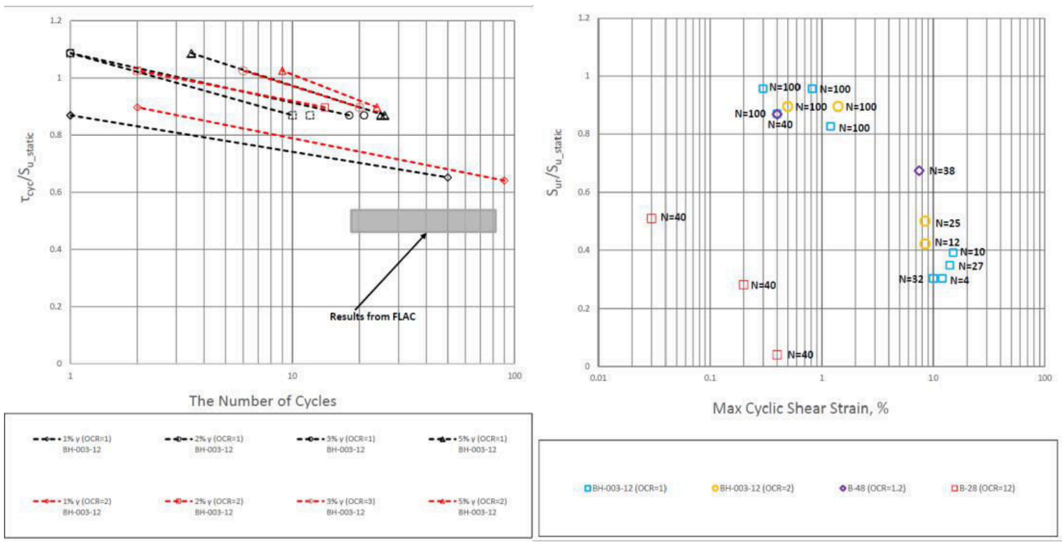


Figure 11. BCF Cyclic Strength Ratio

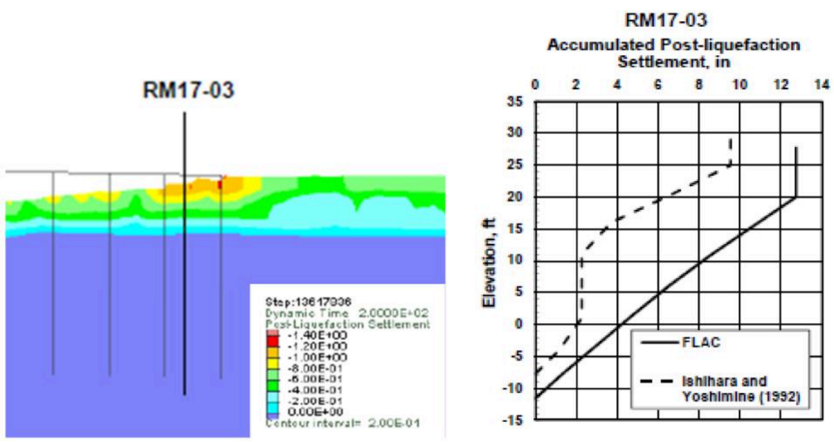


Figure 12. Maximum Shear Stress Contours for CMS and UHS Compatible Motions


Figure 13. Ground Motion Recording Station Location Map

The magnitude of liquefaction-induced ground settlement under the MCE at the approach embankments was also computed using 2D FLAC with the PM4Sand constitutive model to simulate the post-liquefaction reconsolidation settlement landside and along the slope. The maximum excess pore pressure ratio ($r_u$) calculated at the end of shaking was used to determine the volumetric strain. Volumetric strain profiles were then integrated to estimate the total vertical settlement. The relationship between maximum $r_u$ during cyclic loading and the associated post-liquefaction volumetric strain resulting from reconsolidation settlement of silt-rich soils presented by Beaty el. al. (2014) was applied. Based on these analyses, we estimate that the average liquefaction induced settlement will be approximately 13 inches on shore and 5 inches mid-slope. For comparison purposes, liquefaction induced ground settlement using simplified semi-empirical methods developed for sands by Ishihara and Yoshimine (1992) are presented at RM17-03 an on shore boring. In general, the results from FLAC and Ishihara and Yoshimine (1992) are in general agreement and the differences in Ishihara and Yoshimine are attributed to the non-continuous nature of SPT sampling and the corresponding safety factor against liquefaction which is used in the calculation.

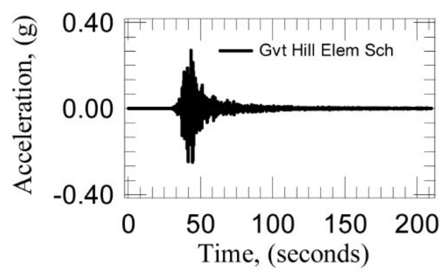


**Figure 14: Surface Motion**

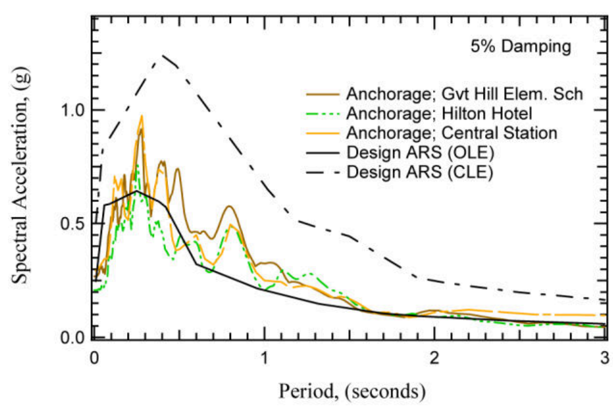


**Figure 15: ARS for M7.0 Earthquake**

## 3D EFFECTS

For the DSM option, the movement of the adjacent untreated slopes will result in additional "drag" forces along the outer DSM walls. This friction force was accounted for in the 2D FLAC model by applying a traction force within the DSM zone equal to the post cyclic shear strength of the tidal silt at 10% shear strain. The traction force was multiplied by 2 to account for each side of the DSM and then divided by the width (transverse) dimension of the DSM. Relative movement within the DSM model and "free field" model at the same node points within the DSM zone were plotted versus time. Based on the relative movements the time frame within the earthquake was identified when the adjacent slope was moving downslope relative to the DSM. At these times the drag force was activated. The relative movement also indicated the maximum relative displacement occurred at the top of the DSM and decreased to almost zero at the base of the DSM. To account for this distribution of relative movement, the drag force was varied linearly from 100% of the shear force to zero at the base. For the MCE events Tohoku and Maule resulted in the greatest slope displacements. The resulting 3D effects increased the displacement of the DSM by about 1 inch.

## NOVEMBER 30TH, 2018 M7.0 POINT MACKINZIE EARTHQUAKE

On November 30th, 2018, about one month after the completion of the ground improvement, a M7.0 earthquake occurred approximately 12 km from Anchorage. As part of a post seismic assessment of the completed ground improvement, a comparison was made to determine how the level of shaking for the M7.0 earthquake compared to the design response spectra at the PCT Site. Based on a comparison of three nearby monitoring stations (Figure 13) it was concluded that the November 30th earthquake approximately corresponds to an OLE event. A plot of the Point Mackenzie ground motion and response spectra comparison are presented in Figure 14 and Figure 15, respectively.

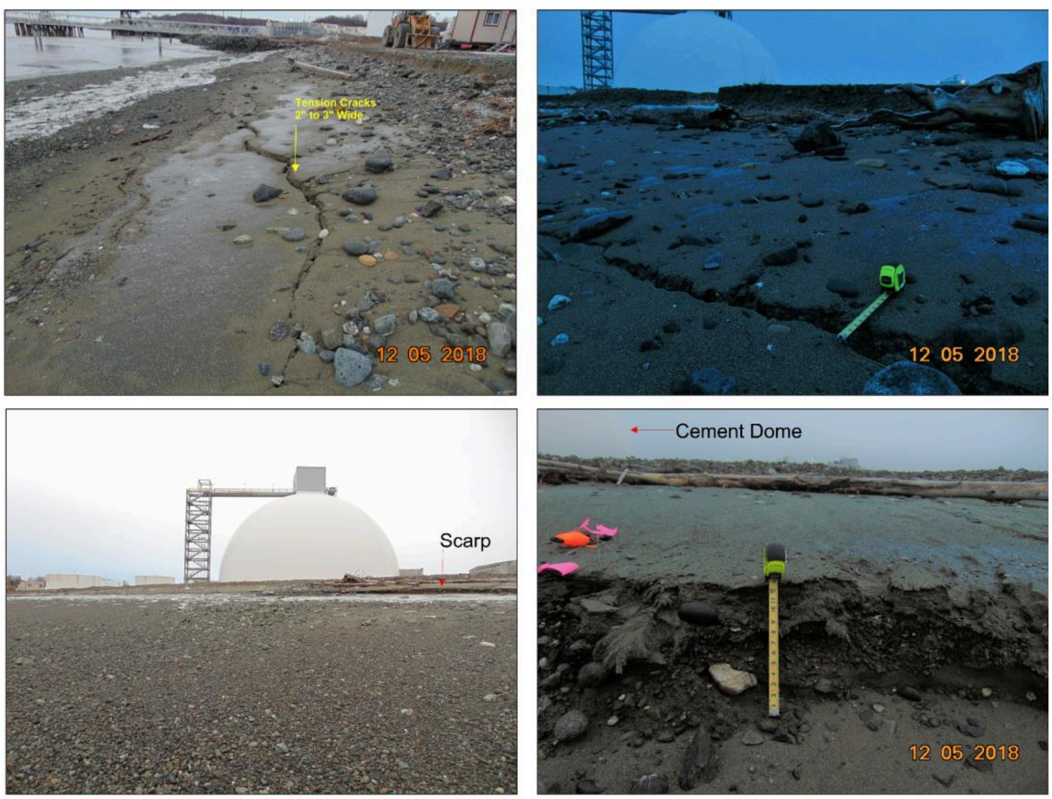


**Figure 16: Field Observations at PCT (December 5th, 2018)**

A post-seismic survey of the PCT site was performed by COWI to observe port-wide ground deformations, including slope movement near or within the ground improvement zone. The slopes adjacent to both sides of the ground improvement showed tension cracks ranging from 2 to 3 inches on the shoreline slope to the north of the ground improvement and up to 6 inches for the slope to the south of the ground improvement. Additionally, scarping of 3 to 6 inches and 6 to 12 inches was observed for the slopes north and to the south of the ground improvement, respectively. The ground improvement zone showed no signs of vertical or horizontal displacement. Photographs of the PCT post seismic survey are presented in Figure 16.

## CONCLUSIONS

This paper has presented an overview of the design methodology used for the new PCT berth at the Port of Alaska. The design process utilized high quality cyclic laboratory data for both the low plasticity Tidal Silt deposit and the well-known Bootlegger Cove Clay deposit which were calibrated to constitutive models used in the dynamic soil-structure-interaction analyses. Methods for accounting for 3D behavior of the ground were presented including time dependent drag forces acting on the sides of the soil improvement and the increase in soil pressure acting on the piles resulting from the presence of the DSM panels. A unique opportunity to validate the numerical model presented itself with the occurrence of M7.0 Point Mackenzie earthquake just after the completion of the ground improvement installation. The initial post seismic survey of the PCT site indicated that unimproved slopes showed several inches of lateral movement and settlement while the DSM improved ground showed no observable displacement under what is estimated to be an OLE event level of shaking.

## REFERENCES

ASCE/COPRI (2014a). ASCE Standard, [ASCE/COPRI 61-14] Seismic Design of Piers and Wharves.

Beaty, M.H., Schlechter, S., Greenfield, M., Bock, J., Dickenson, S. Kempner, L., and Cook, K., 2014, Seismic Evaluation of Transmission Tower Foundations at River Crossings in the Portland-Columbia River Region, 10th U.S. National Conference on Earthquake Engineering Frontier of Earthquake Engineering.

Boulanger, R. W., and Ziotopoulou, K., 2012. “PM4Sand (version 2): A sand plasticity model for earthquake engineering applications.” Report No. UCD/CGM-12/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, May, 100 pp. (Revision 1 posted July 2012)

Beyzaei, C., Cabas, A., Christie, S., Dickenson, S., Franke, K., Koehler, R., Pierce, I., Stuedlein, A., Yang, Z., 2019. Geotechnical Engineering Reconnaissance of the November 30th 2018 Anchorage, Alaska Earthquake, Geotechnical Extreme Events Reconnaissance Association (GEER), GEER-059.

Darendeli, M. B. 2001. Development of a new family of normalized modulus reduction and material damping curves. PhD dissertation, University of Texas at Austin, Austin, Texas.

FHWA 2013, Deep Soil Mixing for Embankment and Foundation Support, HWA-HRT-13-046, (2013).

Ishihara and Yoshimine, 1992. “Evaluation of settlements in sand deposits following liquefaction during earthquakes.” Soils Found., Jpn. Geotech. Soc., 32 (1) (1992).

Itasca Consulting Group. 2014. Fast lagrangian analysis of continua, version 7.0

Ladd, C.C. and Foote, R. (1974). A new design procedure for stability of soft clays. Journal of the Geotechnical Engineering Division. ASCE, Vol. 100, No. GT7. pp. 763-786.

Zapata-Medina, D.G., and Finno, R. J. 2014. "Effects of Construction-Induced Stresses on Dynamic Soil Parameters of Bootlegger Cove Clays." Journal of Geotechnical and Geoenvironmental Engineering. ASCE. Vol. 140. January.

# Seismic Design of Anchored Bulkheads: The Geotechnical Perspective

Monique A. Anderson, P.E., M.ASCE[1]; and SDB Task Committee[2]

[1]Shannon and Wilson, 400 North 34th St., Suite 100, Seattle, WA 98103. E-mail: maa@shanwil.com
[2]ASCE COPRI Task Committee for Seismic Design of Bulkheads

## ABSTRACT

Geotechnical design of anchored bulkheads is largely dependent on the static and seismic increment of active lateral earth pressures acting on the bulkhead, and the lateral resistance provided by the passive earth pressure and anchor systems. The geotechnical design approach for bulkhead design under earthquake conditions has historically been based on approaches developed for dry, uniform soils, rigid walls, or piles, with adjustments intended to account for wall flexibility. Considering the limitations associated with these historical procedures and the widespread adoption of performance-based seismic design principals in contemporary port practice, the ASCE COPRI Task Committee for Seismic Design of Bulkheads has reviewed existing design methods and developed a supplemental approach that supports the need for deformation-based seismic design of anchored bulkheads. This approach will be part of a new guidelines document for seismic design of anchored bulkheads. This paper summarizes the strengths and weaknesses of current seismic design methods, and provides suggested practice-oriented, force- and deformation-based analysis approaches for use by geotechnical engineers. A companion paper will discuss related structural design issues. The guidelines are anticipated to be published in early 2020.

## INTRODUCTION

In 2015, structural and geotechnical engineers serving on the ASCE COPRI Ports & Harbors Committee formed a Task Committee for the Seismic Design of Bulkheads (SDB) to address the following industry considerations:

- Bulkheads are commonly used in seismically active areas.
- Bulkheads are getting larger and more complex with time.
- Many Ports and other agencies have adopted performance-based (deformation-based) design.
- Detailed, consistent guidelines for the performance-based seismic design of bulkheads are not available to practitioners.

The SDB Task Committee has evaluated published seismic design methods and is developing a Guidelines document for the seismic design of 2-D flexible bulkheads, commonly consisting of continuous sheet pile walls, or “combi-walls” (sheet pile and king pile systems). The Guidelines will be specific to the marine engineering industry where the bulkhead may constitute a quay/dock or part of a pile-supported pier or wharf system in the form of a cut-off wall or abutment. It is anticipated that after review and publication, the Guidelines will be re-drafted in the form of a standard, made available for review and input by port engineers, then eventually adopted for use in the upcoming version of ASCE COPRI Standard 61. Figure 1 shows a typical anchored bulkhead structure cross section. Figure 2 shows a photograph of the top of an anchored sheet pile bulkhead from the land side.

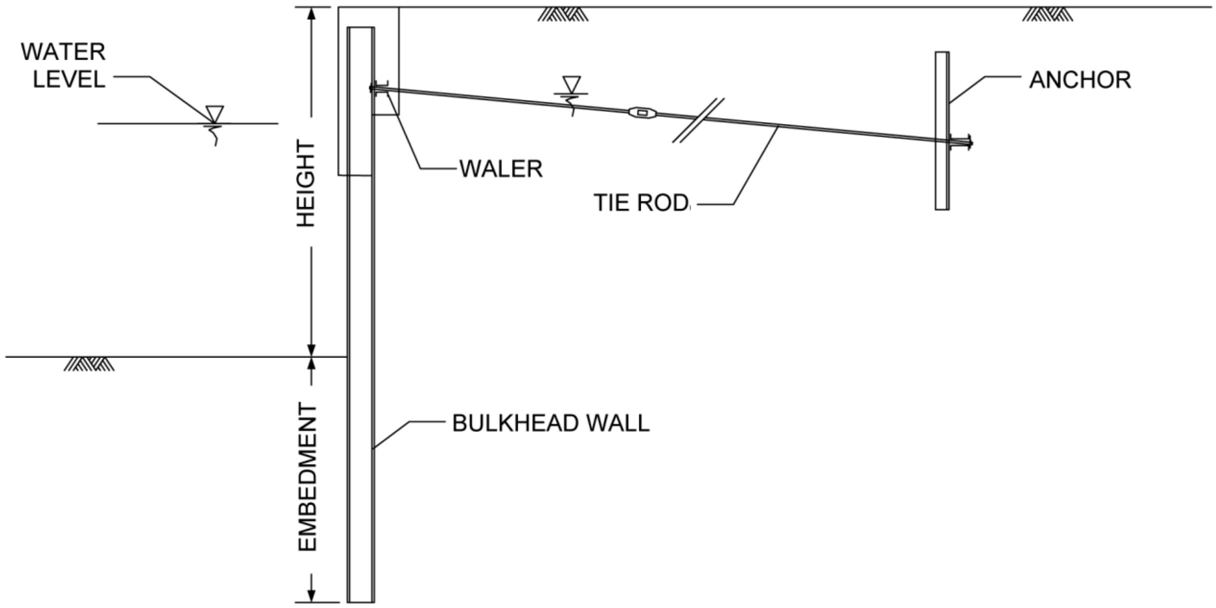


**Figure 1: Typical Anchored Bulkhead**

**Figure 2: Sheet Pile Wall and Combi-Wall (photos provided by Nucor-Skyline)**

This paper focusses on the geotechnical considerations and recommendations that are being developed for the Guidelines document. The recommendations are intended to focus on established analysis methods and reduce the historically inconsistent design approaches taken by the geotechnical community. Incorporation of deformation-based design necessitates that historic analysis methods are supplemented with (not necessarily replaced by) more sophisticated geotechnical and structural analysis methods. The Guidelines, in their current state of development, will include evaluation of planar, anchored bulkheads in uniform and layered soil environments. The evaluation of alternative bulkhead configurations will be addressed in the future. The Guidelines will include considerations for conditions with and without strength loss associated with cyclic loading.

The following sections provide a summary of the planned contents of the geotechnical section of the Guidelines document and the recommended geotechnical design approach. The SDB Task Committee continues to develop these contents; therefore, further study and evaluation is required to develop recommendations appropriate for anchored bulkheads.

## SITE CHARACTERIZATION

From the geotechnical perspective, the design of bulkheads, like other marine and near-shore structures, depends significantly on the site history, local geology, and the characteristics of the underlying soil and/or rock strata. Thus, a well-planned and implemented site investigation is essential for the development of an adequate site characterization on which to base the bulkhead design. For both existing facilities and new development, the site investigation should include the following components:

**Desk Study:** The goal of the desk study should be to compile available information for the site, including geological features, site history, topographic and bathymetric surveys, hydrologic information, and oceanographic information. For seismic design, the emphasis of the desk study should be identifying the geological features and subsurface conditions that will impact the design (e.g. faults, liquefaction, landslides, soft ground, etc.).

**Site Reconnaissance:** The purpose of the site reconnaissance is to observe first-hand the site conditions relative to layout, relief, surface soils, rock outcrops, presence of subsided or eroded areas, presence and condition of existing structures, general topographic features, tide levels and currents, apparent depositional processes, and other unusual or notable features. If the site conditions differ from the conditions surmised from the desk study, it may be necessary to perform new surveys or other studies to develop updated information and to determine the actual site conditions that may not be readily visible during the site visit. For example, if the work involves upgrade of an existing anchored bulkhead, some investigation will be required to observe the condition of the sheet piles and anchors per the ASCE Waterfront Facilities Inspection and Assessment Manual of Practice (ASCE, 2015).

**Subsurface Exploration Program:** The findings of the desk study and site reconnaissance should be used to develop the subsurface exploration program. The subsurface exploration is developed to satisfy two important goals; (1) provide sufficient explorations to define the extent of subsurface conditions (soil, rock, groundwater) that could affect the design, and (2) provide requisite in situ and laboratory data for the static and seismic performance analyses that are planned for the project. With respect to the latter, the extent and integration of the field and lab testing program will reflect the size and importance of the facility, the configuration and complexity of the proposed bulkhead, seismic load levels, and other project-specific factors. Site characterization should incorporate explorations on the land and water side of the bulkhead. While overwater explorations can be costly, it is in the Owner's best interest to have water side explorations to provide subsurface data for stability analyses and reduce risk of claims and changed conditions. The exploration program should include monitoring of water level fluctuations in foundation soil deposits over tidal and seasonal cycles.

**Testing and Evaluation of Soil Parameters:** Integrated field and laboratory testing should be performed as appropriate to support the anticipated methods of seismic analysis, which commonly range from standard General Limit Equilibrium (GLE) method(s) to 2D nonlinear seismic deformation analyses (NDA). Historically, seismic design of bulkheads has been accomplished through parameter correlations with field tests such as Standard Penetration Tests

(SPT) and Cone Penetration Tests (CPT). With the evolution and widespread adoption of performance-based seismic design for port waterfront structures, advanced numerical analyses based on soil-structure interaction (SSI) analysis are routinely used. Appropriate SSI analyses require substantial characterization of soil behavior – soil properties such as shear modulus, damping, and residual shear strength are required for a proper evaluation. The use of SPT or CPT data should be supplemented with testing that provides soil parameters across the range of deformation (i.e., shear strain, compression) that is anticipated on the project. Strain-dependent soil parameters must also account for the duration of seismic loading and potential for cyclic degradation of soil stiffness and strength. Soil laboratory tests such as cyclic triaxial shear, cyclic triaxial simple shear, and centrifuge can provide such insights into the soil behavior during a seismic event. The integration of suitably extensive field and laboratory testing programs improves the site characterization and the reliability of static and cyclic soil behavior, thereby reducing uncertainty that can lead to overly-conservative design. When the soil parameters are more reliable (through additional testing), it can lead to a more cost-effective bulkhead design. Table 1 provides field investigation and laboratory testing recommendations for seismic design of bulkheads.

**Table 1. Recommended Site Characterization Program for Seismic Design of Bulkheads**

| Characterization Method for Analysis Level or Type | Preliminary Siting/ Screening | Preliminary Design | GLE with Newmark | 1D Dynamic Site Response | Piles and SSI | DSSI or NDA |
|---|---|---|---|---|---|---|
| Field/In situ Investigations | | | | | | |
| Topography/Bathymetry | ● | ● | ● | ● | ● | ● |
| Soil Borings and Sampling | ● | ● | ● | ● | ● | ● |
| Piezometers | ● | ● | ● | ● | ● | ● |
| SPT/CPT | ● | ● | ● | ● | ● | ● |
| Shear Wave Velocity[1] | | ● | | ● | ● | ● |
| Laboratory Tests[2] | | | | | | |
| Index Properties | ● | ● | ● | ● | ● | ● |
| Consolidation[3] | | ● | ● | ● | ● | ● |
| Shear Strength[3] | | ● | ● | ● | ● | ● |
| Cyclic Resistance[3] | | | ● | ● | ● | ● |
| Post-Cyclic Stress-Strain[3] | | | ● | | ● | ● |

NOTES:
1. Shear wave velocity measurement is used to establish the trend of low-strain shear stiffness of soils with depth, which is a requisite input parameter in seismic analyses.
2. Recommended tests depend on soil type encountered and analysis level.
3. Performed on high-quality specimens of fine-grained soils.

## EARTHQUAKE GROUND MOTIONS

The Guidelines consider three earthquake performance objectives. These objectives and related geotechnical design recommendations discussed subsequently are summarized in Table 2.

**Table 2. Seismic Performance Requirements and Related Seismic Hazard Recommendations**

| Level | Performance Objectives | $k_h$/PGA | Return Period |
|---|---|---|---|
| Minimal Damage/ Limited Damage | ▪ Structure responds elastically<br>▪ Minor deformation and settlement<br>▪ No loss of serviceability<br>▪ No loss of material containment | 2/3 to 3/4 | Lowest (e.g. 72 years) |
| Controlled and Repairable Damage | ▪ Structure responds in ductile manner<br>▪ Limited inelastic deformation<br>▪ Damage to adjoining infrastructure caused by bulkhead response is repairable<br>▪ Repair can be completed within a few months | 1/2 to 2/3 | Intermediate (e.g. 475 years) |
| Life Safety Protection/ Collapse Prevention | ▪ No collapse occurs<br>▪ Structure is stable after earthquake<br>▪ Damage does not prevent egress<br>▪ Loss of material containment does not pose public or environmental hazards | 1/3 to 1/2 | Highest (e.g. 2500 years) |

$k_h$ = horizontal seismic coefficient; PGA = peak ground acceleration

Although example return periods are shown in Table 2 (based on ASCE 61-14), the earthquake ground motions should be developed based on seismic hazard levels defined in the governing code for the project.

To evaluate the seismic response of bulkheads, pseudo-static analyses are commonly used to simplify complex seismic soil structure interaction. Pseudo static analyses do not directly model time-dependent ground shaking – instead, they approximate the effect of the earthquake loading using the horizontal seismic coefficient ($k_h$). This coefficient is estimated as a fraction of the peak ground acceleration (PGA), and should be selected based on anticipated wall deformation, wall stiffness, wall height (wave scattering), and performance level. The Transportation Research Board's report NCHRP611 (Anderson et al., 2008) provides methods for estimating the $k_h$ value. For seismic design of bulkheads, the Guidelines will recommend that minimum $k_h$/PGA values generally fall within the ranges shown in Table 2. Higher $k_h$/PGA ratios are typically associated with lower seismic regions; stiffer and/or shorter walls, and lower deformations. The vertical seismic coefficient is typically neglected ($k_v$=0) for pseudo-static analyses (Anderson et al., 2008).

For projects where potential bulkhead damage has high cost or risk implications, dynamic soil-structure-interaction (DSSI) models should be considered. Dynamic analyses consider time history input and are generally performed using numerical modeling software. The Guidelines will not address selection or adjustment of time histories as this is sufficiently covered in existing publications (e.g., Anderson et al. 2008, NIST 2011, EERI 2014).

## LIQUEFACTION AND CYCLIC STRENGTH DEGRADATION

Soft, deformable soils and hydraulically-placed, initially loose sands are often present in the waterfront environment. The presence of these soils at the bulkhead site will necessitate evaluation of the potential for strength loss of soils (fine-grained cohesive soils, fine-grained transitional soils, and sandy soils) due to seismic loading. Cohesionless soils below water are often subject to liquefaction during ground motions, even at low amplitude (≥ 0.1 g). Liquefaction and strength loss increase lateral forces on the bulkhead system and decreases resistance at the bulkhead toe and at the anchor system.

Liquefaction can be evaluated using standard, semi-empirical procedures based on SPT or CPT data and estimated cyclic shear strengths associated with design ground motion levels and/or 2D numerical DSSI analyses (effective stress analyses). Liquefaction potential should be evaluated using the peak ground acceleration (PGA) corresponding to non-liquefied conditions. The PGA can be adjusted to account for the wave scattering (i.e., variation in average ground acceleration behind the bulkhead) due to the retained height of the soil. Procedures for adjusting the PGA due to wave scattering are documented in NCHRP 611 (Anderson et al., 2008). Analyses should consider:

- The potential for cyclic excess pore pressure generation resulting in the reduction of stiffness and strength in all soil types.
- The potential for triggering of liquefaction in sandy soils and cyclic degradation (i.e., accumulation of moderate to large shear strain) in fine-grained soils.
- The effect of local ground water fluctuations and sea level rise on the groundwater table for liquefaction triggering analysis.
- The effects of potential cyclic strength loss in land side and water side soils. Site specific laboratory test methods can provide additional information to evaluating cyclic strength loss and residual strengths.
- Seismically-induced settlement behind the bulkhead.

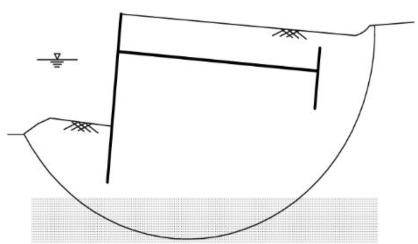

**Figure 3: Sketch of Global Stability Failure**

## GLOBAL STABILITY

Global stability analyses must be performed to evaluate the potential for land side and water side ground failures that impact the anchored bulkhead system. This is most commonly performed using General Limit Equilibrium (GLE) methods of analysis to estimate the margin of stability for the bulkhead. The analyses should be performed for conditions adjacent to the bulkhead as well as broader, deep-seated failures. The analysis results are used to define the minimum depth of the bulkhead piles to meet global stability requirements and to estimate seismic earth pressures (see subsequent discussion). The global stability analyses are typically performed for static, seismic (pseudo-static), and post-seismic conditions. These analyses can also provide information related to the need for and extent of ground treatment. Global stability analyses should consider:

- Appropriate $k_h$/PGA relationships under seismic loading for the project conditions and earthquake performance objective (see Table 2).
- Consideration of circular and non-circular (wedge-type) critical surfaces.