CHAPTER 2 COMPONENT DESIGN AND EVALUATION CONSIDERATIONS

Introduction

Marine oil terminal components are quite broad and varied, and include a range of earthen embankments and berthing structures. The earthen embankments may be plain, armored with rock rip rap or other materials, and may possibly be topped with a concrete structure. Berthing structures at ports may be massive concrete block gravity structures, steel sheet-pile retained earth structures, pile supported marginal wharves, pile-supported piers, or combinations of these.

This chapter provides guidelines to assist the engineer in addressing earthquake engineering aspects of these components. It is organized into subsections which cover the most common waterfront components at ports. This material provides seismic guidelines for specific types of port waterfront components – examples include embankments, piles (which are a common element for many types of waterfront and other types of port components), marginal wharves, gravity retaining structures, and steel sheet-pile wharves. For each of these components, these sections summarize general functional/operational requirements, guidelines for establishing seismic performance requirements, performing preliminary seismic evaluations, and for seismic analysis, seismic design of new components, and seismic retrofit of existing components.

General Seismic Performance Issues for Waterfront Structures

As outlined in the seismic criteria a two-level design approach for port structures has been widely adopted. The Level 1 design considers a moderate level of ground shaking that is likely to occur during the life of the structure (also often termed the Operating Level Earthquake (OLE) ground motions). Under this level of ground shaking, the structure is designed so that its operations are not interrupted and any damage that occurs will be readily repairable within a relatively short time. The Level 2 design considers much stronger motions that are less likely to be exceeded during the life of the structure (commonly called the Contingency Level Earthquake (CLE) ground motions). Under these motions, the structure is designed so that any damage that occurs is controlled and repairable (although possibly over an extended time). In this, the Level 1 design criteria address economic issues associated with a loss of operations at the port and major repair costs, and the Level 2 design criteria address the same issues, and the additional considerations of life safety, structure repairability, environment protection, and collapse avoidance.

Recent guidelines indicate that typical building codes are usually not appropriate for port waterfront components, and recommends that such components be designed in accordance with the specific seismic performance requirements for the component as well as its physical attributes (Werner, 1998). By including other design references, it is recognized that this may develop a dual design criteria because of local building code authority jurisdiction over some waterfront structures. These seismic performance requirements (and associated design criteria) should not only address life safety, but should also consider the importance of the component to overall port operations as well as any special requirements of the component (such as special pollution control requirements), etc. Likewise a reassessment may be required after the occurrence of an earthquake.

The development of more stringent seismic performance requirements and design criteria for waterfront components is on an upswing in the United States. This appears to be due to an increased awareness of the lessons learned from past earthquakes regarding the extent and consequences of inadequately designed waterfront components, and measures that can be implemented to improve the seismic performance of these components.

Embankments

Embankment Types

Earth embankments are commonly the most prevalent waterfront components in ports due to their widespread use as perimeter containment dikes during initial reclamation operations and as breakwaters which protect the inner port from wave action and current-induced scour. This section identifies the most common embankment types and focuses on the seismic performance criteria that are common to each.

Native Soils Natural soil deposits that form banks such as spits or levees can be loosely categorized as embankments, since these natural barriers can provide protection for harbors or river front ports, and these soils are commonly incorporated into engineered earth structures. It is common for engineered fills to be placed on existing rises of native soil in order to minimize the volume of soil required during construction of earth embankments. The heterogeneous and usually weak nature of these native deposits can result in embankments that are marginally stable during an earthquake, and are also prone to loss of strength due to groundwater seepage conditions during extreme high tides at coastal ports or during flood stages along inland waterways. The seismic performance of embankments made of, or on, these deposits has generally been very poor.

Rock and Sand Dike with Backland Fills Rock and sand embankments have been used extensively as perimeter dikes during the construction of offshore reclamation projects. The costs associated with the excavation and transport of the materials will usually determine the relative volumes of sand or rock used in construction of the embankment. In many regions, the inherent benefits of using rock fill in construction are overshadowed by the relatively high cost of transporting the material from distant quarries. A multitude of different embankment configurations have been employed at ports to optimize the use of soil and rock fill. Examples of these embankment types include single lift sand dikes covered with rock armor for wave protection, single lift rock fill dikes, multiple lift rock fill dikes and hybrid dike configurations (Figure 2-1).

The addition of soil fill behind the dikes creates the backland areas of the port. This fill can be placed either concurrently with the construction of the dikes or after construction of the entire dike has been completed. It is quite common for hydraulic placement methods to be used for the fill behind the dikes. In several instances (e.g., at the Port of Osaka, Japan) fine-grained, bay floor sediments have been used as backland fill. This practice has led to long-term settlement problems associated with consolidation of the fill soil. In most cases however, sandy soils are used as backland fill. When placed through slurry pipes or end-dumped through standing water from barges, these sandy soils are very loose and prone to earthquake-induced liquefaction.

Breakwaters Various types of breakwaters used for offshore wave protection at ports are shown in Figure 2-2. The most common types are rubble-mound sloping-type breakwaters (Figure 2-2a), composite-type breakwaters (Figure 2-2b), and, to a lesser degree, specialized breakwaters such as curtain walls, sheet-pile breakwaters, and floating breakwaters. This discussion focuses on rubble-mound and composite-type breakwaters due to their common usage and similar foundation requirements.

Rubble-mound breakwaters, Figure 2-2a, are constructed in much the same manner as sand- and rock-dikes. Additional rock layers are placed on the breakwaters in order to provide protection from the combined action of direct wave impact and littoral currents. These layers are often augmented with shape-designed concrete blocks for the dissipation of wave energy. Composite-type breakwaters, Figure 2-2b, differ from rubble-mound breakwaters in that a soil and rock berm serves as the foundation for a gravity wall (usually a concrete caisson) which acts as the breakwater. The foundation requirements for these two types of breakwaters are similar, although the rubble-mound-type are commonly wider at the base due to the slope angles used in construction.

Bulkheads and Sea Walls Bulkheads and sea walls are onshore structures that serve as both earth retaining systems and wave protection structures. These waterfront structures include gravity walls, cellular sheet-pile bulkheads, anchored sheet-pile walls, and composite concrete faced walls. Seismic guidelines for these structures are provided in subsequent sections.

Bulkheads are usually vertical in section to facilitate berthing for ships, while sea walls are commonly tiered or sculpted to optimize the dissipation or redirection of wave energy. The techniques used for backfilling these structures are equivalent to the hydraulic methods used for reclamation behind dikes. For this reason, the seismic performance of these structures has been similar to that of the other embankment types.

General Functional/Operational Requirements at Ports

As categorized in this subsection, embankments are earth structures, or composite structures that function as either earth-retaining systems, shore-protection components, or both. Functional/operational requirements of the various embankment types are summarized below.

Sand and Rock Dikes with Backland Fills. These dikes are used as perimeter-retaining structures around reclaimed land such as islands and marginal wharves, and also as foundation pads for gravity structures such as caissons and embankments for ground transportation systems. They form the interface between the marine and backland portions of the port. Although these structures are used for earth retention, they are quite distinct from other structural earth-retaining systems which are discussed in subsequent on gravity-retaining structures and steel sheet-pile retaining structures.

Breakwaters. Breakwaters protect the harbor and shore areas from the waves and currents generated at sea. This provides for calm water on the leeward side of the breakwater within the harbor and reduces navigation hazards. In addition, breakwaters can be used to mitigate the migration of sediments into the harbor. Breakwaters are most commonly gravity structures and they can be isolated offshore or connected to land. Both the offshore and land-connected breakwaters are sometimes used as docks.

Sea Walls and Bulkheads. Sea walls are also used as wave protection, although these walls are located along the shoreline and protect the shore from erosion due to wave action and littoral currents. Bulkheads are waterfront retaining walls that include gravity-type quay walls and sheet-pile structures. These structures form the marginal wharves and piers along which berthing and cargo handling operations take place. Their primary function is to maintain adequate freeboard to preclude overtopping by waves. This function is impaired if the structure settles or topples during an earthquake. These potential failure modes are distinct in that settlement is due to densification or deformations of the foundations soils, whereas toppling or sliding may be due to inadequate dimensioning of the embankment.

As the primary waterfront components at most ports, earth retention embankments (dikes and bulkheads) often provide foundation support for pile-supported structures, utility lines, and cargo handling components , for loading and unloading of ships, such as cranes, ramps, and conveyance systems. Embankment failures are manifested as excessive lateral and vertical deformations. These deformations will, in turn, result in damage to the port components located near the waterfront and the disruption of port operations. Given the importance of these components to port operations, the primary requirement of the embankments is that ground deformations be minimized. These include deformations of the foundation soils as well as the embankments themselves. Localized failures such as slumping of the face of the embankment or sliding of armor layers could affect embedded piles or expose the earth structure to wave induced scour. The latter effect would be relatively easy to remedy and would be considered an acceptable consequence of a design-level earthquake in most cases.

Guidelines for Developing Seismic Performance Requirements

The weak cohesive soils and potentially liquefiable sandy soils that are common throughout the marine environment are primary factors in most embankment failures.

This observation has been made for failures due to static and dynamic loading conditions. The seismic performance requirements for embankments should reflect the sensitivity of adjacent port components to ground deformations. For example, acceptable deformation limits for sand and rock dikes will vary, depending on whether the earth structure: (a) is placed as a perimeter dike adjacent to a storage yard or a relatively undeveloped portion of the waterfront; or (b) is incorporated in the development of a sensitive structure such as a pile-supported wharf. In the case of piles, pipelines, or utility lines embedded in the embankment, the allowable deformation of these components will dictate the ground movements that can be tolerated. Post-earthquake serviceability requirements of the waterfront components that are founded on or near the embankments should guide the specification of seismic performance requirements of the embankments.

In addition to assessing the impact of embankment deformations on components in immediate contact with the retaining structure, the influence of the associated ground movements in the backland soils should also be considered. In several ports, efforts have been made to ensure that embankment deformations will remain small, although such efforts have not been made during the design of the cargo storage areas behind the embankments. Here, the soils are allowed to remain unimproved and potentially vulnerable to liquefaction. This is because any liquefaction and associated differential settlement and pavement damage that occurs in these backland areas would not suspend port operations, and regrading could be carried out quickly and relatively inexpensively.

The design of gravity breakwaters includes the bearing capacity of the foundation soils, settlement of the foundation soils due to consolidation, and stability due to wave loading. Experience at numerous ports around the world has demonstrated that the primary modes of failure due to seismic loading are foundation failures and excessive settlement. When this occurs, composite breakwaters may retain their vertical orientation, yet become submerged due to densification and deformations of the foundation soils. Given that the freeboard of a breakwater is the key issue, the performance requirements should focus on the potential for deformations of the foundation soils.

Guidelines for Preliminary Seismic Vulnerability Assessment

Preliminary seismic vulnerability assessment of existing embankments should be based on visual observation in the field and review of relevant office documents. Visual observation of embankments, dikes, and bulkheads is difficult because of their limited accessibility (i.e., they are buried or are commonly covered by soil layers or pavements). Breakwaters pose additional difficulties, since their offshore location limits direct viewing at the dredge line. Nevertheless, visual observation is an important basis for assessing the integrity of these embankments. Visual evidence of foundation degradation, excessive settlements, etc. often indicates conditions that decrease the seismic stability of embankments. In particular, the engineer performing the visual observation should be aware of the following factors that could indicate a potential for poor seismic performance of embankments:

Any observed undermining of foundation soils around breakwaters or bulkheads due to scour represents a possible location of large deformations during an earthquake. Inspection methods for scour include surveys by divers, or profiling techniques such side-scan sonar.

Slumping of earth embankments due to washing of soils from behind armor layers can compromise the surficial layers of dikes and affect adjacent structures. Ground cracks caused by embankment settlements are evidence of weak, compressible foundation soils. Recurring tension cracks in backland areas indicate marginal static stability and significant vulnerability to earthquake-induced deformations. This global type of movement can also be indicated by deformations of the piles beneath wharf decks that are embedded in the embankment, and the misalignment of crane rails.

The office documents reviewed during a preliminary seismic vulnerability assessment of an embankment should include geotechnical reports, construction documents, as-built records, maintenance reports, etc. This review should focus on the seismic design provisions adopted, if any, and the construction methods used to place backland fill soil. Evaluations of the potential for liquefaction of the foundation soils and backfill, as well as global stability of the embankment should be emphasized. Maintenance reports for waterfront areas can provide evidence of in-stability of embankments, and may help to prioritize areas of the port for seismic retrofit.

A notable example of the possible benefits of preliminary pre-earthquake seismic vulnerability evaluation of embankments is the Port of Valdez, Alaska. In the two-to-three decades preceding the 1964 Alaska earthquake, pile-supported wharves embedded in gravel fill dikes suffered several failures under static loading conditions. Some of the failures were induced by cargo loads transmitted to weak foundation soils, while others were due to unstable slopes which may have failed in response to uncommonly great tidal fluctuations. These factors may have contributed to the catastrophic flow slide that occurred at the Port of Valdez during the 1964 Alaska Earthquake , which included almost 1200 m of shoreline at the port and claimed 30 lives. While this is an extreme example of what can happen, the potential for poor seismic performance that may be indicated by such pre-earthquake observations should be kept in mind during preliminary seismic vulnerability evaluations of port waterfront embankments.

Guidelines for Seismic Analysis

A seismic analysis of a port waterfront embankment should focus on two issues; (a) the stability of the embankment itself, and (b) the global stability of the embankment, backfill, and foundation soils. In most cases, common pseudostatic rigid body methods of evaluation will suffice for evaluating the stability of the embankment. These methods of evaluation are well established in the technical literature (e.g., Ebeling and Morrison, 1993; Kramer, 1996). However, although pseudostatic methods are useful for approximate analysis of seismic stability, they suffer from the following limitations: (a) they do not indicate the range of embankment deformations that may be associated with various factors of safety; (b) the influence of excess pore pressure generation on the strength of the soils can only be approximated; and (c) coupled analyses that account for such factors as the degradation of soil strength and soil-structure interaction are not possible. Therefore, for those embankments where damage could lead to unacceptable risks to port operations, more refined analysis procedures that are summarized below should be used.

Enhancements to traditional pseudostatic limit equilibrium methods for estimating embankment deformations and the degradation of soil strength due to liquefaction or collapsible soil behavior have been proposed by numerous investigators (e.g., Makdisi and Seed, 1978; Byrne et. al., 1994). These methods are based largely on rigid sliding-block methods, wherein a portion of the embankment slides in response to ground motions that exceed a critical acceleration.

In situations involving pile supported structures embedded in dikes, or other embankment and structure deployment where soil-structure interaction effects could be significant, it is becoming more common to rely on numerical modeling methods to ascertain the likely range of embankment deformations during design-level earthquakes. Two-dimensional numerical models such as FLUSH (Lysmer et. al., 1975), FLAC (Itasca, 1995), and DYSAC have been used to model the seismic performance of waterfront components at ports (e.g., Werner and Hung, 1982; Roth and Inel, 1992; Dickenson and McCullough, 1998). These numerical analyses models differ primarily in the soil models employed and in their ability to model permanent deformations. Each has been useful in evaluating various aspects of dynamic soil-structure interaction.

Guidelines for Seismic Design of New Embankments

The seismic design of new embankments structures should address: (a) static stability issues (e.g., bearing capacity, sliding, forces due to wave loading, and (b) dynamic loading considerations that include the influence of inertial body forces on overall stability, the dynamic behavior of embankment and foundation soils, and soil-structure interaction effects. Also, the heights of breakwaters must be specified to account for consolidation settlements that may occur. As previously discussed, these methods of analysis include standard pseudostatic rigid-body analyses, non-coupled analyses (which account for the loss of soil strength and stiffness as well as permanent deformations), and advanced numerical modeling techniques. The allowable deformations of the embankments and adjacent soils will reflect the importance of the components along the waterfront, which may vary at specific sites within individual ports.

During the seismic design process, the presence of any potentially liquefiable materials in backfill areas must be fully analyzed and expected settlements computed. Specific attention should be paid to the acceptability of the amount of settlements which can be tolerated, which will depend on the type and importance of the port operations in the vicinity Under a Level 1 seismic design, large deformations resulting in widespread pavement disruption should be avoided where economically feasible. In a Level 2 design, larger deformations of the embankment may be permitted, as long as the duration and costs of disruptions to the surrounding area are within acceptable limits and consistent with performance goals.

For Level 1 seismic design, the Factor of Safety against liquefaction in the backfill should be 1.5 or higher with settlements of about 1 inch or less and lateral deformations of about 3 inches or less.) For the Level 2 design, the Factor of Safety against liquefaction in the backfill should ***ideally*** be 1.0 or higher with settlements of about 4 inches or less and lateral deformations of about 6 to 12 inches or less. Where it may not be possible to achieve a Factor of Safety greater than 1.0, a Factor of Safety greater than 0.9 may be considered as long as the computed deformation state is shown to have limited controlled settlements and lateral spread equivalent to the values stated.

If potentially unstable foundation soils are identified during the design phase of development, remedial strategies such as soil replacement (key trenches with engineered fill) or soil improvement may be used. Ground treatment can be carried out concurrently with reclamation and construction of the embankments, resulting in an expedient construction sequence.

Guidelines for Seismic Retrofit of Existing Components

Experience has demonstrated that, even at modern ports, embankments have been susceptible to earthquake-induced damage. Soil liquefaction and insufficient stability of the underlying foundation soils repeatedly appear as the predominant causes of earthquake-induced failures of existing embankments and associated damage to waterfront components. Seismic retrofit of embankments may include remedial measures to the embankment, to the foundation soils, or both depending on the results of seismic stability analyses. In the case of a marginally stable embankment founded on competent soils, remedial measures may include one or more of the following:

Modifying the geometry of the embankment, either with berms or by reducing slope angles.

Improving the strength of the soils by using mechanical densification, soil replacement, or cementation techniques.

Strengthening the embankment through the use of structural stabilization techniques such as mixed in-place soil-cement walls, drilled piers, and driven displacement piles adjacent to the toe of the embankment. The latter method should be used with caution, since construction-induced vibrations can lead to excessive deformations of marginally stable embankments.

In many instances the foundation soils beneath the embankment are unstable under seismic loading. Soil improvement techniques can be used to mitigate these hazards. From a practical perspective, guidelines for specifying the volume of soil to be treated and the degree of improvement required to insure that earthquake-induced embankment deformations are held to within allowable limits have not been well developed. Recommendations have been provided (e.g., PHRI, 1997) but very few case histories exist for evaluating the performance of embankments that have undergone soil treatment. The limits of soil treatment are usually determined by performing a series of sensitivity analyses wherein the width of the improved soil zone is related to either the computed factor of safety against sliding or the estimated deformations. The requisite volume of soil improvement will reflect site specific factors such as the embankment type and geometry, depth of weak soils, density of the backland soils adjacent to the structure, as well as the characteristics of the design level ground motions.

The constructability of these remedial strategies is complicated by the location of preexisting port components such as piles, pipelines, above- and below-ground utility conduits and overhead structures. In addition to limiting access and constraining the locations of work platforms, existing buried components can be adversely affected by several of the ground treatment methods. Soil densification techniques that rely on vibration (e.g., vibro-compaction, deep dynamic compaction or soil displacement (compaction grouting) increase lateral earth pressures. The potential for this increased pressure should be acknowledged when improving soils in close proximity to buried structures.

It should be noted that there may be a need for a seismic reassessment following an earthquake. This reassessment should be triggered when excessive deformations are observed. The decision process depends on the specifics of the geometry and soils present. Typical soil limit deformations are given in following sections.

Gravity Retaining Structures

Types of Gravity Retaining Structures

Gravity earth-retaining structures are widely used along the waterfront for quay walls, sea walls, and lock and dam structures Numerous wall types and wall geometries have been employed at ports (Figure 2-3). A broad categorization of the most common retaining structures is provided below.

Concrete Block Walls These structures are composed of smooth or interlocking blocks that are stacked one on top of another to the design height. Pile foundations are used in regions with weak foundation soils or other areas with a limited supply of suitable fill for key trenches. These walls can be either vertically faced or stepped to slope at specified angles. The primary advantages of these walls include: (a) durability to environmental agents and impact by vessels; (b) good quality control achieved during fabrication; (c) simple construction; and (d) adaptability to a variety of foundation conditions.

The basic design of block-work walls can be generally classified as follows: (a) bonded construction using solid concrete blocks; and (b) walls formed with hollow or special concrete blocks (Tsinker, 1997). The type of bonding between the blocks will influence the seismic performance of the walls, since sliding may tend to occur at the interface between adjacent blocks during shaking.

Concrete Caissons The two most predominant types of concrete caissons used at ports are "box-type" cassions and counterfort caisson walls. These structures are built onshore, transported to the waterfront and sunk into position. Box-type caissons can be floated into place. Pile foundations are often used in weak soils when suitable replacement soils are not readily available. In the case of box-type caissons, internal walls in the caisson form cells that are filled with granular material (e.g., soil, slag, concrete construction debris) or water, depending on the lateral earth pressures that must be resisted and the allowable bearing pressures on foundation soils. The caissons are usually placed on a prepared foundation pad of granular fill and backfilled with sand or rubble. The density of the foundation and backfill soils will have a significant effect on the seismic performance of the caisson.

Cellular Sheet Pile Structures Cellular steel sheet pile bulkheads are usually constructed from flat web sheet piles that are driven with vibratory equipment. The shape of the cell is maintained during the construction process with the use a of template for guiding the sheet piles during placement. Arc sections are driven on one or both sides of the bulkhead, and granular soils are used to fill the cells. The fill soil is often densified to increase the lateral stability of the cellular bulkhead. This is advantageous for the seismic performance of the bulkhead and the densification also reduces the liquefaction susceptibility of the fill.

These earth retaining structures differ from the others listed in this section in that they are flexible. This flexibility provides some reduction in the dynamic earth pressures experienced during earthquakes, although progressive permanent deformations can occur. Excessive deformations can lead to high interlock tension and potential failure. Liquefaction of the interior fill will also result in excessive interlock tension. The failure of a 45-year-old sheet pile cellular bulkhead occurred due to liquefaction generated by ground motions of moderate intensity (PGA approx. 0.15 g to 0.20 g). Post-earthquake investigations of this facility revealed that corrosion of the sheet piles and improper placement of the sheet piles had contributed to the failure. Standard methods of analysis, including pseudostatic seismic design, are presented by Schroeder (1990).

Steel Plate Cylindrical Caisson. Large diameter cylindrical caissons have been used in numerous ports. The primary advantages of this caisson type over comparable sheet pile structures are: (a) the cell is fabricated onshore; and (b) placement is much faster than with cellular sheetpile bulkheads. However, the cylindrical caisson does require the preparation of a reasonably flat bedding pad. The steel cell is placed by sinking it into place and embedding it into the soil through driving with vibratory hammers. In most cases, steel arc sections are then placed on both sides of the wall and joined to the cylindrical caisson by interlocks.

Cribwork Quay Walls Crib walls at ports have been constructed from timber cribwork and concrete cribwork. Cribs are rather labor intensive, and they must be constructed onshore and launched and sunk into position. They are then filled with gravel or rock fill to form a gravity structure. The crib wall can either be full height or provide a supporting base for mass concrete superstructure walls which are placed on the cribwork (Tsinker, 1997).

General Functional/Operational Requirements at Ports

The primary operational requirement of gravity retaining structures is to resist lateral earth pressures with minimal deformation. These structures resist the lateral earth pressures by virtue of their body weight and the resulting frictional resistance mobilized between the structure and the foundation soil. These massive structures require strong foundation soils and it has often been necessary to enhance the bearing capacity of the foundation by excavating trenches and replacing the weak soils with cohesionless fill, or by supporting the structures on piles or pile supported relieving platforms. Specifications for allowable wall-backfill deformations will be based on the sensitivity of the structures located in close proximity to the retaining structures.

Guidelines for Developing Seismic Performance Requirements

Where gravity retaining structures are deployed alongside key waterfront cargo handling operations, the seismic stability of such structures is a major concern. Permanent deformations of the retaining structures and surrounding soils must be minimized to ensure serviceability following design-level earthquake motions. Therefore, it follows that the seismic performance requirements for gravity should focus establishment of allowable wall deformations under the design-level earthquake motions such that: (a) the operations of key components that are supported on the retaining structures will not be adversely affected; and (b) associated ground movements in the backland soils will lead to acceptable levels of damage to the structures and cargo storage facilities in those areas.

In general, waterfront retaining walls should perform to the following standards;

1. To resist earthquakes of moderate size, Level 1, which can be expected to occur one or more times during the life of the structure without significant damage (i.e. displacement). As a general guideline, the deformations associated with this performance requirement are roughly 1 inch or less of settlement and lateral deformations of about 3 inches or less.
2. To resist major earthquakes, Level 2, which are considered infrequent rare events maintaining life safety and precluding total collapse, but allowing a measure of controlled inelastic behavior which will require repair. The allowable deformations for Level 2 earthquakes are a maximum of 4 inches of settlement and lateral deformations of about 6 to 12 inches or less.

In order to ensure that the approximate deformation limits are not exceeded, liquefaction hazards must be fully evaluated. Specific attention is to be paid to the acceptability of the amount of settlements. Under Level 1 earthquake motions, large deformations resulting in widespread pavement disruption should be avoided where economically feasible. At several ports, liquefaction mitigation efforts have focused on limiting earthquake-induced wall deformations and applying only nominal soil improvement in backland areas such as cargo storage yards located well behind the walls. Although liquefaction in these areas would result in differential settlements and damage to pavements, these effects would not suspend port operations and regrading could be carried out quickly.

Guidelines for Preliminary Seismic Vulnerability Assessment

Preliminary assessment of the seismic vulnerability of existing gravity retaining structures should be based on field inspections and a review of design and construction documents. Field inspections of retaining walls are onerous due to the development of the waterfront around the structure. The waterfront location of the walls and their partial burial with backfill will conceal them from direct view. In addition, retaining walls are commonly covered with pavements and structures. However, inspections can provide evidence for of existing conditions that could lead to poor seismic performance during future earthquakes such as degradation of the structure, evidence of ground movement under static conditions, and other adverse conditions such as excessive scour beneath the gravity wall. This is especially true for steel sheet pile bulkheads which are prone to corrosion. Evidence of foundation degradation, excessive settlements, etc. is often indicative of conditions that decrease the seismic performance of retaining structures. The following is a partial list of potential factors that could adversely affect the seismic performance of waterfront gravity walls, the following have been observed at ports:

> Undermining of foundation soils around quay walls due to scour. Inspection methods for scour include surveys by divers or profiling techniques such side-scan sonar.

Slow, yet continuous deep-seated rotation due to the marginal bearing capacity of foundation soils. This global type movement can also be indicated by rotation of the gravity walls, deformations of relieving platforms, persistent tension cracks in backland pavements, and the misalignment of crane rails or utility lines that are supported on the gravity wall. These observations provide evidence low static stability and significant vulnerability to earthquake-induced deformations.

An important aspect of the vulnerability assessment should also include a thorough review of office documents (e.g., geotechnical reports, construction documents, as-built records, and maintenance reports). This review should focus on the seismic design provisions adopted, if any, and the construction methods used to place backland fill soil. Evaluations of the liquefaction susceptibility of foundation soils and backfill, as well as global stability of the retaining wall should be emphasized. Maintenance reports for waterfront areas can provide evidence for marginal stability of retaining walls and may help to prioritize areas of the port for retrofit strategies.

Guidelines for Seismic Analysis

Seismic analyses for new and existing retaining structures must focus on the dynamic behavior of the foundation and backfill soils, as well as the overall stability of the walls. Potential failure modes include: sliding, overturning (for rigid walls only), bearing capacity failure, and deep seated instability.

The most commonly used seismic design methods for gravity structures are based on standard pseudostatic limit equilibrium methods of analysis wherein a static horizontal seismic coefficient is applied as an additional body force (Ebeling and Morrison, 1993). The pseudostatic method of analysis suffers from two significant deficiencies when applied to waterfront retaining structures: (a) the loss of soil strength associated with the generation of excess pore pressures during earthquakes can only be approximately accounted for using post-liquefaction residual undrained strengths for the sandy soils; and (b) the deformations of the wall and adjacent soil can not be evaluated. The limitations imposed by these design methods can be significant in light of the role that liquefaction plays in the seismic performance of waterfront retaining structures.

As a means of estimating earthquake-induced deformations of gravity walls, limit equilibrium analyses can be supplemented with rigid body, sliding block-type displacement analyses which are used to estimate the seismically induced movement of retaining walls acceleration. This method of analysis is similar to the procedures for analysis of earthquake-induced deformations of slopes previously summarized. In this method, the lateral acceleration that yields a factor of safety against sliding equal to unity is defined as the critical (or yield) acceleration. A suite of appropriate acceleration time histories is then used in conjunction with the critical acceleration and the permanent displacements calculated. This technique has been used as the basis for several common methods that have been developed for the estimation of gravity wall displacements (e.g., Richards and Elms, 1979; Elms and Richards, 1990; Whitman and Liao, 1985).

The allowable deformations of the retaining structure should reflect the impact that the deformations have on the stability of the wall, and as well as the sensitivity of nearby waterfront components to lateral and vertical deformations of the retaining structure. Key considerations pertinent to the specification of allowable deformations may include: (a) whether the crane rails are tied together; (b) whether utility conduits are rigidly fastened to or pass between construction joint in the retaining structures; (c) whether pile supported structures are connected to the retaining structure and, if so, the ductility of these connections.

In projects involving displacement-sensitive retaining walls, advanced numerical modeling techniques are recommended for estimating permanent displacements due to earthquakes. The primary advantages of these models include: (a) complex wall geometries can be evaluated; (b) sensitivity studies can be readily performed to estimate the influence of various parameters on the seismic stability of the retaining structure; (c) dynamic soil behavior is much more realistically reproduced; (d) coupled analyses can be used that account for such factors as excess pore pressure generation in contractive soils during ground shaking and the associated reduction of soil stiffness and strength; (e) soil-structure interaction effects and permanent deformations can be evaluated.

Despite the above advantages of numerical modeling, several practical issues may limit the utilization of this analysis tool. These concerns include; (a) the engineering time required to construct the numerical model can be extensive for complex geometries; (b) numerous soil parameters are often required, thereby increasing the cost of geotechnical investigations; and (c) because very few of the available models have been validated with well-documented case studies of the seismic performance of actual retaining structures, the level of uncertainty in the analysis is often difficult to assess.

Experience demonstrates that the primary source of damage to waterfront retaining structures is liquefaction of sandy soils in the backfill, foundation, and below the dredgeline in front of the walls. Therefore, the presence of any potentially liquefiable soils should be fully analyzed and expected settlements should be computed. In many cases, remedial ground treatment will be required to increase the liquefaction resistance of the soils. Along these lines, it is noted that an important benefit of the advanced numerical modeling tools discussed above is their ability to assess the relative effectiveness of alternative methods and extents of soil improvement in reducing the potential for liquefaction and improving the seismic performance of the retaining structure.

Guidelines for Seismic Design of New Components

Current standards of practice for the seismic design of gravity retaining structures are well documented in a number of very useful and up-to-date manuals and textbooks - (e.g., Ebeling and Morrison, 1993; Kramer, 1996). These methods commonly use rigid-body limit-equilibrium methods of analysis which offer the following practical advantages: (a) the techniques are familiar to most engineers; (b) requisite input includes standard geotechnical parameters that are obtained during routine foundation investigations; and (c) the methods have been coded in very straight-forward and efficient computer programs that facilitate the performance of sensitivity studies for various design options.

Widely-used limit equilibrium methods of analysis require that potential seismically-induced movement of the wall be estimated in order to evaluate the state of stress in backfill soils (i.e., yielding versus non-yielding backfills). Determining the lateral earth pressures acting on the retaining structure is a necessary first step in the stability analyses. To estimate the dynamic lateral earth pressures, a static body force representing the inertial effects imposed by the ground motions must be added to the wall and backfill soil. Seismic design factors in the form of the pseudostatic seismic coefficients (k_h and k_v) are determined as a fraction of the maximum peak accelerations generated by the design earthquake motions. For retaining structure design, the seismic coefficients are commonly specified as one-third to one-half of the peak horizontal ground surface acceleration (pga). The standard that has been adopted in Japanese practice relates the horizontal seismic coefficient to the peak surface acceleration in the following relationship:

$$k_h = (pga/g) \qquad \text{for pga} < 0.2\text{ g}$$

$$k_h = 1/3\,(pga/g)^{0.3} \qquad \text{for pga} > 0.2\text{ g}$$

Limit equilibrium analyses can be performed as standard pseudostatic analysis (e.g., Mononobe-Okabe method as described by Whitman and Christian, 1990), or in rigid body, "sliding-block" type displacement analyses (e.g., Elms and Richards, 1990 or Whitman and Liao, 1985) which are used to estimate the seismically induced movement of the retaining structure. The specification of allowable wall deformations should take into consideration the displacement sensitivity of appurtenant structures and adjacent components.

Guidelines for Seismic Retrofit of Existing Components

In seismically active regions of the world, one the most pressing issues at ports is the anticipated seismic performance of existing gravity retaining structures. In many cases, existing structures have performed poorly for one or more of the following reasons: (a) inadequate height-to-width ratios due to the use of low seismic coefficients in original design; (b) the presence of weak foundation soils which could lead to deep-seated foundation failures; and (c) use of fill placement methods during initial construction that have resulted in loose soils that are prone to liquefaction.

Assessment of this seismic performance of existing gravity retaining structures subjected to their design-level ground motions should be evaluated using appropriate seismic analysis procedures, together with liquefaction hazard analysis procedures. Depending on the results of these analyses, appropriate seismic retrofit methods may be implemented. The most common retrofit methods include: (a) implementation of anchor systems for the gravity structure; (b) augmentation of the wall to increase its cross sectional area; (b) construction of a new wall outboard of the existing structure; or (c) replacement of the wall. Any of these methods should be supplemented with soil improvement of the surrounding fills, because of the demonstrated effectiveness of soil improvement methods in improving the seismic performance of gravity retaining walls during past earthquakes.

Soil improvement techniques have been used to mitigate liquefaction hazards to waterfront retaining walls at numerous ports throughout the world (e.g., Iai et. al., 1994). All other factors being equal, the effectiveness of the soil improvement is a function of the level of densification and the volume of soil that is treated. Although few case histories exist for the performance of improved soils subjected to design-level earthquake motions, experience has shown that caissons in improved soils have performed much more favorably than have adjacent caissons at unimproved sites which experienced widespread damage (e.g., Iai et. al., 1994).

The Japan Port and Harbour Research Institute (PHRI, 1997) has produced one of the few design guidelines that exist for specifying the extent of soil improvement adjacent to waterfront retaining structures. The recommended extent of ground treatment is shown in Figure 2-4. The stability of the caisson is evaluated using standard limit equilibrium methods in which a dynamic pressure and a static pressure corresponding to an earth pressure coefficient K=1.0 are applied along plane CD due to liquefaction of the unimproved soil. These guidelines for establishing the soil improvement area and evaluating caisson stability are valuable design tools, however, they do not address the seismically-induced deformation of the caisson and backfill soils.

In order to develop a simplified technique for estimating seismically-induced deformations of gravity caissons Dickenson and Yang (1998) have developed simplified design charts for the application of soil improvement adjacent to gravity retaining walls. The authors utilized a numerical model, validated with well-documented case histories, for parametric studies of caisson performance. The results of the study have been synthesized into practice oriented design charts for estimating the lateral deformations of gravity quay walls.

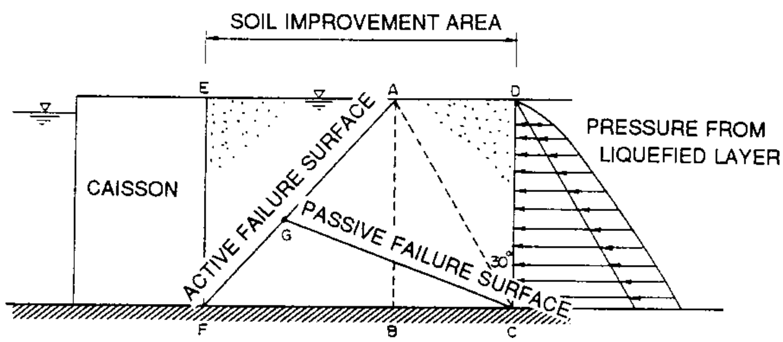

Figure 2-4: Schematic diagram for investigation of stability with respect to pressures applied from the liquefied sand layer (PHRI, 1997)

The results of the parametric study demonstrate the influence of ground motion characteristics, geotechnical parameters, and caisson geometry on the deformations of the retaining walls. These results have been synthesized into normalized parameters, where possible, to incorporate the key variables into straightforward design parameters. For example, the wall geometry has been expressed by W/H ratios as previously mentioned, the width of the zone of soil improvement is given as a function of the height of the wall (L/H). In order to account for the duration of the earthquake motions a normalized ground motion intensity has been used. This parameter is defined as the maximum horizontal acceleration at the top of the dense soil $(A_{max})_D$ divided by the appropriate magnitude scaling factor (Arango, 1996). The magnitude scaling factors are provided in Table 2-1. It is recommended that if a site specific seismic study is not performed to determine $(A_{max})_D$, then the peak ground surface acceleration can be reduced using the reduction factor (r_d) developed for estimating the variation of cyclic shear stress (or acceleration) with depth (Seed and De Alba, 1983). The values of r_d for 15 and 7.5 meter walls are approximately 0.78 and 0.95, respectively. It should be noted that the reduction factor was developed using one-dimensional dynamic soil response methods and this will yield approximate acceleration values for the two-dimensional soil-structure interaction applications discussed herein.

Table 2 –1. Magnitude Scaling Factors Derived by Arango (1996)

Earthquake Magnitude	8.25	8	7.5	7	6	5.5
MSF	0.63	0.75	1	1.25	2	3

The results of the parametric study are shown in Figure 2-5. The normalized lateral deformations at the top of the wall, X_d/H, are plotted versus the normalized width of the improved soil, L/H, and as functions of backfill density and the W/H ratios of the caissons. The numbered triangles superimposed on the charts correspond to field case histories. In this figure, the rubble fill adjacent to the caissons has been treated as non-liquefiable soil, thereby contributing to the "effect" width of the improved (i.e., non-liquefiable) soil. In the case of triangular, single-lift sections of rock fill the width of the rubble fill has been approximated as one half of the width of this fill at its base. The relationships provided in Figure 2-5 clearly demonstrate the benefit of ground treatment on the seismic performance of the caissons. It is also evident that the incremental benefit of a wider zone of ground treatment begins to decline once the soil improvement extends more than about 2.0 to 3.5 times the total height of the wall. At this point the cost of additional soil improvement may outweigh the benefits. It is interesting to note that the soil improvement guidelines prepared by the PHRI (1997) correspond to a normalized width of soil improvement of roughly 1.3 to 1.6, as supported by the work of Iai (1992).

As a screening tool for estimating the seismically-induced displacements of caissons, the recommended procedures for utilizing the results of the parametric study include:

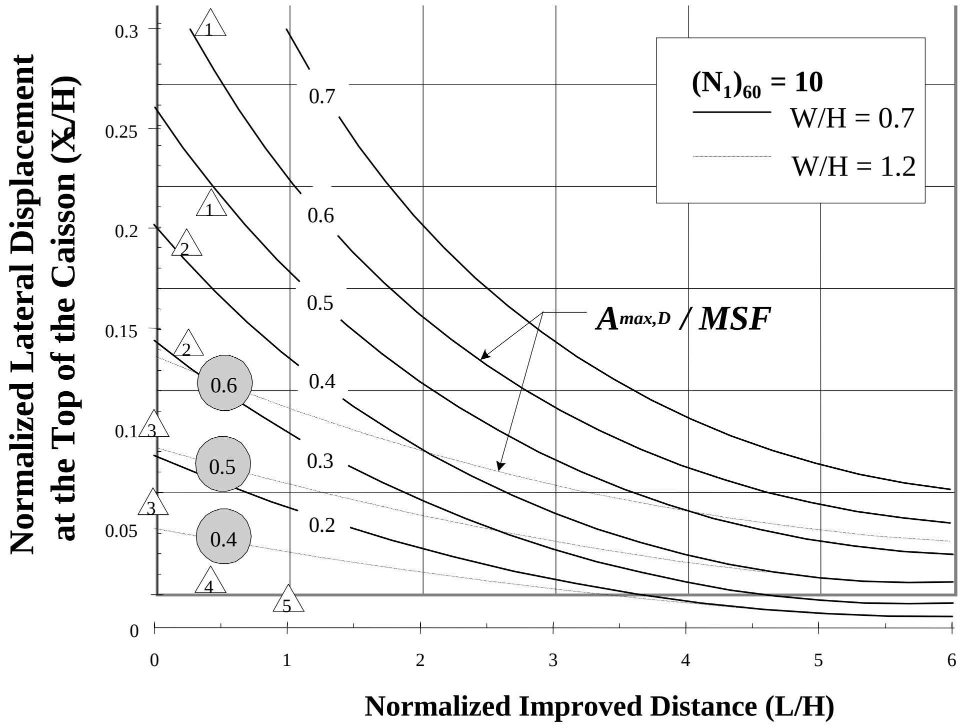

Figure 2-5(a): Normalized lateral displacements for backfill $(N_1)_{60}=10$

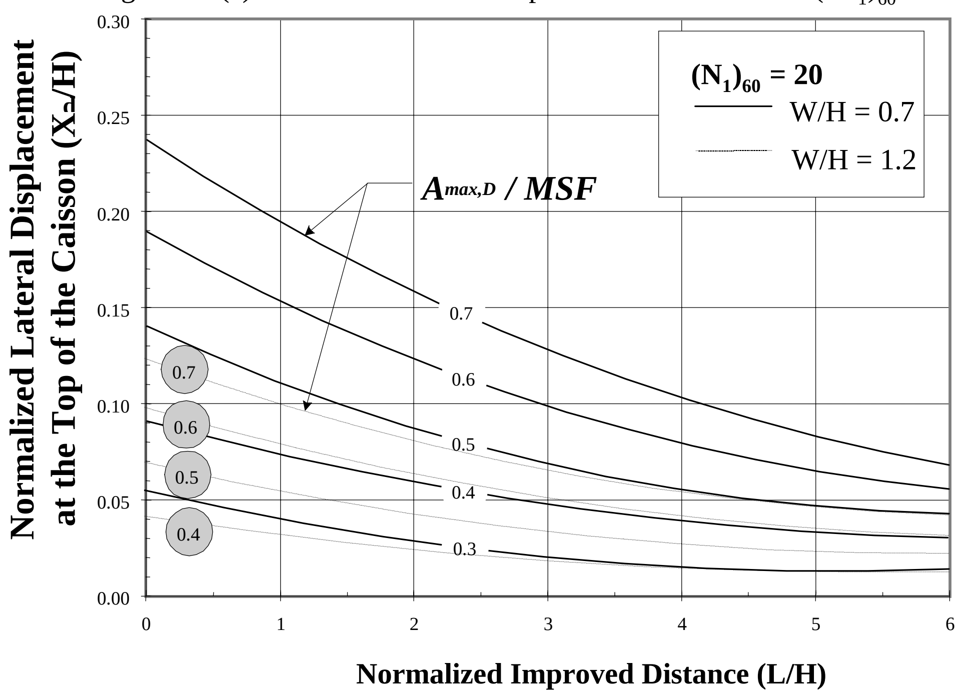

Figure 2-5(b): Normalized lateral displacements for backfill $(N_1)_{60}=20$

Figure 2-5 Design Charts for Estimating Lateral Displacements of Caissons.

1. Design the wall using standard pseudostatic limit equilibrium methods to determine the wall geometry (W/H).

2. Determine *$(A_{max})_D$* based on a site response analysis or approximate with empirical soil amplification factors to yield the peak ground surface acceleration and the reduction factor (r_d).

3. Select the magnitude scaling factor (MSF) for the specified earthquake magnitude, and compute the ground motion intensity factor as $(A_{max})_D$/MSF.

4. Given the standard penetration resistance of the backfill soils, the width of the ground treatment behind the caisson, and the ground motion intensity factor, enter Figure 2-5a or 2-5b and obtain the normalized lateral displacement. From this, the deformation at the top of the wall (X_d) can be estimated.

When it is shown to be impossible to or uneconomical to achieve the levels of performance associated with new components, an acceptable risk assessment (including economic life cycle cost analysis) should be performed to establish the most appropriate performance level form a cost-benefit standpoint.

The critical role of soil liquefaction in most earthquake-induced waterfront retaining wall failures requires that this seismic hazard be evaluated for the backfill and foundation soils. Under Level 1 earthquakes large deformations resulting in widespread pavement disruption should be avoided where economically feasible. The following guidelines for both new and existing facilities have been recommended for use at U.S. Navy facilities (Ferritto, 1997a):

> For a Level 1 earthquake the Factor of Safety against liquefaction in the backfill should be 1.5 or higher with settlements of about 1 inch or less and lateral deformations of about 3 inches or less.
>
> For a Level 2 earthquake the Factor of Safety against liquefaction in the backfill should be 1.0 or higher with settlements of about 4 inches or less and lateral deformations of about 6 to 12 inches or less. Where it may not be possible to achieve a Factor of Safety greater than 1.0, a Factor of Safety greater than 0.9 may be considered as long as the computed deformation state is shown to have limited controlled settlements and lateral spread equivalent to the values stated.

Steel Sheetpile Wharves

Types of Anchored Steel Sheet Pile Bulkheads

Steel sheet piles bulkheads function as earth retention systems, berthing structures, flood walls, and sea walls. These structures have been used where soil conditions permit driving of these relatively flexible piles. Common configurations include: (a) relatively short cantilever sheet pile walls, which derive support solely through pile stiffness and passive soil resistance beneath the dredge line; and (b) taller anchored sheet pile structures, which are supported by tie rods (at one or more elevations) fixed to mechanical anchors. The combination of weak soils commonly found in the marine environment and the wall heights required at berths precludes the use of cantilever walls in most waterfront applications. This subsection addresses seismic performance issues specifically associated with anchored sheet pile bulkheads.

A variety of anchored wall configurations have been used in the development of marginal wharves at ports (Figure 2-6). The more common configurations are briefly summarized below.

Sheet Pile Wall with Deadman Wall Anchorage. As shown in Figure 2-6, lateral support for the sheet pile wall can be provided by tie rods that extend to concrete blocks (deadman) or a continuous wall. Tie rod spacing will reflect factors such as wall height, soils in the backfill and foundation below the dredge line, and wall stiffness.

Sheet Pile Wall with Batter Pile Anchorage. In situations where adequate lateral restraint cannot be provided by a deadman, an anchor system made up of piles can be used. This is common for tall walls in relatively weak soils where the lateral earth pressures that must be resisted by the bulkhead would exceed the passive resistance provided by shallow anchor blocks. Single vertical piles, small pile groups, and batter piles have been used as effective anchors. Batter piles offer the advantage of increased lateral resistance due to their orientation relative to the wall, although their very stiff connection at the pile cap lead to problems during an earthquake.

Double (Paired) Sheet Pile Walls In regions where steel sheet piles are readily available, it is often beneficial to forego batter pile anchorages for a sheet pile wall support system. The sheet pile anchor wall is usually constructed with the same materials and geometry as the bulkhead, but is usually much shorter. The paired walls are connected with same wale and tie rod arrangement used for other anchored walls.

General Functional/Operational Requirements at Ports

The primary operational requirement of sheet pile bulkheads is that they resist lateral earth pressures with minimal deformation. These requirements are the same as those for gravity retaining structures.

Guidelines for Seismic Performance Requirements

The seismic performance requirements for anchored sheet pile bulkheads are essentially the same as the requirements for gravity retaining structures previously discussed. Briefly summarized, the design of anchored sheet pile bulkheads should limit permanent lateral displacement at the top of the sheet pile to values that are based on the displacement tolerances of the key port components in the vicinity of the sheet pile structure. For example, the limiting displacement criteria for sheet pile bulkheads at U.S. Navy ports is as follows (Ferritto, 1997a): (a) under the Level 1 ground motions, the permanent lateral displacement at the top of the bulkhead must be less than 1 in.; and (b) under the Level 2 ground motions this permanent lateral displacement must be less than 4 in. These values are presented as examples only, and different displacement values may be selected at a port, depending on the port's overall seismic performance requirements and those of the port components located near the bulkhead. The results of advanced numerical modeling of anchored sheet pile bulkheads commonly indicate that the 4-inch displacement limit used by the Navy for Level 2 earthquake motions cannot be met by standard bulkheads. In addition, in cases where the anchor does not experience catastrophic failure, the maximum displacements during the stronger earthquake motions do not occur at the top of the bulkhead wall, but instead occur between the elevation of the anchor and the dredge line. These lower displacements will still yield excessive ground surface deformations. These factors should be considered when establishing seismic performance requirements for sheet pile bulkheads.

Guidelines for Preliminary Seismic Vulnerability Assessment

Review of Design and Construction Documents As for other port waterfront structures, preliminary assessment of the seismic vulnerability of existing sheet pile bulkheads should be based on visual inspections and review of design and construction documents. Because of the lack of accessibility to underwater or underground elements of the bulkhead, many aspects of its seismic vulnerability can only be assessed through review of its design and construction documents. Documents that should be reviewed for sheet pile bulkheads are design drawings and calculations, soils reports, and construction documents.

The review of design and construction documents should focus on: (a) review of pertinent geotechnical reports and construction documents to identify the existence of if the soils below the dredge line, in the backfill, or in the foundation are potentially unstable; and (b) the review of the seismic design procedures (if any) that were employed during the design of the sheet pile bulkhead, to check whether the original design assumptions are consistent with current knowledge and practice. In this, special attention should be paid to the type of tie-rod\ anchor system (e.g., tie rods and deadman, tie-rods and anchor wall, tiebacks with grouted anchor, tie-rods and pile anchors) and the location of the anchor. Field experience, supplemented by numerical analyses, demonstrate that the anchor must be located further behind the wall than that specified for static design. Also, tie-rod failures constitute one of the most common failure modes of sheetpile bulkheads during earthquakes; therefore reevaluations should focus on the dynamic forces expected during the design level earthquakes. As-built construction data should be reviewed to determine the capacity of the tie rods and the connections between the tie rods and the bulkhead.

Field Inspection Despite the above-indicated accessibility problems, visual inspection can still provide important information for assessing the seismic vulnerability of existing sheet pile bulkheads. For example, such assessments may uncover evidence of ground movement under static conditions in the vicinity of the bulkhead that could indicate a potential for poor performance during future earthquakes. They can also serve as a means for documenting the existence of any visible corrosion of the sheet piles, as well as the types of port components and utilities in the vicinity of the sheet pile structures. This can provide a basis for assessing whether the allowable ground displacement criteria previously established for the sheet pile bulkhead are consistent with the limiting displacement and deformation tolerances of these other components.

In cases where serious corrosion has been observed, it may be necessary to remove specimens of the sheet piles for inspection and testing to insure the integrity of the corroded sections. In some cases, evidence of severe corrosion of sheet piles beneath the waterline has been manifested as sinkholes at the ground surface adjacent to the structure, due to the loss of backfill soil through holes in the sheet piles. Divers and/or side-scan sonar techniques should also be used to facilitate underwater inspection of the sheet piles and the depth of potential scour along their face. Additional evaluation may involve excavating along selected portions of the bulkhead to reveal the integrity of the tie rod-wale connections.

Guidelines for Seismic Analysis

Seismic analyses for new and existing bulkheads must focus on the dynamic behavior of the foundation and backfill soils, as well as the overall stability of the walls. Again, potential failure modes include: excessive deformation of the bulkhead, passive failure of soil in front of anchors, tie rod failure, wale system failure, loss of passive soil resistance beneath the dredge line, interlock failure between sheet piles, and global stability when founded on weak soils, and potential damage to very stiff batter pile supported anchors at the connection of the piles to the anchor.

Widely used limit equilibrium methods require that potential seismically-induced movement of the wall be estimated in order to evaluate the state of stress in backfill soils (i.e., yielding versus non-yielding backfills). It should be noted that the nonuniform deformations typical of flexible sheet pile structures are not strictly accounted for in standard rigid body, limit equilibrium analyses. This variation in soil deformation over the height of the wall results in a statically indeterminant problem. Approximate methods are nonetheless used to evaluate the static and dynamic performance of these structures (e.g., free-earth support and fixed-earth support methods of analysis). Recommended references for seismic analysis and design of sheet pile bulkheads are provided by; Ebeling and Morrison (1993), USACOE (1994), and Kramer (1996).

As with the gravity retaining walls, determining the lateral earth pressures acting on the retaining structure is a necessary first step in the seismic analysis of sheet pile bulkheads. In order to estimate the dynamic lateral earth pressures, a static body force representing the inertial effects imposed by the backfill soil during the ground shaking must be applied to the bulkhead wall. and backfill soil. (Since the mass of the bulkhead is very small, the resulting inertia forces due to structure weight will be small when compared to the effective soil mass.) Seismic design factors in the form of the pseudostatic seismic coefficients (k_h and k_v) are determined as a fraction of the maximum peak accelerations generated by the design earthquakes. As for gravity retaining wall design, the seismic coefficients established for design of sheet pile bulkheads are commonly specified as one-third to one-half of the peak horizontal ground surface acceleration (pga). The appropriate ratio to use will reflect the specified factor of safety for stability and the allowable deformations (i.e., the smaller the allowable deformations the larger the lateral seismic coefficient). The pseudostatic forces computed using these seismic coefficients are applied to the body of soil behind the sheet pile bulkhead structure. Japanese standards for establishing these seismic coefficients are the same as previously described for gravity retaining structures.

There is extensive experience on the performance of anchored sheetpile walls. Extensive liquefaction of loose saturated cohesionless soils in the backfill have caused major failures. Typical failures take the form of excessive permanent seaward tilting with associated movement of the anchor block. Associated with this is the settlement and cracking of the backfill soil. Gazetas and others (1990) review procedures used to analyze quaywalls. Pseudostatic procedures are used to determine lateral earth pressures after the well known Mononobe-Okabe approach. Statistics show that performance of quaywalls over the last 45 years has not improved despite increases in the seismic coefficients and refinements in the design methods. The dominant factor in failures of these walls is the loss of strength of the backfill and foundation soils. The pseudo static method of analysis suffers from three significant deficiencies: the failure to account for the loss of strength associated with the generation of excess pore pressure, the overestimation of the passive soil resistance of the anchor, and the inability to include the deformation and movement of the wall and soil. Many designs underestimated the level of seismic exposure and the design procedure ignores the vertical component of acceleration. which can increase the effective acceleration relating to active and passive earth pressures.

Gazetas and others (1990) developed an empirical design chart based on numerous case studies of sheetpile walls at sites where liquefaction was not observed at the ground surface. This chart based screening tool can be used to enhance conventional pseudostatic procedures. A horizontal acceleration factor is defines as:

$$k_h = \frac{2a_h}{3g} \qquad \text{2-1}$$

A vertical acceleration factor may be assumed as two-thirds of the horizontal.

$$k_v = 2/3\ (k_h) \qquad \text{2-2}$$

An effective acceleration coefficient is defined as:

$$k_e = k_h / (1 - k_v) \qquad 2\text{-}3$$

For cohesionless soils under water, the value of k_e may be increased by 1.5 to account for the potential of strength degradation from porewater pressure buildup. Figure 2-7 shows the nomenclature used. Figure 2-8 shows relationships for the active failure surface inclination, α_{ae}, and the active and passive seismic pressure coefficients as functions of the effective acceleration. The effective anchor distance, EAI is defined as

$$EAI = d / H \qquad 2\text{-}4$$

Having the effective acceleration coefficient one may determine the failure surface inclination and the seismic pressure coefficients. A trial value of EAI may be selected and the anchor length determined using Figure 2-9 and:

$$EPI \approx \frac{K_{PE}}{K_{AE}} \;(r2 \quad (r+1)) \qquad 2\text{-}5$$

where

$$r = f / (f + H) \qquad 2\text{-}6$$

$$L \geq (H + f) \operatorname{Cot}(\alpha_{ae}) + (EAI_c) \; H \qquad 2\text{-}7$$

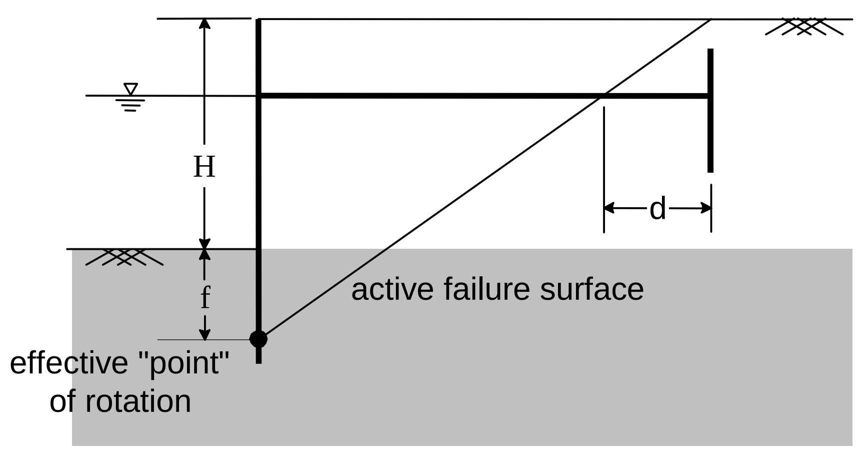

Figure 2-7: Definition of Effective Ancho Index : EAI = d/H (Gazetas et. al., 1990).

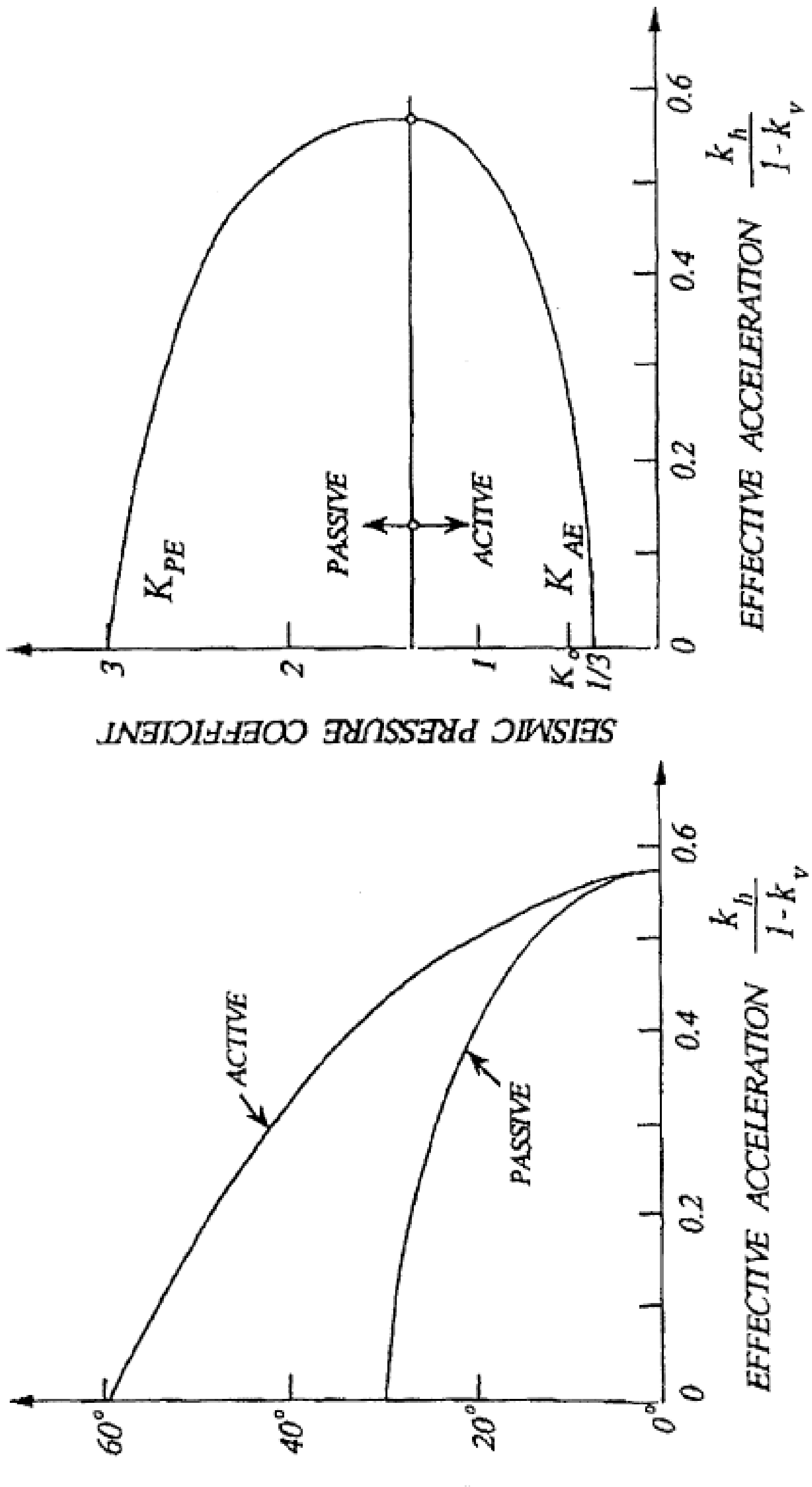

Figure 2-8: Effect of Horizontal and Vertical Seismic Coefficients on the Angle of the Active and Passive Sliding Wedges (Gazetas et. Al, 1990).

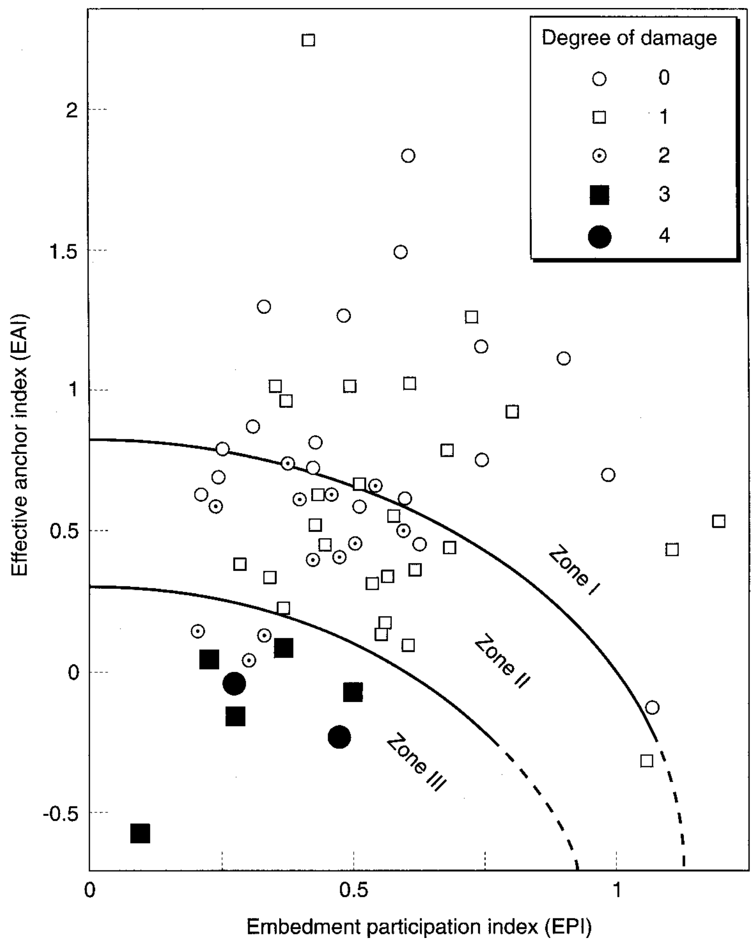

Figure 2-9: The Seismic Design Chart of Gazetas et. al. (1990).

The procedure developed by Gazetas and his coworkers (Dennehy, 1985; Gazetas et. al., 1990) establishes a minimum anchorage length for safe performance based on field observations of damaged structures at sites where surface evidence of liquefaction was lacking. In a recent study, the results of recent parametric studies (McCullough and Dickenson, 1998) which included non-liquefiable soils were compared to the design chart in Figure 2-9.. Figure 2-10 presents the comparison between the parametric study (for non-liquefiable soils) plotted as solid stars (with the calculated displacements in parenthesis) and the design chart presented by Gazetas and others. One point (center-right) is a standard 7.5 m wall. Two points (top-right and bottom-right) are from the parametric study varying the length of the tie rod anchor, and the last point (left-center) comes from the parametric study varying the depth of embedment.

It is evident from Figure 2-10 that the computed deformations vary significantly at each data point, and that some of the plotted points have calculated displacements that both fit, and do not fit the proposed design chart (especially the point on the center-right). The variations in the displacement values for each of the plotted parametric study points can be attributed to variations in the earthquake motions. Larger earthquake motions produced larger displacements, whereas the method proposed by Dennehy and Gazetas does not directly include any earthquake motion parameters (intensity, frequency or duration). It is significant to note that many of the computed deformations that fall in Zone I (deformations approximately less than 10 cm) would be considered unacceptable by many port engineers for an operating or contingency level earthquake motion (Ferritto, 1997).

A comparison can also be made between the parametric study on the depth of sheet pile embedment and the proposed chart by Dennehy and Gazetas. It was noted from the parametric study that the depth of embedment had very little effect on the performance of the bulkhead over the range of the modeled values, but the contour lines constructed by Dennehy and Gazetas show a clear variation in performance over the range of interest (EPI = 0.25 to 0.75).

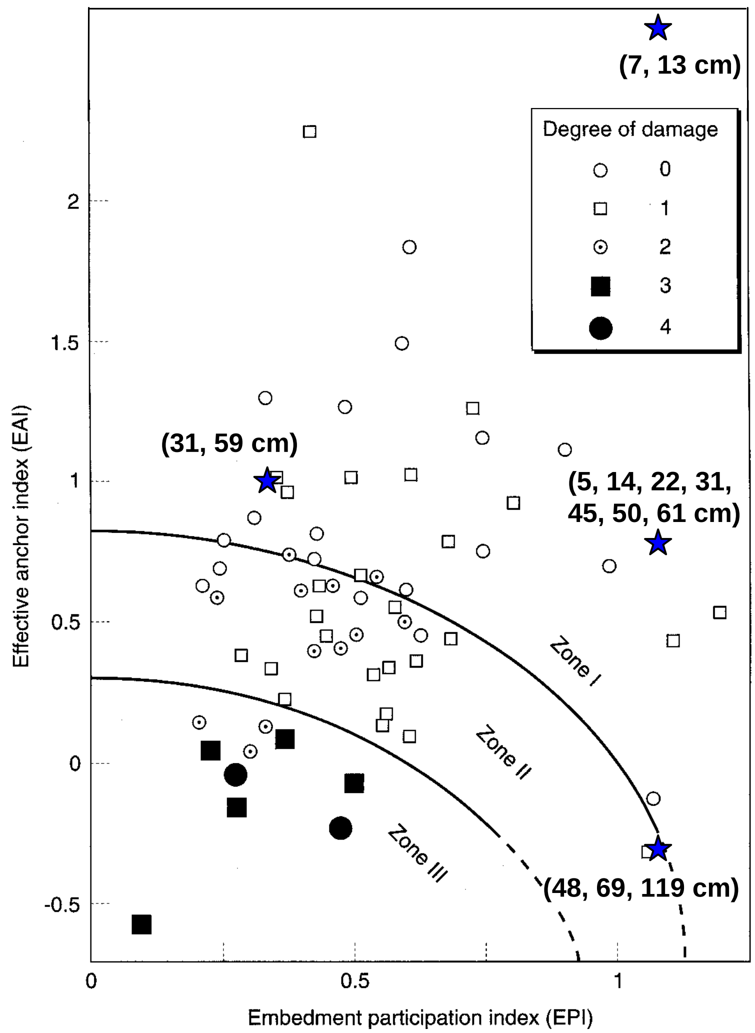

Figure 2-10: Including Data from the Parametric Study for Models with Improved Soils (solid stars)

As previously discussed for gravity retaining structures, advanced numerical modeling techniques are recommended for estimating earthquake-induced permanent displacements in displacement-sensitive sheet pile bulkheads. The applicability and benefits of advanced numerical procedures seismic analysis of sheet pile bulkheads are the same as previously discussed for gravity retaining structures.

An example of an extensive parametric study of anchored sheetpile bulkheads using a dynamic effective stress numerical model is provided by McCullough and Dickenson (1998). The evaluation of five design parameters were examined in the parametric runs, including: (a) depth of embedment of the sheetpile wall (*D*), (b) stiffness of the sheetpile wall (*EI*), (c) length of the tie rod, (d) density of the backfill soil, and (e) extent of soil improvement (*SI*). A definition sketch of the modeled geometry is provided in Figure 2-11.

To increase the applicability of this study, normalized dimensionless factors were developed. A normalized displacement factor (Equation 3) was developed by normalizing the displacements at the top of the wall (ΔX) by the wall stiffness (EI),

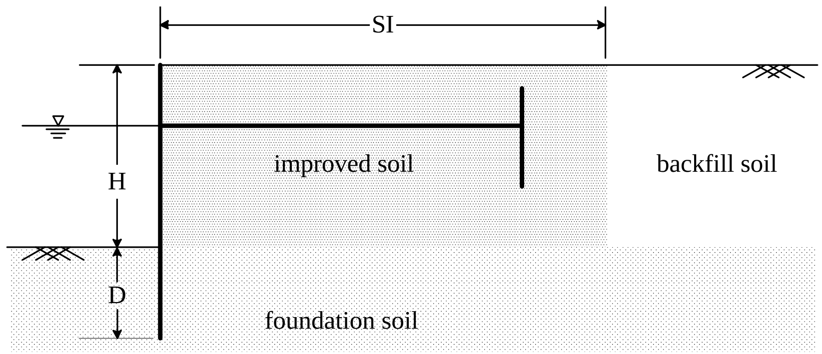

Figure 2-11: Definition Sketch Of An Anchored Sheetpile Bulkhead With Soil improvement

total wall height (H+D), and the buoyant unit weight of the backland soil adjacent to the sheet pile wall.

$$\frac{\Delta X \cdot EI}{(H + D)^5 \cdot \gamma_b} \qquad 2\text{-}8$$

A normalized soil improvement factor (n) was also developed by dividing the extent of soil improvement (*SI*) by the total wall height (H+D).

The normalized earthquake intensity was developed by normalizing the maximum backland acceleration at the elevation of the dredge line ($A_{max@dredge}$) the magnitude scaling factor (*MSF*) (Arango, 1996). The *MSF* factors are listed in Table 2-2. In the absence of a site specific seismic study, it is recommended that the reduction factor (r_d) from Seed and Idriss (1982) be used to approximate $A_{max@dredge}$ from the maximum ground surface acceleration. The values of r_d for 15 and 7.5 meter walls are approximately 0.78 and 0.95, respectively. It should be noted that this is a one-dimensional approximation for the two-dimensional soil-structure interaction.

Table 2-2: Magnitude Scaling Factors (Arango, 1996)

Magnitude					
5.50	***6.00***	***7.00***	***7.50***	***8.00***	***8.25***
3.00	2.00	1.25	1.00	0.75	0.63

The results of the parametric study are presented in Figure 2-12. The contour lines indicate various levels of earthquake intensity for backfill soils with blowcounts of 10 and 20 blows/30 cm. The effectiveness of soil improvement for minimizing bulkhead deformations is clearly demonstrated by the design chart. It is also noted that incremental benefit of soil improvement beyond *n* values of approximately 2.0 decreases considerably. In comparison, the *n* values as determined from the PHRI (1997) recommendations for the parametric study sheetpile bulkheads are approximately 1.9 to 2.5.

There are fourteen case histories plotted on the chart, which are arranged according to the blowcounts of the backfill soils. Table 2-3 presents pertinent data from the case histories. It should be noted that seven case histories are closely predicted by the chart, five case histories are significantly over-predicted and only two of the case histories are significantly under-predicted. These results indicate that the design chart can be conservatively used as a preliminary design chart or screening tool.

The results of the study indicate that it would be very difficult to limit the deformations to 10 cm utilizing only densification methods of soil improvement for moderate to high earthquake motions ($A_{max} \geq 0.3g$). In cases such as these, it may be necessary to consider soil-cement techniques and/or structural improvements. The results of this study have been synthesized into a simplified design chart for use in estimating permanent lateral displacements for sheetpile bulkheads with or without soil improvement. This design chart is applicable for the preliminary design of new bulkheads and as a screening tool for existing bulkheads.

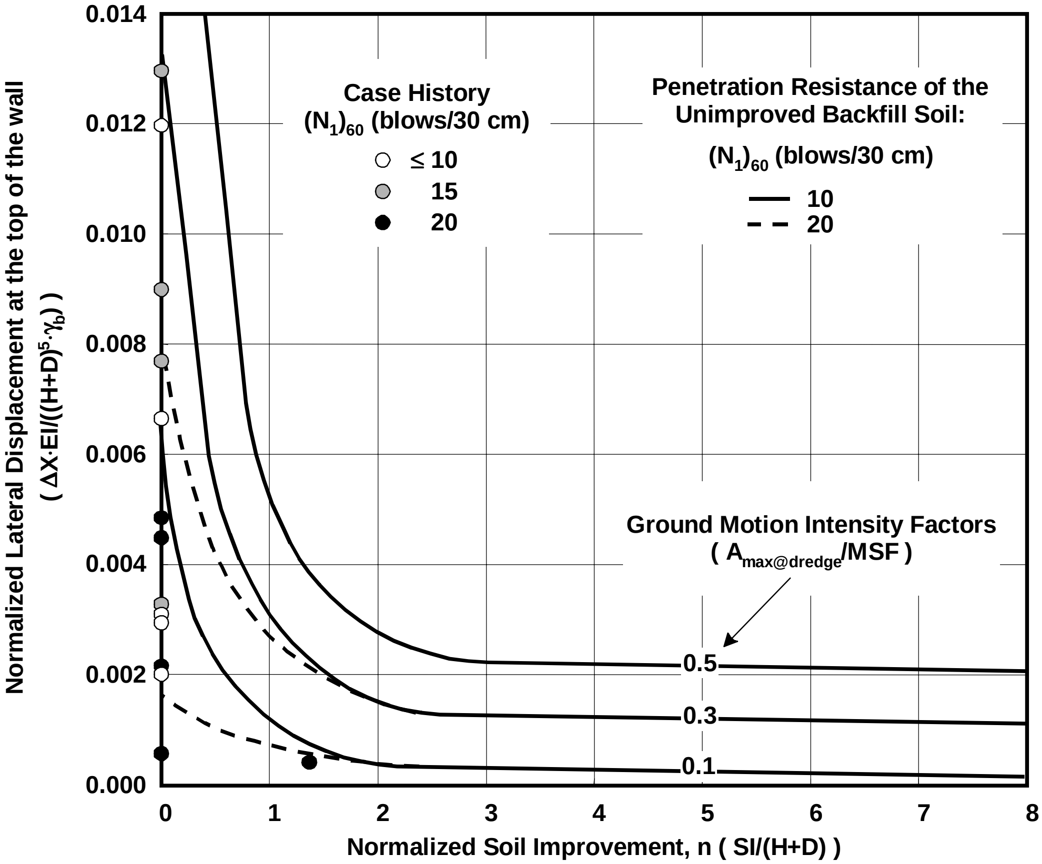

Figure 2-12: Permanent Horizontal Displacements at the Top of Anchored Bulkheads

Table 2-3: Plotted Case Histories

Earthquake	*$(N_1)_{60}$*	*$A_{max@dredge}$/MSF (g)*	*Displacement (cm)*	*Normalized Displacement X EI/((H+D)^5 $_b$)*
1968 Tokachi-Oki	6	0.26	12 to 23	0.0120
1993 Kushiro-Oki	6	0.20	19	0.0020
1964 Niigita	10	0.14	100	0.0031
1983 Nihonkai-Chubu	10	0.10	110 to 160	0.0066
1968 Tokachi-Oki	15	0.23	16 to 57	0.0090
1968 Tokachi-Oki	15	0.23	12 to 19	0.0029
1968 Tokachi-Oki	15	0.12	30	0.0077
1973 Nemuro-Hanto	15	0.16	30	0.0031
1978 Miyagi-Ken-Oki	15	0.10	87 to 116	0.0129
1968 Tokachi-Oki	20	0.26	~ 60	0.0049
1983 Nihonkai-Chubu	20	0.09	~ 5	0.0006
1993 Kushiro-Oki	20	0.16	50 to 70	0.0044
1993 Guam	20	0.18	61	0.0019
1993 Kushiro-Oki	30	0.21	no damage (~ 5)	0.0004

The recommended procedures for utilizing the results of the parametric study to estimate the permanent displacement at the top of sheetpile walls, include;

1) Design the wall using current pseudo-static methods to determine the wall geometry (*H*, *D*, *EI*, and anchor length).

3) Determine $A_{max@dredge}$ from a site-specific seismic study or an approximate empirical relationship.

4) Determine the earthquake intensity factor for the specific earthquake by dividing the magnitude scaling factor into $A_{max@dredge}$.

5) Based on the density of the backfill and the extent of soil improvement, estimate the permanent lateral displacement at the top of the sheetpile wall (ΔX) using Figure 2-12 and the normalized displacement equation (Equation 2-8).

Current pseudo-static design methods allow for determination of the sheetpile wall section, tie rod length, and depth of embedment, but since they are limit-equilibrium based, it is not possible to estimate lateral deformations using these methods.

Guidelines for Seismic Design of New Components

As previously mentioned regarding the seismic design of gravity retaining structures, the seismic design of steel sheet pile bulkheads must focus on the dynamic behavior of the foundation and backfill soils, as well as the overall stability of the bulkhead wall retaining structure. Widely used limit equilibrium methods require that potential seismically-induced movement of the walls be estimated in order to evaluate the state of stress in backfill soils (i.e., yielding versus non-yielding backfills). The flexibility of anchored sheet pile bulkheads has led designers to assume yielding backfill and employ the dynamic earth pressure method of Mononobe and Okabe (as outlined by Ebeling and Morrison, 1993). Enhancements to the standard pseudostatic methods of analysis have been made by numerous investigators (e.g., Neelakantan et. al., 1992; Power et. al., 1986; Steedman and Zeng, 1990). It is noted that these methods do not indicate the lateral deformations of the bulkhead that would likely occur during the design earthquake.

Based on an extensive review of seismic performance data for anchored sheet pile bulkheads, Kitijima and Uwabe (1979) concluded that the level of damage to these structures is related to the permanent deformations of the top of the bulkhead wall during the earthquake. Their general observations are summarized in Table 2-4, and underscore the importance of the structure deformations and the associated lateral ground movement for the establishment of seismic design criteria for anchored sheet pile bulkheads. Rigid body, "sliding-block" type displacement analyses have been used as the basis for estimating the seismically-induced movement of anchored retaining structures (Towata and Islam, 1987). In addition, a semi-empirical method based on the performance of anchored bulkheads at sites which did not exhibit significant liquefaction has been developed by Gazetas et. al. (1990) for estimating the deformations of anchored bulkheads based on dynamic earth pressures and the bulkhead-to-anchor spacing. These techniques are recommended as initial screening methods for evaluating the seismic performance of anchored bulkheads.

Table 2-4
Relationship Between The Deformation Of Anchored Sheet Pile Retaining Walls And Observed Damage
(Kitijima And Uwabe, 1979)

Description of Damage	**Permanent Displacement at Top of Sheet Pile**	
	cm	**inches**
No Damage	<2	<1
Negligible Damage to Wall itself, and Noticeable Damage to Appurtenant Structures	10	4
Noticeable Damage to Wall	30	12
General Shape of Anchored Sheet Pile Preserved, but Significantly Damaged	60	24
Complete Destruction. No Recognizable Shape of Wall	120	48

As with all marginal wharf structures, seismic hazards associated with soil liquefaction must be mitigated in order to reduce earthquake-induced deformations to within allowable limits. Soil improvement techniques have been used at ports throughout the world to increase the liquefaction resistance of soils adjacent to waterfront retaining structures. Although ground treatment is applicable adjacent to anchored sheet pile bulkheads, the flexible nature of the bulkhead walls is such that lateral deformations should be expected even in competent, non-liquefiable soils subjected to high intensity shaking. As mentioned in previous sections of this chapter, the allowable deformations will reflect the sensitivity of appurtenant structures. It should also be noted that several of the soil improvement techniques may not be applicable in close proximity to the sheet pile bulkhead (within 20 ft. or less). In several documented case studies, lateral deformations of the backfill soils adjacent to the bulkhead during densification or grouting have resulted in increased loads in the tie rods and wales, and increased bending stresses in the piles (PHRI, 1997). Finally, structural measures may be required to restrengthen anchor systems. Retrofit strategies could include a second row of anchors tied to the first, the construction of more robust anchors (i.e., larger deadman, larger piles, etc.), reconstructed tie rod-wale connections.

The advantages and disadvantages of the various analytical procedures for analyzing the seismic performance of sheet pile structures (e.g., pseudostatic limit equilibrium, sliding block type analyses, advanced numerical modeling) limit are similar to those outlined for gravity retaining structures.

Guidelines for Seismic Retrofit of Existing Components

Experience during past earthquakes demonstrates that liquefaction-related phenomena constitute the primary seismic hazard to sheet pile bulkheads. Therefore, one of the most effective measures that can be used to reduce the seismic risk due to failure of such structures is the use of soil improvement adjacent to the bulkhead wall and anchor system (e.g., vibro-techniques, stone columns, gravel drains, grouting). Soil improvement may also be required in front of wall to ensure adequate passive resistance below the dredge line. Guidelines for the utilization of soil improvement for sheet pile bulkheads have been presented PHRI (1997). Figures 2-13 and 2-14 show their recommended layout for the volume and extent of soil improvement that is required to minimize the bulkhead deformations to allowable levels under various levels of ground shaking. Again, as previously noted, design engineers should keep in mind that several of the soil improvement techniques may not be applicable in close proximity to the bulkhead. Lateral deformations of the backfill soils adjacent to the bulkhead during densification or grouting have been observed to increase loads in the tie rods and wales.

Re-evaluation of the bulkheads may indicate that anchor systems should be further strengthened. In these situations, retrofit strategies could include a second row of anchors tied to the first, and the construction of more robust anchors (i.e., larger deadman, larger piles, etc.), reconstructed tie-rod\ wale connections.

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