedures to be followed in the seismic design process should be determined clearly.

4.2.1.1.1 Size of Earthquakes

The basic parameters which characterize the size of earthquakes are intensity, magnitude and energy release.

- **Intensity**

Intensity of the earthquake is a measure of destructiveness of the earthquake, as evidenced by human reaction and observed damage. It varies from one location to another, depending on the size of earthquake, the focal distance and the local site conditions. Seismic damage and the corresponding intensity depend on characteristics of seismic motion, (acceleration, duration and frequency content) as well as the natural frequencies and vulnerability of the affected structures. Intensity is the best single parameter to define the destructiveness of an earthquake at a given site, but it cannot be used as input for dynamic analysis. In many cases, especially for historic earthquakes, it is the only parameter available for characterizing the earthquake motion.

Several different seismic intensity scales have been adopted in different part of the world. Based on intensities at different locations, a map of contours of equal intensity, called an isoseismal map, is plotted.

- **Magnitude and Energy Release**

Magnitude of the earthquake is a physical measure of the size of the earthquake, typically evaluated based on the recorded data. There are several scales based on the amplitude of seismograph records: the Richter local magnitude M_L, the surface wave magnitude M_S, the short-period body wave magnitude m_b , the long-period body wave magnitude m_B and the Japan Meteorological Agency magnitude M_J. Moment magnitude M_W is calculated from the seismic moment, which is a direct measure of the factors that produce the rupture along the fault. The use of M_W is presently preferred by seismologists to avoid the saturation deficiency of the other scales. M_W can be obtained by energy release calculations (Lay & Wallace).

$M_W = (\log M_0 / 1,5) - 10,73$ where M_0 is the seismic moment.

Generally, determination of M_0 is much more complicated than magnitude measurement, although modern seismic analyses are routinely providing M_0 for all global events larger than M_W=5.

Earthquakes with magnitude less than 3 are considered as microtremors, while those measuring up to 5 are considered minor earthquakes with little associated damage. Maximum recorded magnitude is about M_W=9,5.

4.2.1.1.2 Strong Ground Motion Parameters

Earthquakes are characterized by the ground motions that they produce, which is usually described by means of one or several of the following parameters or functions. The most important 3 are:

- Peak Ground Horizontal Acceleration, PGH_H, or simply PGA, is the maximum absolute value reached by ground horizontal acceleration during the earthquake. It is also called peak acceleration or maximum acceleration.
- Peak Ground Horizontal Velocity, PGV_H, or simply PGV, is the maximum horizontal component of the ground velocity during the earthquake.
- Acceleration Response Spectrum S_A (T,D) represents the maximum acceleration (absolute value) of a linear single degree-of-freedom (SDOF) oscillator, with period T and damping D% of critical, when the earthquake motion is applied to its base. The SDOF oscillator is the simplest model of a structure. Thus the spectrum represents a good approximation of the response of the different structures when they are subjected to an earthquake. Similarly, there is a Velocity Response Spectrum, S_V (T,D)

4.2.1.1.3 Seismic Source and Travel Path Effects

The tectonic mechanism in the seismic zone, the source-to-site distance, and the attenuation characteristics of the motions along the travel path, influence the resulting ground motion at the site of interest.

In practice, the effects of seismic source and travel path are taken into account through magnitude and distance. The movements at the bedrock or at an outcropping rock has an amplitude that increases with magnitude and decreases with distance. Predominant periods are influenced by same factors. Generally the greater the magnitude or focal distance is, the greater the pre dominant period.

4.2.1.1.4 Seismic Hazard and Design Earthquake Motion

Peak horizontal acceleration, peak horizontal velocity and response spectra ordinates are commonly used to characterize the seismic hazard at a given site.

Probabilistic determination method of Seismic Hazard and Design Earthquake Motion is given below:

Several values of the earthquake motion parameters are used, usually acceleration or response spectra ordinates, associated with annual exceedance probability. The procedure for probabilistic analysis is shown schematically in Fig 4.2.

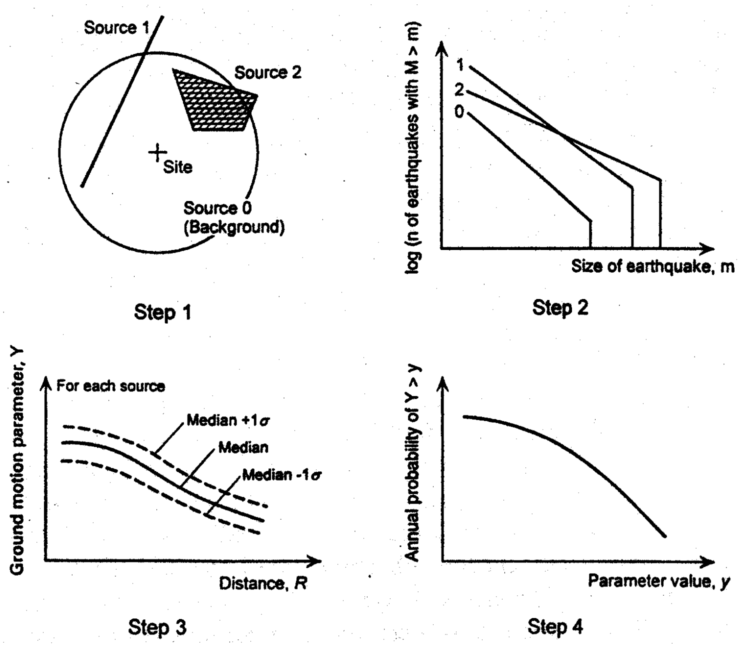


Figure 4.2 Main steps of a probabilistic seismic hazard analysis (PIANC, 2001)

The process includes the following steps: (PIANC, 2001)

Step-1. Identification of active faults and other seismic sources.

All sources capable of producing significant ground motion at the site must be considered. The locations and other parameters of active, and potentially active seismic sources, should be identified. The temporal occurrence of earthquakes should also be characterized. In addition to hazards associated with the specific faults, broader seismotectonic provinces, i.e. regions with uniform tectonic and seismic conditions, are often defined. Recent earthquakes in several seismically active regions of the world demonstrate that the current state of knowledge of both the spatial and temporal occurrence of potentially damaging earthquakes can be incomplete. This uncertainty in the characterization of the seismic hazard is compounded in regions of low- to moderate-seismicity, and in areas where the seismic sources are not well understood. In light of the seismic hazard associated with unidentified sources, the inclusion of areal, or 'random', sources is warranted in most regions of the world. The distribution and rate of occurrence of earthquakes associated with areal sources are specified based on the nature of the seismotectonic province.

The area studied should include seismic sources, both on shore and off-shore. The methodology to be applied includes the interpretation of:

- Topographic and bathymetric maps;
- Seismicity maps;
- Geophysical surveys;
- Repeated high precision geodetic measurements;
- Aerial photographs;
- Geomorphological data;
- Stratigraphic correlations;
- Paleoseismicity (i.e. geologic guidance for pre-historic earthquakes).

Step-2**.** Characterization of each seismic source activity.

The parameters of the earthquake occurrence statistics are defined, including the probability distribution of potential rupture locations within each source and the recurrence relationship. Commonly, it is assumed that all points within the source have the same probability of originating an earthquake. The recurrence relationship specifies the average rate at which an earthquake of a given size will be exceeded, and also, the maximum earthquake.

For modelling the occurrence of earthquakes of different magnitudes, the conventional exponential model (Gutenberg and Richter, 1944), or any of its variants are commonly used. They are based on the Gutenberg-Richter relationship that relates magnitude (or intensity) M with the mean annual number of events, n, that exceeds magnitude (or intensity) M:

$$\text{Log } n = a - bM$$

The coefficients a and b must be obtained by regression of the data of each seismic source. They could depend on the range of earthquake sizes used in the regression. If the seismic catalogue is incomplete for small earthquakes, as it is usual, only earthquakes with a size beyond a certain threshold level must be used.

Step-3. Determination of the attenuation relationship for the acceleration, response spectra ordinates or other parameters of interest.

Ground motion parameters (e.g., PGA, PGV, spectral acceleration) are routinely estimated in practice based on the probabilistic evaluation of routinely mean values obtained from the attenuation relationships. In regions of high seismicity, or in applications involving long exposure intervals that approach the return period for the largest earthquake(s) expected in the region, the standard deviation term established for the specific attenuation relationship being employed will often be used. The application of the standard deviation term in estimates of the ground motion parameters accounts for the probability of experiencing greater than mean motions during the period of interest. The mean attenuation function and the standard deviation should be computed through the statistical analysis of data from earthquakes of the same region or, at least, from earthquakes of similar tectonic environment, recorded in stations with travel paths and local ground conditions similar to those of the site of interest.

In recent years, it has become common practice to use specific attenuation relationships for each of the spectral ordinates of different periods. A general expression for an attenuation relationship is:

$$\text{Log } y_g = f_1(F_T) + f_2(M) + f_3(R) + f_4(S_T) + \varepsilon_\sigma$$

Where:

y_g = ground motion parameter or response spectrum ordinate

F_T = a set of discrete variables describing the fault type

M = magnitude

R = a measure of distance

S_T = a set of discrete variables describing the site subsoil conditions or a continuous variable depending on the average shear wave velocity in the deposit

f_i =functions, f_2 is often assumed linear in powers of M, f_3 depends on R, log R and, sometimes, M

ε_σ = a random error term with zero mean and σ standard deviation

This procedure allows the inclusion of fault type and distance effects and supplies appropriate response spectra for rock. However, results for soil sites are averages of values from different soil conditions and will not represent any particular site. In many cases, it may be preferable to first obtain the ground motion parameters in rock and then compute the seismic response at the ground surface.

Step-4. Definition of the seismic hazard.

Ground motion, primarily described by PGA and spectral ordinates must be defined.

Other parameters such as intensity or duration can also be obtained in a similar way.

i. Calculate the annual number of occurrences of earthquakes from each source which produce, at the site, a given value of the PGA_H (or other earthquake motion parameters)

ii. Calculate the total number, n, of exceedance.

iii. Calculate the reciprocal of the mean annual rate of exceedance for the earthquake effect (PGA_H):

T_R =1/n

This is the return period of earthquakes exceeding that PGA_H

iv. Calculate the probability $P_R(a_{max}, T_L)$ of the PGA_H being exceeded in the life T_L of the structure.

$P_R(a_{max}, T_L)=1-(1-n)^{TL} = T_{L*}n = T_L/ T_R$ (for $T_L/ T_R << 1.0$)

The results of the analysis, sensitive to the details of the procedures used, reflect the seismic hazard of each site.

Step-5. If the attenuation relationships do not match with the local site conditions (e.g. if attenuation relationships for rock are used and there is a surface soil deposit), convert the motion parameters to the specific site conditions using empirical amplification ratios or numerical dynamic soil response models.

4.2.2 Second Step of Seismic Analysis

The second step of seismic analysis involves the following two interrelated aspects of dynamic soil response (1) an evaluation of local site effects for obtaining the earthquake motions at or near ground surface; and (2) an assessment of the liquefaction resistance of the near surface sandy soils and the associated potential for ground failures.

4.2.2.1 Local Site Effects

The soil deposits at a particular site may significantly modify the bedrock ground motion, changing the amplitude, frequency content and duration. This is due to the dynamic response characteristics of the soils, and it has been termed 'local site effects'. Local site effects depend on the material properties of the subsoil and stratigraphy, as well as the intensity and frequency characteristics of the bed rock motion. As strong ground motion propagates upwards, towards the ground surface, the reduction in the strength and stiffness of soil deposits tends to amplify the ground motions. In engineering practice, local site effects are evaluated either by using prescribed site amplification based on statistical analysis of existing data or a site specific response analysis. The site amplification factors are often specified in codes and standards, and used to scale the bedrock PGA or PGV to obtain the corresponding values at the ground surface, or used to scale bedrock response spectra to define the ground surface response spectra. Proposed site classification system for seismic site response is given in Table 4.3. The graphs for maximum acceleration at soil site and amplification factor for the variable periods are given in Fig 4.3.

Table 4.3 Proposed Site Classification System For Seismic Site Response (Seed et al, 1997)

Site Class	Site Condition	General Description	Site Characteristics
(A_0)	A_0	Very hard rock	V_s(avg)>5,000ft/sin top 50ft.
A	A_1	Component rock with little or no soil and/or weathered rock veneer.	2,500 ft/s ≤ V_s(rock) ≤ 5,000ft/s and $H_{soil+weathered\ rock}$ ≤ 40 ft.with V_s>800 ft/s (in all but hte top few feet[3])
AB	AB_1	Soft, fractured and/or weathered rock	For both AB_1 and AB_2: 40ft ≤ $H_{soil+weathered\ rock}$ ≤150ft,and V_s>800ft/s (in all but hte top few feet[3])
	AB_2	Stiff, very shallow soil over rock and/or weathered rock	
B	B_1	Deep, primarily cohesionless[4] soils (H_{soil}<or= 300 ft.)	No "soft clay" (see note 5), and $H_{cohesive\ soil}$< 0.2 $H_{cohesionless\ soil}$
	B_2	Medium depth, stiff cohesive soils and/or mix of cohesionless with stiff cohesive soils; no "soft clay".	$H_{all\ soils}$ ≤ 200ft, and V_s (cohesive soils)>600ft/s (see note 5)
C	C_1	Medium depth, stiff cohesive soils and/or mix of cohesionless with stiff cohesive soils; thin layer(s) of soft clay.	Same as B_2 above,except 0ft<$H_{soft\ clay}$ ≤10ft (see note 5)
	C_2	Very deep, primarily cohesionless soils.	Same as B_1 above, except H_{soil}>300ft.
	C_3	Deep, stiff cohesive soils and/or mix of cohesionless with still cohesive soils; no "soft clay"	H_{soil}>200ft, and V_s (cohesive soils)>600ft/s
	C_4	Soft, cohesive soil at small to moderate levels of shaking.	10ft< $H_{soft\ clay}$ ≤ 90ft, and $A_{max,rock}$<or=0.25 g
D	D_1	Soft, ccohesive soil at medium to strong levels of shaking.	10ft< $H_{soft\ clay}$ ≤ 90ft, and 0.25g< $A_{max,rock}$ ≤ 0.45g, or (0.25g< $A_{max,rock}$ ≤ 0.55g and M ≤ 7-1/4)
(E)[6]	E_1	Very deep, soft cohesive soil	$H_{soft\ clay}$>90ft (see note 5)
	E_2	Soft cohesive soil and very strong shaking	$H_{soft\ clay}$>10ft and either $A_{max,rock}$>0.55g or $A_{max,rock}$>0.45g and M>7-1/4
	E_3	Very high plasticity clays.	H_{clay}>30ft with PI>75% and V_s<800ft/s
(F)[7]	F_1	Highly organic and/or peaty soils.	H>10ft of peat and/or highly organic soils.
	F_2	Sites likely to suffer ground failure due eitherto significant soil liquefaction or other potential modes of ground instability.	Liquefaction and/or other types of ground failure analysis required.

Notes:
1. H=total (vertical) depth of soils of the type or types referred to.
2. Vs= seismic shear velocity (ft/s) at small shear strains (shear strain 10^{-4}%).
3. If surface soils are cohesionless, V_s may be less than 800ft/s in top 10 feet.
4."Cohesionless" soils = soils with less than 30% "fines" by dry weight. "Cohesive soils" =soils with more than 30%"fines" by dry weight, and 15% <or=PI (fines)<or=90%. Soils with more than 30% fines, and PI (fines)<15%are considered "silty" soils herein, and these should be (conservatively) treated as "cohesive" soils for site classification purposes in this Table.
5. "Soft clay" is defined as cohesive soil with (a) Fines content >or= 30%, (b) PI (fines)>or=20%, and (c) V_s<or=600ft/s.
5. "Soft Clay" is defined as cohesive soil with (a) fines content >or=30%, (b)PI (fines)>or=20%, and (c) V_s<or=600ft/s
6. Site-specific geotechnical investigations and dynamic site response analyses are strongly recommended for these conditions. Response characteristics within this Class (E) of sites tends to be more highly variable than for classes A_0 through D, and the response projections herein should be applied conservatively inthe absence of (strongly recommended) site-specific studies.
7. Site-specific geotechnical investigations and dynamic site response analyses are required for these conditions. Potentially significant ground failure must be mitigated, and/or it must be demonstrated that the proposed structure/facility can be engineered to satisfactorily withstand such ground failure.
8. 1ft=0.3m.

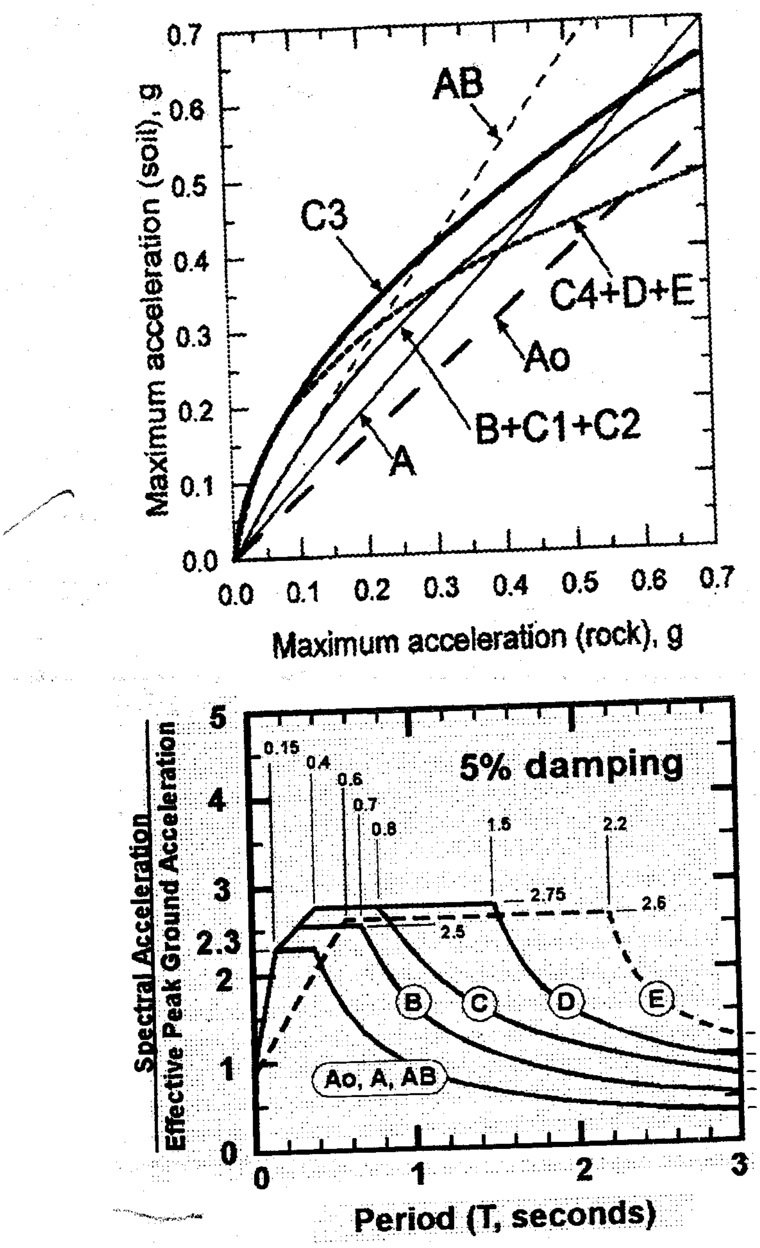


Figure 4.3 Graphs for obtaining amax due to site classification and amplification factor for variable periods (Seed et.al. 1997)

Site response analysis details can be found in Table 4.4.

In simplified analysis local site effects are evaluated based on the thickness of the deposits and the average stiffness to a specified depth (generally 30 m), or over the entire deposit above the bedrock. This information is then used to establish the site classification, leading to the use of specified site amplification factors or site dependent response spectra. This type of procedure is common in codes and standards.

In simplified dynamic analysis, local site effects are evaluated numerically with models such as common equivalent linear, total stress formulations. Soil layers are idealized as horizontal layers of infinite lateral extent. (i.e. 1D). These methods are used to generate time histories acceleration, shear stress, and shear strain at specified locations in the soil profile.

In both of these categories of analysis, the computed ground surface earthquake motion parameters are used as input for subsequent simplified structural analysis.

Table 4.4 Methods for site response analysis (PIANC, 2001)**

Type of analysis		Simplified analysis	Simplified dynamic analysis	Dynamic analysis***
Site response analysis	Method	Site category	1D total stress (equivalent linear) analysis	1D effective stress (non linear) analysis, or 1D total stress (equivalent linear) analysis*
	Input parameters	Peak bedrock acceleration CPT q_c/SPT N-values Stratigraphy	Time history of bedrock earthquake motion V_s,G/G_0-γ, D-γ curves	For effective stress analysis: Time history of bedrock earthquake motion Undrained cyclic properties For total stress analysis: the same as those for simplified dynamic analysis
	Output of analysis	Peak ground surface motion (PGA, PGV) Design response spectra	Time history of earthquake motion at ground surface and within the subsoil Computed response Spectra at ground surface	Time history of earthquake motion at ground surface and within the subsoil

*If the bottom boundary of the domain in a soil-structure interaction analysis differs from the bedrock (i.e. if the bedrock level is too deep for soil-structure interaction analysis), local site effects below the bottom boundary of the soil-structure analysis domain may be evaluated based on 1D effective stress (non-linear) or equivalent-linear (total stress) analysis.

**CPT: cone penetration test, SPT standard penetration test, PGA, PGV: peak ground acceleration and velocity, V_s: shear wave velocity, G/G_0: secant shear modulus (G)over shear modulus at small strain level (G_0), D: equivalent damping factor γ: shear strain amplitude q_c: CPT tip penetration resistance.

***Details of outputs from dynamic analysis are in Table 4.5.

Table 4.5 Outputs from dynamic analysis (PIANC, 2001)

Structure and geotechnical modeling		Structure modeling	
		Linear	Non-linear
Geotechnical modeling	Linear (Equivalent linear)	Peak response displacement/stresses	Failure mode of structure Peak and residual displacement/ductility factor/stresses for structures (assuming there are no effect from residual displacement of soils)
	Non-linear	Failure mode due to soil movement Peak and residual displacement/stresses from soils movement (assuming structure remains elastic)	Failure mode of soil-structure systems Peak and residual displacement/ductility factor/stresses including effects from residual displacements of soils

4.2.2.2 Liquefaction Potential Assessment

Details of Liquefaction Potential Assessment can be found in Table 4.6.

In simplified analysis, the liquefaction potential of sandy soils is evaluated based on standard penetration tests (SPT) or cone penetration tests (CPT) through empirical criteria.

In simplified dynamic analysis, liquefaction potential is evaluated based on comparison of computed shear stresses during the design earthquake and the results of cyclic laboratory tests, and/or based on SPT/CPT data.

The liquefaction potential evaluated through these categories of analysis are used later as input for subsequent simplified deformation analysis of structures at liquefiable sites.

In dynamic analysis, liquefaction potential is often not evaluated independently but is evaluated as part of the soil-structure interaction analysis of port structures.

Table 4.6 Methods for liquefaction potential assessment(PIANC,2001)**

Type of analysis		Simplified analysis	Simplified dynamic analysis	Dynamic analysis
Liquefaction potential assessment	Method	Field correlation (SPT/CPT/V_s)	Laboratory cyclic tests and/or Field correlation (SPT/CPT/V_s) +1D total stress analysis	Laboratory cyclic tests and/or Field correlation (SPT/CPT/V_s) +1D effective stress analysis or 1D total stress analysis*
	Input parameters	Peak ground surface acceleration (PGA) CPT q_c/SPT N-values/V_s Stratigraphy	Time history of earthquake motion at ground surface, or time histories of shesr stresses in the sub soil Liquefaction resistance, (τ/σ'_{v0}) or γ_{cyc} based on laboratory cyclic tests and/or SPT/CPT/V_s	For effective stress analysis: Time history of bedrock earthquake motion Undrained cyclic properties based on laboratory cyclic tests and/or SPT/CPT/V_s For total stress analysis: the same as those for simplified dynamic analysis
	Output of analysis	Liquefaction potential (F_L)	Liquefaction potential (F_L) Excess pore water pressure ratio (u/σ'_{v0})	Excess pore water pressure ratio (u/σ'_{v0}) Depth and time at the onset of liquefaction

*If the bottom boundary of the domain in a soil-structure interaction analysis differs from the bedrock (i.e. if the bedrock level is too deep for soil-structure interaction analysis), local site effects below the bottom boundary of the soil-structure analysis domain may be evaluated based on 1D effective stress (non-linear) or equivalent-linear (total stress) analysis.

**CPT: cone penetration test, SPT standard penetration test, PGA: peak ground acceleration, V_s: shear wawe velocity, γ_{cyc}: cyclic shear strain amplitude, q_c: CPT tip penetration resistance, F_L:factor of safety against liquefaction

u/σ'_{v0}:excess pore water pressure (u) over initial effective vertical stress (σ'_{v0}), τ/σ'_{v0}:shear stress ratio

4.2.3 Third Step of Seismic Analysis (Soil-structure Interaction Analysis)

Once the ground motion and geotechnical parameters have been established, then seismic analysis of the port structure(s) can proceed.

The method of analysis for a port structure depends on structural type. The appropriate method may be chosen by referring to Table 4.7.

Table4.7 Analysis methods for port structures (PIANC, 2001)*

<table>
<tr><td rowspan="2">Type of analysis</td><td rowspan="2">Simplified analysis</td><td rowspan="2">Simplified dynamic analysis</td><td colspan="2">Dynamic analysis</td></tr>
<tr><td>Structural modeling</td><td>Geotechnical modeling</td></tr>
<tr><td>Gravity quay wall</td><td rowspan="2">Empirical/pseudo-static methods with/without soil liquefaction</td><td rowspan="2">Newmark type analysis
Simplified chart based on parametric studies</td><td rowspan="6">FEM/FDM**

Linear or Non-linear analysis

2D/3D***</td><td rowspan="6">FEM/FDM**

Linear (Equivalent linear) or Non-linear analysis

2D/3D***</td></tr>
<tr><td>Sheet pile quay wall</td></tr>
<tr><td>Pile-supported wharf</td><td>Response spectrum method</td><td>Pushover and response spectrum methods</td></tr>
<tr><td>Cellular quay</td><td>Pseudo-static analysis</td><td>Newmark type analysis</td></tr>
<tr><td>Crane</td><td>Response spectrum method</td><td>Pushover and response spectrum methods</td></tr>
<tr><td>Breakwater</td><td>Pseudo-static analysis</td><td>Newmark type analysis</td></tr>
</table>

* Proposed damage criterias for port structures can be found at PIANC, 2001

** FEM/FDM: Finite element method/finite difference method.

*** 2D/3D: Two/three-dimensional analysis.

CHAPTER V

APPLICATION OF THE PERFORMANCE-BASED METHODOLOGY

As performance-based design is an emerging methodology and is not well-known as conventional seismic design, a typical port structure, gravity quay wall (Grade A) is selected to illustrate the basic procedures employed. This example is based on field case studies (Yüksel et.al, 2003), modified slightly to fit, where necessary, to the seismic guidelines. Thus the example given is intended to present a case study for a hypothetical gravity quay wall structure constructed on the İzmit Bay coast. This design example will illustrate only the application of the simplified and simplified dynamic analysis procedures for preliminary design at low level of excitations.

Major input parameters for analysis and analysis output for a gravity quay wall are given Tables 5.1 - 5.2 respectively.

5.1 Case Study

Cross section and dimensions of the gravity quay wall selected as a design example are given in Fig 5.1. Simplified geotechnical conditions of the case study are given in Fig 5.2 where backfill soil is considered as non liquefiable soil.

Grade A is selected as performance grade. Therefore reference levels of earthquake motions and corresponding acceptable level of damages becomes as:

(L1) $\Rightarrow$ Degree I: Serviceable

(L2) $\Rightarrow$ Degree II: Repairable

Lifetime (T_L) of the structure is taken as; T_L=50 years.

Design earthquake motions at bedrock are given for İzmit Bay region as PGA (Peak Ground Acceleration):

For L1 with %50 exceedance (frequent) a_{max}=0.06g.

For L2 with %10 exceedance (rare) a_{max} =0.25g. (Çetin et.al, 2002).

Table 5.1 Major input parameters for analysis for gravity quay wall (PIANC, 2001)

Type of analysis	Simplified analysis	Simplified dynamic analysis		Dynamic analysis
Method	Pseudo/empirical methods	Newmark type method	Simplified chart based on parametric studies	FEM/FDM
Design parameters	k_e: equivalent seismic coefficient k_t: threshold seismic coefficient (Geometrical extent of liquefiable soils relative to the position and dimensions of a wall for a liquefiable site)	Empirical equations: a_{max}: peak acceleration v_{max}: peak velocity Time history analysis: time histories of earthquake motions a_t: threshold acceleration	a_{max}: peak acceleration at the bedrock Cross section of wall Index properties of soil Including SPT N-values	Time histories of earthquake motions at the bottom boundary of analysis domain Cross section of wall For equivalent linear Geotechnical analysis: G/G_0-γ & D-γ cuves For non-linear geotechnical analysis: Undrained cyclic properties And G, K shear and bulk modulus, in addition to the Geotechnical parameters For pseudo-static and Simplified analyses
Input parameters	Results of site response analysis, including a_{max}, and liquefaction potential assessment Cross section of wall Geotechnical parameters, including c,ϕ: cohesion and internal friction angle of soils; μ_b, δ: friction angles at bottom and back face of wall; ground water level			

Table 5.2 Analysis output for a gravity quay wall (PIANC,2001)

Analysis type	Simplified analysis	Simplified dynamic analysis	Dynamic analysis
	Threshold limit Order of magnitude displacement	Wall displacement	Response/failure modes Peak and residual displacements

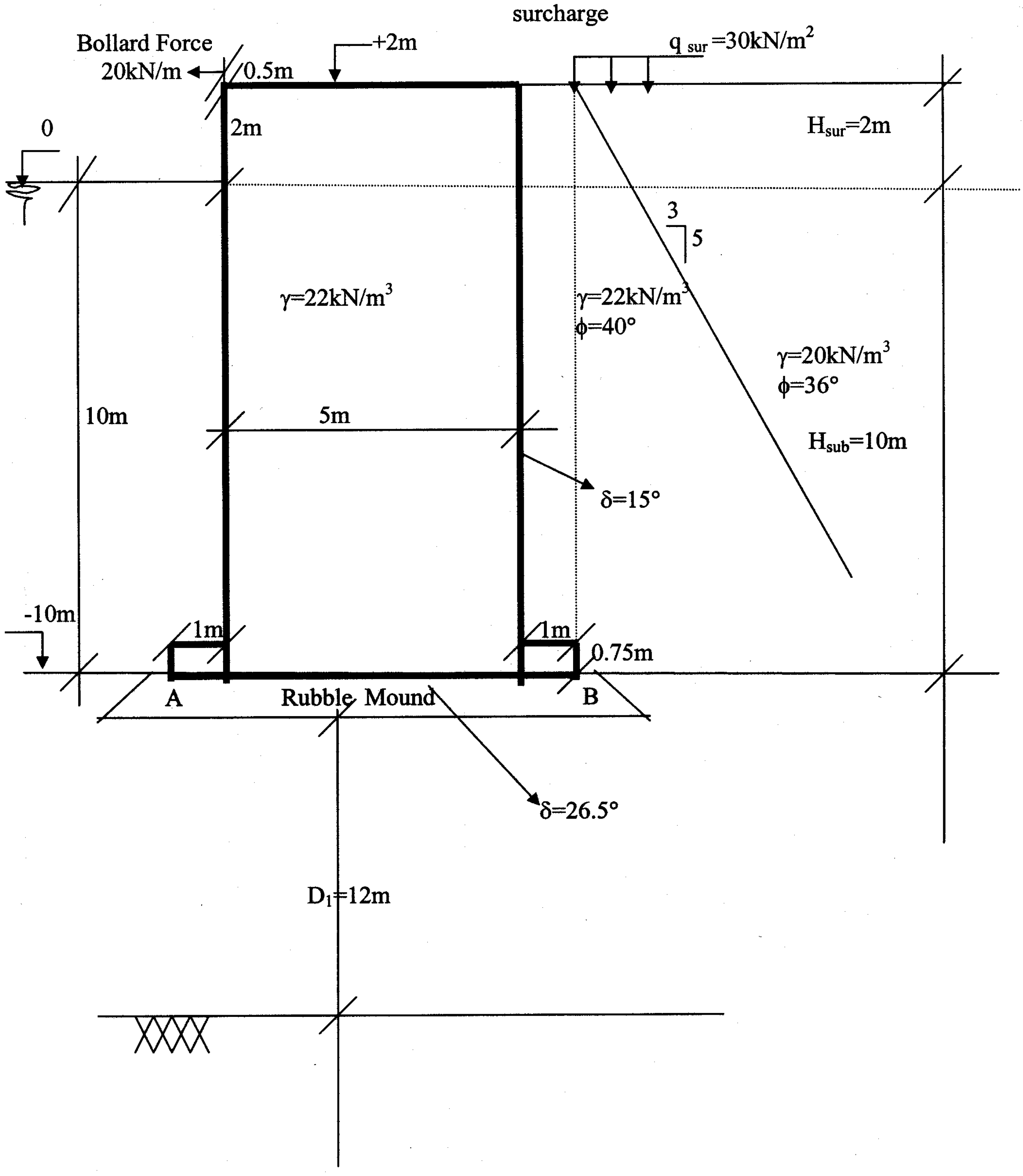


Figure 5.1 Cross section and dimensions of the wall

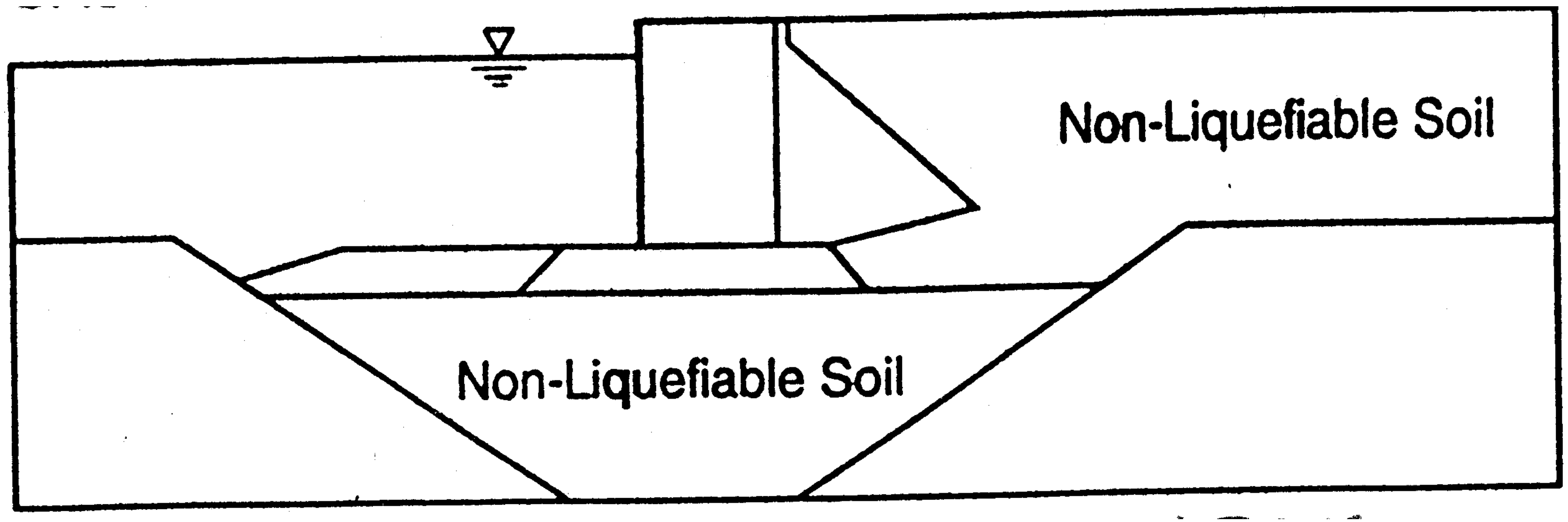


Figure 5.2 Simplified Geotechnical Conditions For a Gravity Quay Wall (PIANC,2001)

Damage criteria for a grade A gravity quay wall is given in Table 5.3 (PIANC, 2001).

Table 5.3 Damage criteria for a grade A gravity quay wall (PIANC, 2001)

Extent of damage		Degree I	Degree II	Degree III	Degree IV
Gravity wall	Normalized residual horizontal displacement (d/H)*	Less than 1.5%	1.5-5%	5-10%	Larger than 10%
	Residual tilting towards the sea	Less than 2°	2-5°	5-8°	Larger than 8°

* d: residual horizontal displacement H: height of gravity wall.

From Table 5.3, maximum allowable displacements (m) for design motions L1 (Degree I) and L2 (Degree II) are obtained in terms of normalized displacement (d/H) and presented in Table 5.3 (a). Using Table 5.3 (a) limiting curve for horizontal displacement is drawn and presented in Fig 5.3 (a).Similarly from Table 5.3 maximum allowable tilting (°) are computed for L1 and L2 and presented in Table 5.3 (b). In Fig 5.3 (b) limiting curve for tilting are presented.

Table 5.3 (a) Maximum allowable limits of displacement (m.)

Reference level of earthquake motion and corresponding damage levels	Normalized displacement (d/H*)	Maximum allowable displacement (m.) (d)
L1⇒ Degree I: Serviceable	<0.015	0.18
L2⇒ Degree II: Repairable	0.015-0.05	0.6

*H=12m

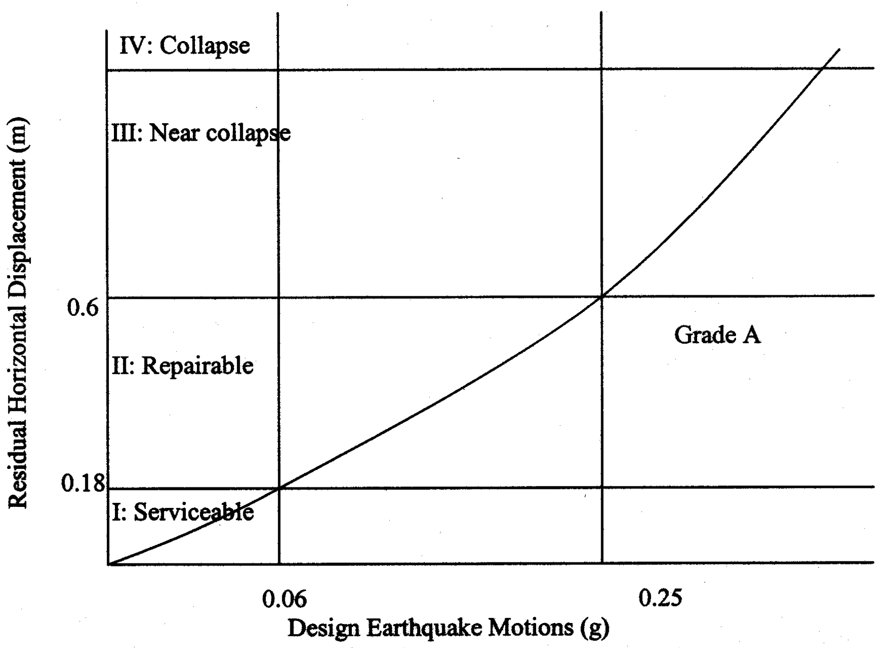


Figure 5.3 (a) Limiting curve for horizontal displacement

As it is seen from Figure 5.3 (a) for grade A performance requirement which is selected for the gravity quay wall, under the design earthquake motions 0.06g for level L1 (serviceable) and 0.25g for level L2 (repairable), the maximum allowable displacements are obtained as 0.18m and 0.6m respectively.

Table 5.3 (b) Maximum allowable limits of tilting (°)

Reference level of earthquake motion and corresponding damage levels	Residual tilting towards the sea	Maximum allowable tilting (°)
L1⇒ Degree I: Serviceable	<2°	2°
L2⇒ Degree II: Repairable	2-5°	5°

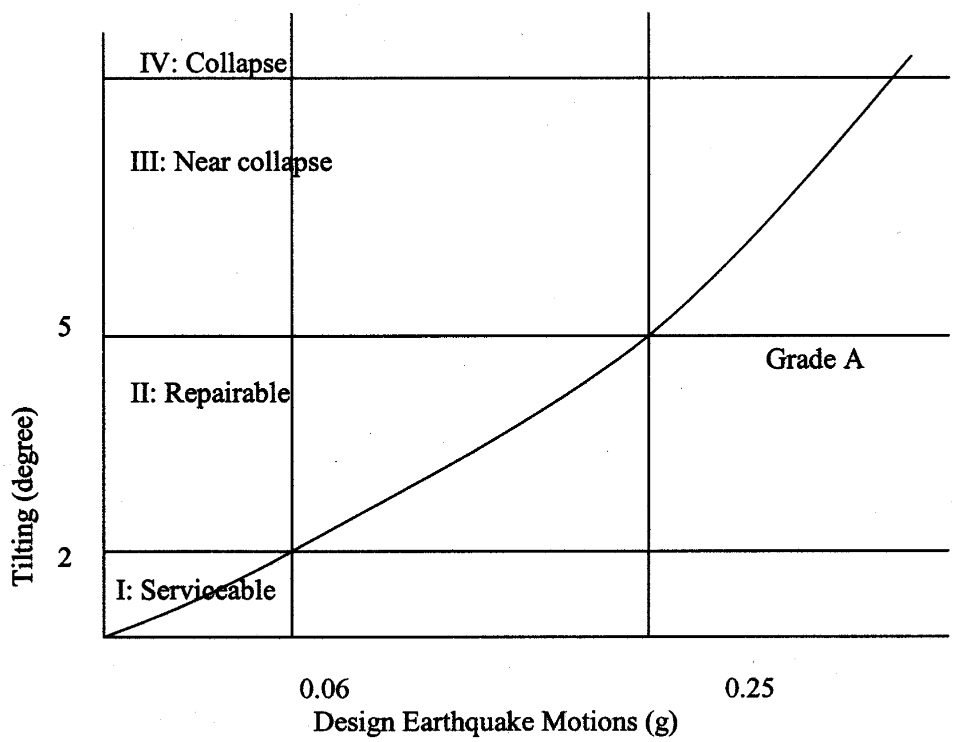


Figure 5.3 (b) Limiting curve for tilting

As it is seen from Figure 5.3 (b) for grade A performance requirement which is selected for the gravity quay wall, under the design earthquake motions 0.06g for level L1 (serviceable) and 0.25g for level L2 (repairable), the maximum allowable tiltings are obtained as 2° and 5° respectively.

The stability computations for the gravity quay wall for the required performance grade A is carried out in two stages as simplified and simplified dynamic analysis.

5.1.1 Simplified Analysis:

Simplified analysis is appropriate for evaluating approximate threshold limit for displacements and/or elastic response limit and an order-of-magnitude estimate for permanent displacements due to seismic loading. The simplified analysis was based on pseudo-static analysis.

Results of the simplified analysis are appropriate for evaluating the approximate threshold level of damage, which ensures at least the repairable state of structural performance for L1. Whether or not the approximate threshold level ensures the serviceable state of structural performance for L1 depends on the details in evaluating the design parameters for the pseudo-static method.

Input parameters and geotechnical investigation methods for simplified analysis for a gravity quay wall are given in Tables 5.4 - 5.5 respectively (PIANC, 2001).

Table 5.4 Input parameters for simplified analysis for gravity quay wall (PIANC, 2001)

Conditions	Design parameters	Input parameters
Earthquake	k_e: equivalent seismic coefficient	a_{max}: Regional PGA at bedrock Site Category (site amplification factor)
Structural	k_t: threshold seismic coefficient (Geometrical extent of liquefiable soils relative to the position and dimensions of a wall for a liquefiable site)	W, H & water level: Cross-sectional dimensions of a gravity wall
Geotechnical		c,ϕ: cohesion and internal friction angle of soils μ_b,δ: friction angles at the bottom and back face of the wall ground water level (SPT/CPT for a liquefiable site)

Table 5.5 Geotechnical investigation methods for simplified analysis for gravity quay wall (PIANC, 2001)

Type of analysis	Analysis method	Input seismic action	Soil parameters	Geotechnical investigation methods			
				Gathering existing data	Field	Laboratory	
					PT	Index Prop.	Static
SRA	Site category	a_{max} at bedrock	Soil profile and classification	□	□	◇	
LIQ	SPT/CPT liquefaction charts	a_{max} at surface	N/q_c	□	□	◇	
SSI	Pseudo-static	Output of SRA	C, Φ,μ_B,δ	□	□	◇	◇

Key: SRA=Seismic Response Analysis; LIQ=Liquefaction; SSI=Soil Structure Interaction. PT=Penetration Tests,

□ used as a standard ◇ use depends on design conditions

5.1.1.1 Computations for Simplified Analysis

Computations for the gravity quay wall were carried following the Seismic Design Guidelines for Port Structures (PIANC, 2001). Definitions of the parameters and equations are given below together with the reference Fig5.4. For active earth pressure the design parameters are:

ψ:seismic inertia angle:

$\psi=\tan^{-1}(k_h/1-k_v)$.. (5.1)

where

k_h: horizontal seismic coefficient

An average relationship between the effective seismic coefficient and peak ground acceleration is as given as:

$k_h=0.6*(a_{max}/g)$... (5.2)

where 0.6 is reccommended as the coefficient between k_h and a_{max}/g. (PIANC, 2001)

k_v: vertical seismic coefficient $k_v \approx 0$ in practice (PIANC, 2001)

ϕ : angle of internal friction

δ : friction angle at wall-soil interface

α_{ae} :seismic active angle of failure:

$\alpha_{ae}=\phi+\arctan[(-\tan\phi+\sqrt{\tan\phi(\tan\phi+\cot\phi)(1+\tan\delta\cot\phi)})/(1+\tan\delta(\tan\phi+\cot\phi))]$.......................................(5.3)

K_{ae}: dynamic active earth pressure coefficient

$K_{ae}=\cos^2(\phi-\psi)/[\cos\psi\cos(\psi+\delta)(1+\sqrt{\sin(\phi+\delta)\sin(\phi-\psi)/\cos(\delta+\psi)})^2]$ ....(5.4)

(Mononobe-Okabe, 1924)

P_{ae}: dynamic active earth trust

where

H: Height of the structure

γ_d: unit weight of dry backfill

q_{sur}: uniformly distributed surcharge

$P_{ae}=K_{ae}\gamma_d(1-k_v)H^2/2$ ....(5.5)

if q_{sur} exist γ_d should be substituted with $(\gamma_d+(q_{sur}/H)$.

For the case study, values of γ, ϕ, δ, q_{sur} are given in Table 5.6, and Fig 5.1.

In simplified analysis, computations for the gravity quay wall were carried out for level L1, for İzmit Bay where PGA at bedrock =0.06g is taken. Then the site response was performed and PGA=a_{max}=0.1g at surface was found (Çetin et.al, 2002). Horizontal seismic coefficient k_h is obtained from Eq.5.2 as k_h=0.6*0.1=0.06. Computations for the forces are given in steps (PIANC, 2001).

1) Active earth pressure + thrust

For gravity quay wall for active pressures reference figure is given in Fig5.4.

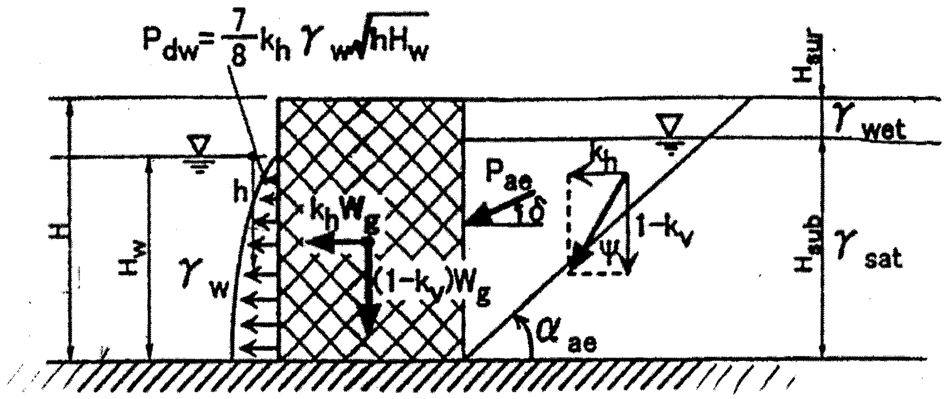


Figure 5.4 Active earth pressures (PIANC, 2001)

Table 5.6 Parameters for the case study

Parameters	Density (γ) (kN/m^3)	Internal friction angle (ϕ) (°)
Caisson	22	--
Backfill soil	20	36
Foundation rubble and rubble backfill	22	40

Joint elements: Friction angle δ=26.5°(bottom of caisson); 15°(behind caisson)

Using parameters given for the case study, design parameters are calculated.
Seismic active angle of failure (α_{ae}) from Eq.5.3
α_{ae}=49.5°
Active soil wedge measured from the vertical direction is 90-α_{ae}=90-49.5=40.5°
Within the soil wedge, areas in m^2 are computed and given with square marks as shown in Figure 5.5.

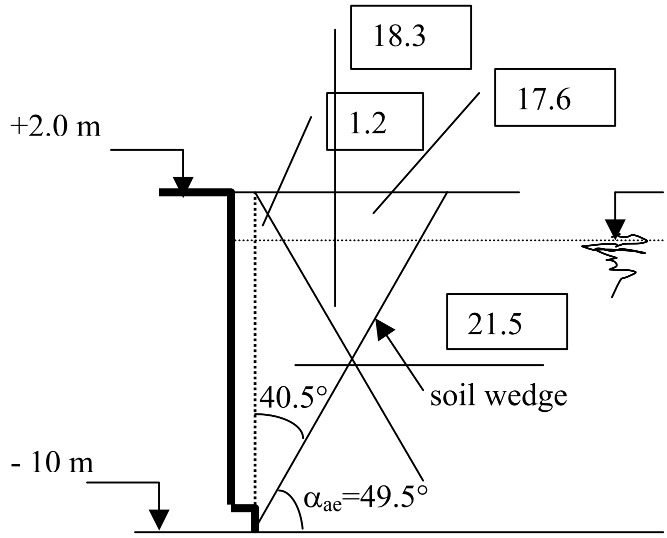
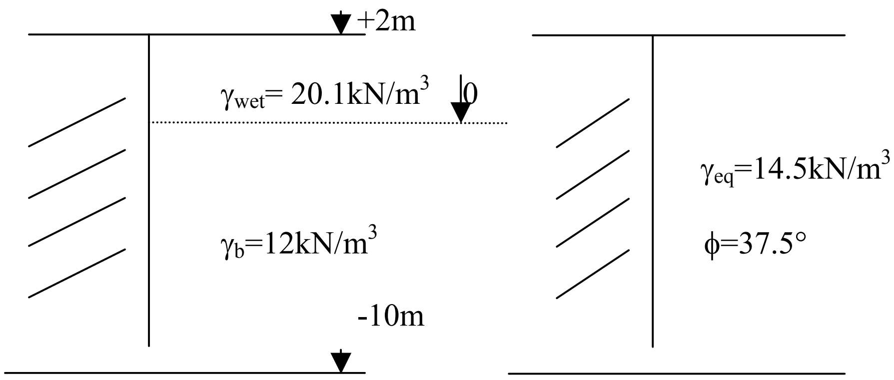


Figure 5.5 Diagram for computations (areas in square marks are in m^2)

Using these areas, a representative unit weight of the backfill material above water table is computed.
Representative unit weight of material above the water table, γ_{wet}
(using weighted areas from Fig 5.5) γ_{wet}=(1.2*22+17.6*20)/(1.2+17.6)=20.1kN/m^3
Computation for equivalent unit weight of backfill (γ_{eq})
For a partially submerged soil γ_{eq} is given by; (PIANC, 2001)
$\gamma_{eq}=\gamma_{wet}*[1-(H_{sub}/H)^2]+\gamma_b*(H_{sub}/H)^2$...(5.6)
where for backfill γ_b, bouyant unit wt of soil is γ_{dry}-γ_{water}=22-10=12kN/m^3
γ_{eq}=20.1*[1-(10/12)^2]+12*(10/12)^2 =14.5 kN/m^3
Representative ϕ for the backfill (using weighted areas from Fig 5.5)
ϕ =(17.6*36+18.3*36+1.2*40+21.5*40)/(17.6+18.3+1.2+21.5)
=37.5°

Original and equivalent parameters are given in Figure 5.6 (a) and (b).



(a) **(b)**

Figure 5.6 Figure for average values

(a)Original parameters

(b)Equivalent parameters

In practice in seismic analysis half the operational surcharge is present during the earthquake (PIANC, 2001).

The modified seismic coefficient (k_h') is:

$k_h' = k_h*(q_{sur}*H/2+\gamma_{wet}*H_{sur}^2/2+\gamma_{wet}*H_{sub}*H_{sur}+\gamma_{sat}H_{sub}^2/2)$ /

$(q_{sur}*H/2+\gamma_{wet}*H_{sur}^2/2+\gamma_{wet}*H_{sub}*H_{sur}+\gamma_b H_{sub}^2/2)$..........................(5.7)

then;

$k_h' = k_h*(30*12/2+*20.1*2^2/2+20.1*10*2+22*10^2/2)/$

$(30*12/2+*20.1*2^2/2+20.1*10*2+12*10^2/2)$

=0.06*1722.2/1222.2

=0.085

k_h' is used to compute ψ, K_{ae} and P_{ae}.

Then, seismic inertia angle $\psi=\tan^{-1}(k_h/1-k_v)$ from Eq.(5.1) becomes;

$\psi=\tan^{-1}k_h'=4.86°$

Dynamic active earth pressure coefficient K_{ae} given in Eq.(5.4) :

$K_{ae}=\cos^2(\phi-\psi)/[\cos\psi\cos(\psi+\delta)\ (1+\sqrt{\sin(\phi+\delta)\sin(\phi-\psi)/\cos(\delta+\psi)}\ )^2]$

Then,

K_{ae}=cos^2(37.5°-4.86°)/[cos4.86°*cos(15°+4.86°)*

(1+$\sqrt{\text{sin(37.5°+15°)sin(37.5°-4.86°)/cos(15° +4.86°)}}$)^2]=0.32

Total earth trust given in Eq.(5.5) in terms of γ_{eq} and q_{sur} (q_{sur} is taken 1/2 q_{sur} for seismic design)

P_{ae}=K_{ae}*(γ_{eq}+1/2*(q_{sur}/H) (1-k_v)H^2/2..(5.8)

=0.32*(14.5+15/12)*12^2/2=362.8 kN/m

The horizontal earth force is

P_{ae}cosδ=362.8*cos15°=350 kN/m

As recommended application point is at 0.45H=5.4m above bed.

The vertical earth force is

P_{ae}sinδ=362.8*sin15°=94kN/m

at the interface between structure and soil. (6m from point A at the base of the wall)

2) Hydrodynamic force

During seismic shaking, the free water exerts dynamic loading most critical at the phase of suction. Load can be calculated by (Westergaard, 1933):

P_{dw}=7*k_h*γ_w*H_w^2/12.. (5.9)

Then;

P_{dw}=7*0.06*10*10^2/12=35kN/m at 0.4*10=4m above bed.

3) Inertia and driving forces due to earthquake

k_h=0.06 is used in these computations.

Caisson: 5*11.25*22*0.06=74.75kN/m at 6.375 m above bed

Footing: 7*0.75*22*0.06=6.93kN/m at 0.375m above bed.

The backfill inertia force: 1*11.25*22*0.06=14.85kN/m at 6.375m above bed.

The static bollard pull (%50 reducing its value during earthquake): 20/2=10kN/m at 12.5m above bed.

4) Vertical Forces:

Caisson dry: 5*2*22=220kN/m at 5/2+1=3.5m from A.

Caisson wet: 5*(10-0.75)*(22-10)=555kN/m at 3.5m from A.

Footing: 7*0.75*(22-10)=63kN/m at 3.5m from A.

Caisson: Σ=838kN/m at 3.5m from A.

Backfill material above water table: 1*2*22=44 kN/m at 7-0.5=6.5m from A.

Backfill material under water table: 1*(10-0.75)*(22-10)=111 kN/m at 6.5m from A.

Backfill material above + under water table: Σ=155 kN/m at 6.5m from A.

The vertical earth force is 94kN/m at 6m from A.

Σ Vertical Forces=838+155+94=1087 kN/m

Loads acting on structure during earthquake are given in Fig-5.7.

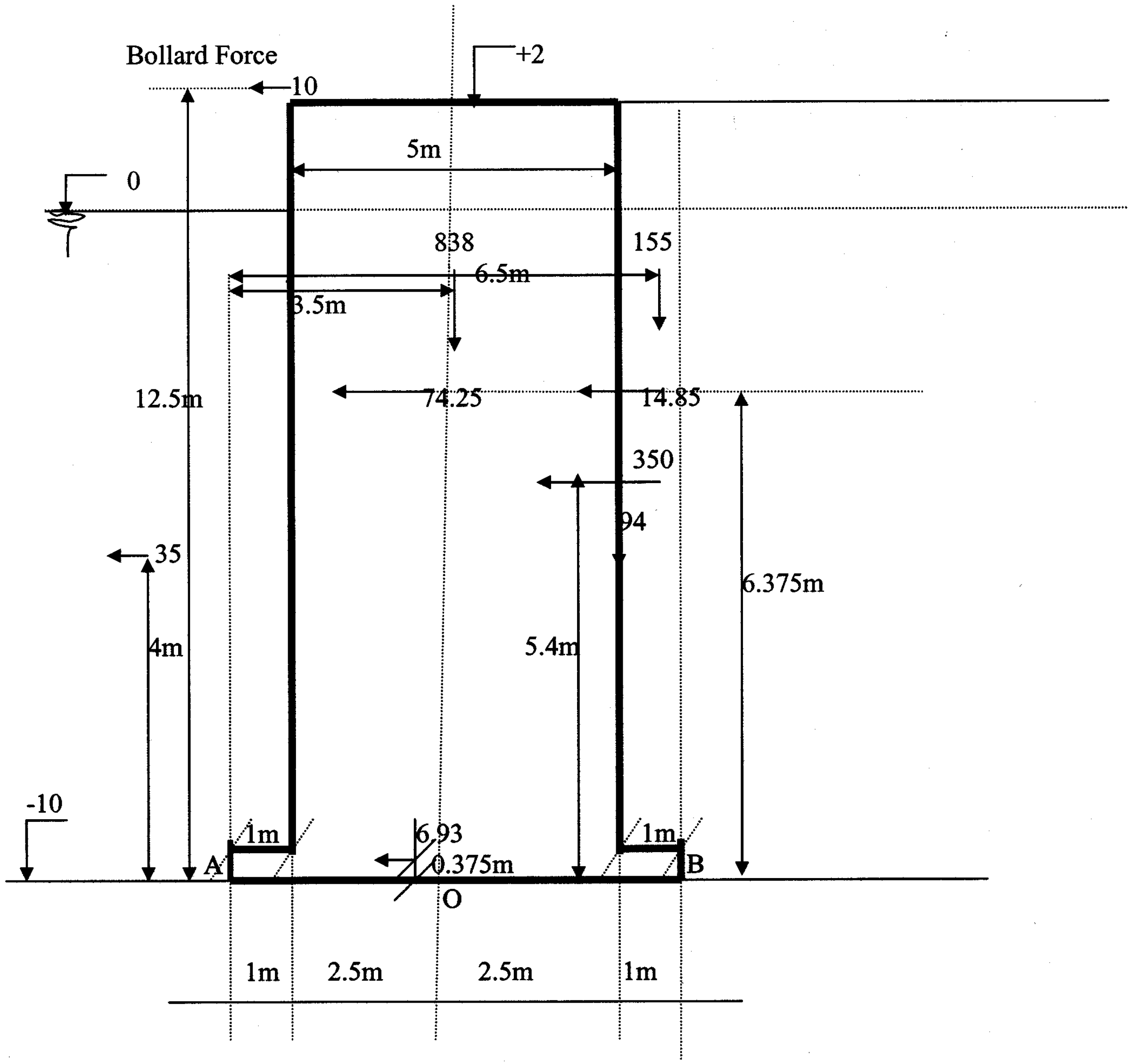


Figure 5.7 Loads acting on structure during earthquake (forces are in kN/m)

The overturning and stabilizing moments with respect to point A.

-The stabilizing moments with respect to point A

Backfill force $\Rightarrow$ (44+111)*6.5=1007.5 kNm/m

Caisson+footing $\Rightarrow$ (220+555+63)*3.5=2933 kNm/m

Vertical earth thrust $\Rightarrow$ 94*6=564 kNm/m

Σ=4504.5 kNm/m

-The overturning moments with respect to point A

Static bollard pull $\Rightarrow$ 10*12.5=125 kNm/m

Caisson inertia $\Rightarrow$ 74.25*6.375=473.3 kNm/m

Footing inertia $\Rightarrow$ 6.93*0.375=2.6 kNm/m

Hydrodynamic force $\Rightarrow$ 35*4=140 kNm/m

Horizontal earth thrust $\Rightarrow$ 350*5.4=1890 kNm/m

Backfill inertia $\Rightarrow$ 14.85*6.375=94.7 kNm/m

Σ=2725.6 kNm/m

The factor of safety against overturning

FSo= stabilizing moments/ overturning moments..(5.10)

FSo=4504.5/2725.6=1.65

The factor of safety against sliding

FSs=ΣVertical force * μ_b/ΣHorizontal force.. (5.11)

FSs= (44+111+220+555+63+94)*μ_b/(10+74.25+6.93+35+350+14.85)

where μ_b is friction coefficient at the bottom of wall (PIANC, 2001), $\mu_b \approx \tan 26.5° = 0.5$, FSs= 1.1

In the case study carried out, the gravity quay wall with cross section given, has factor of safeties for sliding and overturning as FSs= 1.1 and FSo=1.65 respectively at L1 earthquake motion. In this case study sliding is found to be more critical

Pressures at foundation:

Soil bearing capacity is checked by using the pressure distribution at the foundation, where eccentricity (e_{ex}) with respect to centerline of the cross section at point O in Fig 5.7 is given as;

e_{ex} =[2725.6-94*5/2-155*3]/(155+220+555+63+94)=1.86m>7/6=1.17

Therefore the vertical stress at both ends of footing A(seaside) and B (landside) are given as:

σ_A=2*(total vertical load)/(3*a) where ; a=(base width/2)- eccentricity=(W/2)-e_{ex}

=2*1087/3*(7/2-1.17)

=311kN/m^2

add surcharge at half its value

σ_A=311+1/2*30=326kN/m^2

Looking SPT N-value 20 bearing capacity of foundation is sufficient.

5.1.2 Simplified Dynamic Analysis

Simplified dynamic analysis is appropriate for evaluating approximate range of deformations expected under given earthquake motion. The major design parameters and the typical cross section of the gravity quay wall is given in Fig 5.7.

The preliminary analysis includes the use of a simplified method to predict the approximate range of deformations expected. This method utilizes non-dimensional parameters with respect to caisson geometry, thickness of soil deposit below the caisson, and geotechnical conditions represented by SPT N-values of the subsoil below and behind the wall. The displacement at the top of the caisson under the prescribed earthquake motion was estimated.

Input parameters and geotechnical investigation methods for simplified dynamic analysis for gravity quay wall are given in Tables 5.7 - 5.8 respectively.

In these tables the basic parameters are;

- Non-dimensional parameters wrt caisson geometry, thickness of soil deposit below the caisson.

W/H=Width/Height

D_1/H=Thickness of soil below/ Height

- SPT N-values of soil below and behind the caisson.

SPT-N: Obtained from field+lab. investigations of soil.

Table 5.7 Input parameters for simplified dynamic analysis for gravity quay wall (PIANC, 2001)

<table>
<tr><td>Conditions</td><td>Design parameters</td><td>Input parameters</td></tr>
<tr><td>Earthquake</td><td>empirical equations:
a_{max}: peak acceleration
v_{max}: peak velocity
time history analysis:
time histories of earthquake motions</td><td>Bedrock earthquake motions</td></tr>
<tr><td>Structural</td><td rowspan="2">sliding block analysis:
a_t: threshold acceleration
simplified chart:
W,H & water level: Cross-sectional dimensions of a gravity wall
SPT N-values
(extent of soil improvement)</td><td>W, H & water level: Cross-sectional dimensions of a gravity wall</td></tr>
<tr><td>Geotechnical</td><td>G-γ & D-γ curves (for site response analysis
c,ϕ: cohesion and internal friction angle of soils
μ_b,δ: friction angles at the bottom and back face of the wall
ground water level
undrained cyclic properties and/or SPT/CPT data</td></tr>
</table>

Table 5.8 Geotechnical investigation methods for simplified dynamic analysis for gravity quay wall (PIANC,2001)

Type of analysis	Aspects	Analysis method	Input seismic action	Soil parameters	Geotechnical investigation methods						
					Gathering existing data	Field		Laboratory			
						PT	GT	Index Prop.	Static	Cyclic DT	Dynamic LT
SRA		1D total stress analysis	a(t) at bedrock	$V_s(z)$ profile	□		◇	◇			
				G:γ,D:γ curves	□			□		◇	
LIQ	Field based approach	SPT/CPT/V_s liquefaction charts	A_{max} at surface	N/q_c/v_s	□	□	◇	◇			
	Lab. based approach	1D total stress analysis	τ(t) at surface	Cyclic strength curves	□			◇			□
SSI		Newmark	Output of SRA	c, ϕ,μ_B,δ	□	□		◇	◇		◇

Key: SRA=Seismic Response Analysis; LIQ=Liquefaction; SSI=Soil Structure Interaction, PT=Penetration Tests, GT=Geophysical Tests, DT=Pre-failure/Deformation Tests, LT= Failure/Liquefaction Tests

□ used as a standard ◇ use depends on design condition

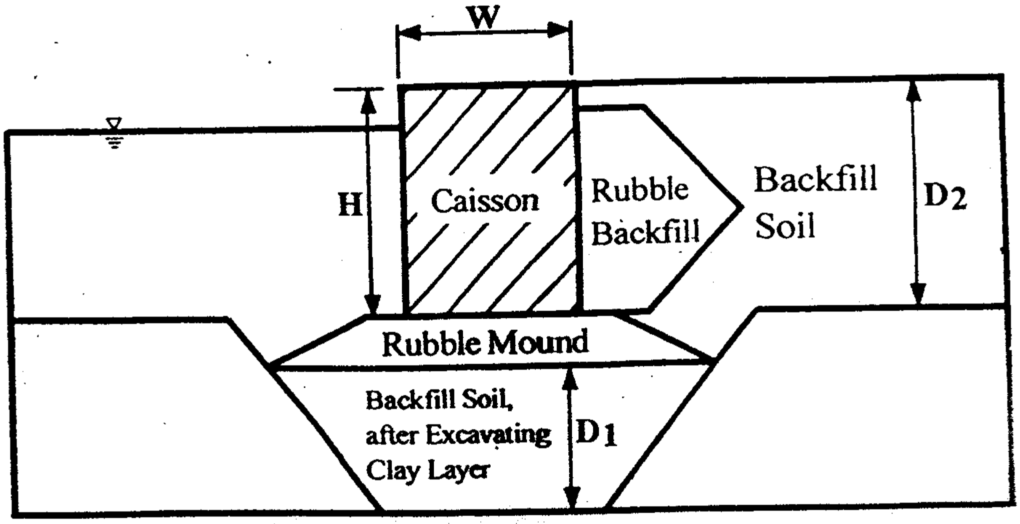


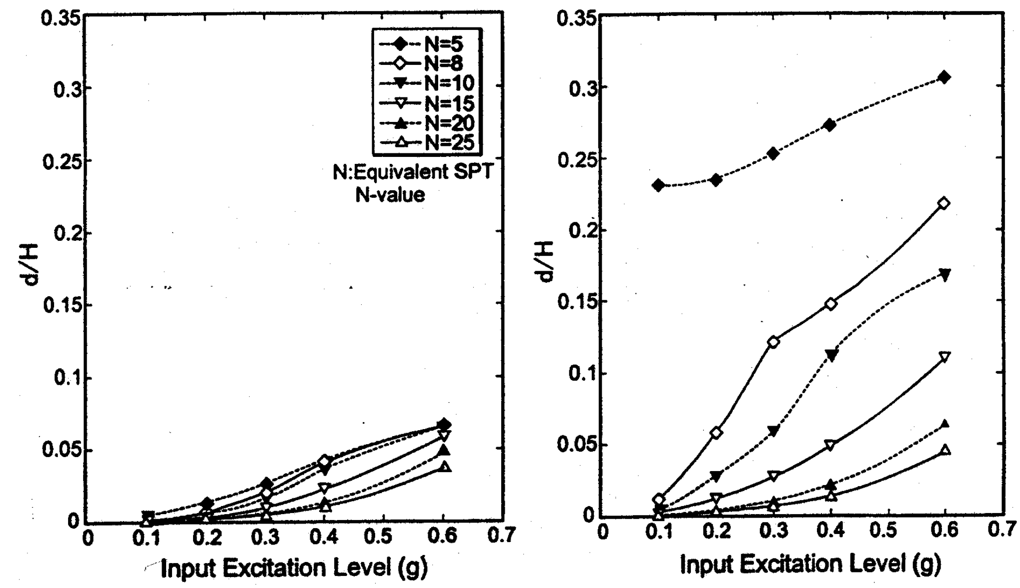
Figure 5.7 Typical Cross Section of a Gravity Quay Wall for Parametric Study (PIANC, 2001)

In Simplified Dynamic Analysis the below given assumptions are used.

-Geotechnical conditions of soil deposit below and behind area assumed to be identical and represented by an equivalent SPT-N_{65} which is corrected for the effective vertical stress of 65 kPa. (PIANC, 2001)

-$D_2 \approx H$ is taken.

The charts for parametric study are given in Figs 5.8, 5.9, 5.10 and 5.11. In these figure displacement of the structure (d) under different input excitation level (g) and different N (equivalent SPTN-value) values are given in terms of normalized displacement (d/H) for certain D_1/H and W/H values.

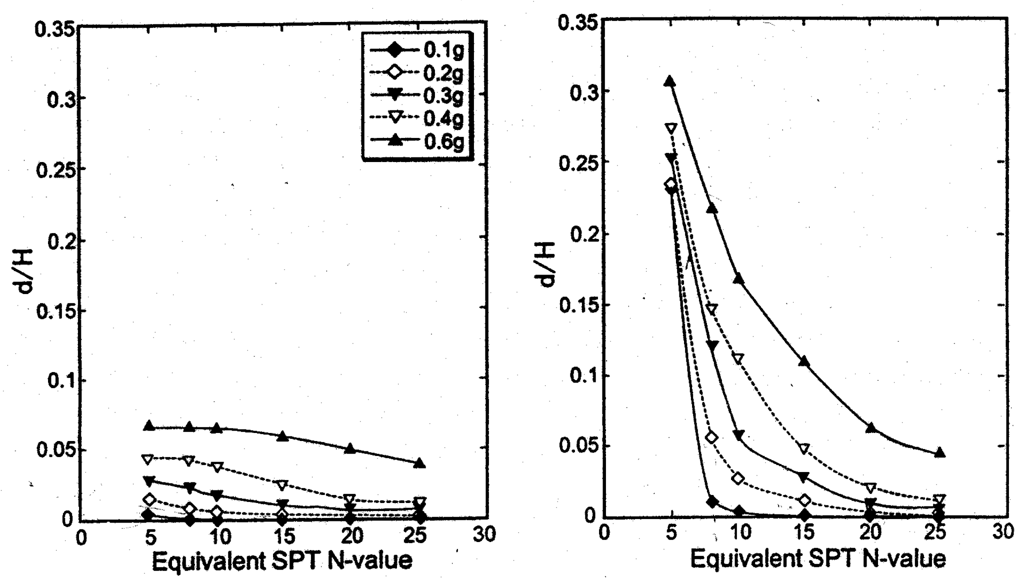


(a) (b)

Figure 5.8 Effects of Input excitation level (for W/H=0,9) (PIANC,2001).

(a) $D_1/H=0.0$.

(b) $D_1/H=1.0$.

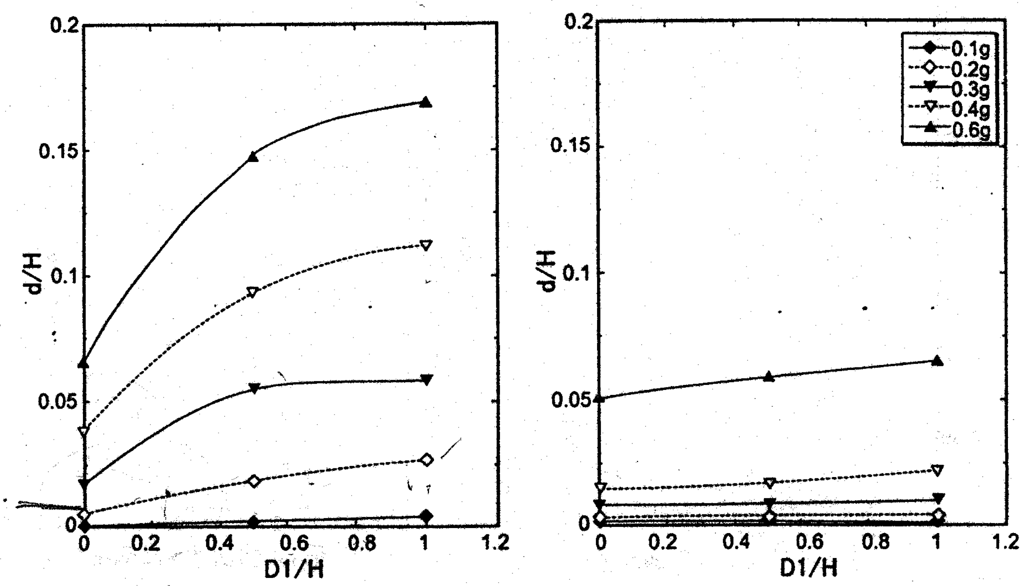


(a) (b)

Figure 5.9.Effects of equivalent SPT N-Value (for W/H=0,9) (PIANC,2001).

(a) $D_1/H=0.0$.

(b) $D_1/H=1.0$.

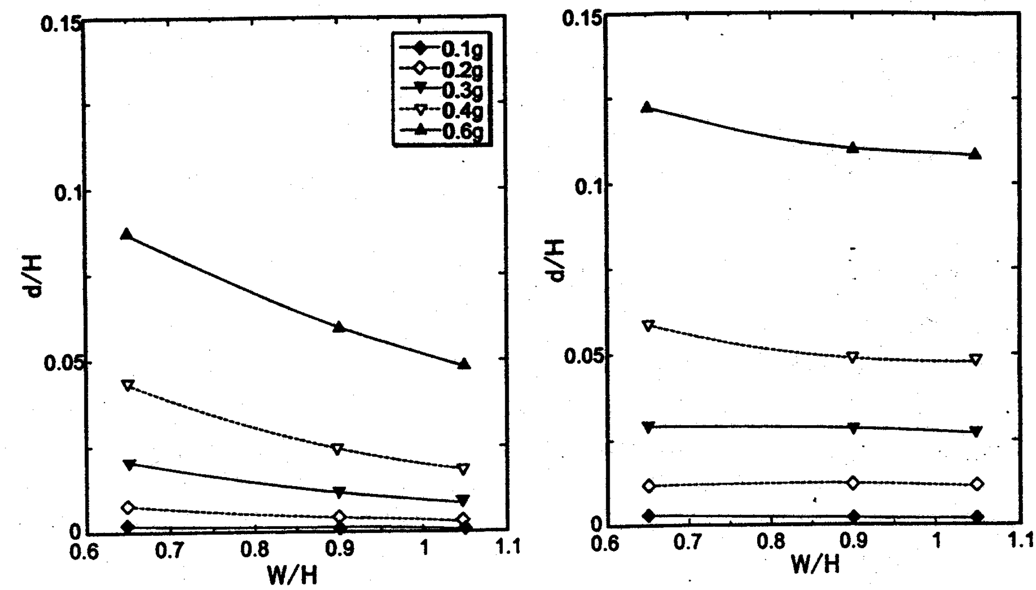


(a) (b)

Figure 5.10 Effects of thickness of soil deposit below the wall (for W/H=0,9) (PIANC, 2001).

(a) Equivalent SPT N-Value (10).

(b) Equivalent SPT N-Value (20).



(a) (b)

Figure 5.11 Effects of width to height ratio W/H (for equivalent SPT N-value of 15) (PIANC, 2001).

(a) $D_1/H=0.0$.

(b) $D_1/H=1.0$.

5.1.2.1 Computations for Simplified Dynamic Analysis

Simplified dynamic analysis is applied to gravity quay wall in case study with the cross section and dimensions as given in Fig 5.1. Simplified dynamic analysis computations are carried out in steps as given below;

Step-1: Normalized dimensions

Since the height (H) and width (W) of the structure are H=12m and W=7m respectively and D_1=12m then; D_1/H=12/12=1 W/H=7/12=0.6

Step-2: Design bedrock acceleration (a_{max}) for L1 and SPTN-value (N_{65})

For L1 design level, for İzmit Bay the normalized displacement d/H is obtained by using Fig 5.8(b) with the below given values:

Design bedrock acceleration a_{max}=0.06g (Çetin et. al,2002), N_{65}=20

D_1/H=1 and W/H =0.9

Computed W/H=0.6 is approximated as 0.9 by using Fig 5.11.(b)

From Fig 5.8(b) d/H is obtained as d/H≅0. Therefore displacement (d) is d=0m

Step-3: Design bedrock acceleration (a_{max}) for L2 and SPTN-value (N_{65})

For L2 design level, for İzmit Bay the normalized displacement d/H is obtained by using Fig 5.8(b) with the below given values:

Design bedrock acceleration a_{max}=0.25g (Çetin et. al,2002), N_{65}=20

D_1/H=1 and W/H =0.9

Computed W/H=0.6 is approximated as 0.9 by using Fig 5.11.(b)

From Fig 5.8(b) d/H is obtained as d/H≅0.01. Therefore displacement (d) is d=0.12m.

From Table 5.3 (a) allowable limits of displacements according to Grade A performance requirements.

Reference level of earthquake motion and corresponding damage levels	Normalized displacement (d/H)	Max. allowable displacement (m.) (d)
L1⇒ Degree I: Serviceable	<0.015	0.18
L2⇒ Degree II: Repairable	0.015-0.05	0.6

From Table 5.3 (b) allowable limits of tilting according to Grade A performance requirements.

Reference level of earthquake motion and corresponding damage levels	Residual tilting towards the sea	Max. allowable tilting (°)
L1⇒ Degree I: Serviceable	<2°	2°
L2⇒ Degree II: Repairable	2-5°	5°

Displacement for L1 d=0m< 0.18m

Displacement for L2 d=0.12m<0.6m

Tilting for L1=arctan d/H=0°<2°

Tilting for L2=arctan d/H=0.6°<5°

Since, both displacement and tilting requirements for L1 and L2 design levels are satisfied, this wall satisfies the required performance criteria.

CHAPTER VI

CONCLUSIONS

The performance-based design is an emerging methodology whose aim is to overcome the limitations present in conventional seismic design and to provide most economical design according to the importance of the structure and the risk of bedrock motion. This work is the first example of an application in Turkey, on this subject. Performance-based methodology and the related parameters are clearly given together with the map of Turkey with earthquake zones in this study and performance-based design has been applied to a case study in İzmit Bay by obtaining ground motion parameters from the existing data (Çetin et. al,2002). In the case study, gravity type quay wall is selected as a design structure. The dimensions are taken from similar structures in İzmit Bay those damaged by the 1999 İzmit earthquake (Yüksel et.al 2003). This design example will illustrate only the application of the simplified and simplified dynamic analysis procedures for preliminary design at low level of excitations. In the application, grade A is selected as performance grade. Therefore reference levels of earthquake motions and corresponding acceptable level of damages are taken as,

(L1) $\Rightarrow$ Degree I: Serviceable

(L2) $\Rightarrow$ Degree II: Repairable

Lifetime (T_L) of the structure is taken as; T_L=50 years.

Design earthquake motions at bedrock are given for İzmit Bay region as PGA (Peak Ground Acceleration):

For L1 with %50 exceedance (frequent) a_{max}=0.06g.

For L2 with %10 exceedance (rare) a_{max} =0.25g (Çetin et.al 2002).

For simplified analysis, the gravity quay wall with cross section given in Fig 5.1, has factor of safeties for sliding and overturning as FSs=1.1 and FSo=1.65 respectively at L1 earthquake motion. In this case study sliding is found to be more critical.

For simplified dynamic analysis, the gravity quay wall satisfied the required performance criteria for L1 design level with displacement d=0m and tilting 0° and for L2 design level with displacement d=0.12m and tilting 0.6°.

Performance-based methodology is not only applicable in the design of new structures but also applicable for remediation studies on existing structures to mitigate hazards and losses due to earthquakes. In the performance-based method applications, geotechnical investigations and the design earthquake motions are the most important design procedures. In general the rubble backfill soil characteristics plays a very important role especially in increasing the factor of safety against sliding since the friction angle between the structure and rubble backfill depends on the soil characteristics. Similarly, with increased SPT-N values, which effectively depends on compaction characteristics, horizontal displacement of the structure decreases and the bearing capacity of the foundation increases.

For future studies, earthquake zones of Turkey should be clearly identified since the fundamental input to performance-based method requires %10 and %50 exceedance probabilities for earthquake motion.

Tsunami effects should be included in the analysis. Liquefaction studies have to be made for remedial measures.

Finally, as a recommendation, the performance-based method computations should be carried out not only for simplified and simplified dynamic analysis but also for dynamic analysis.

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