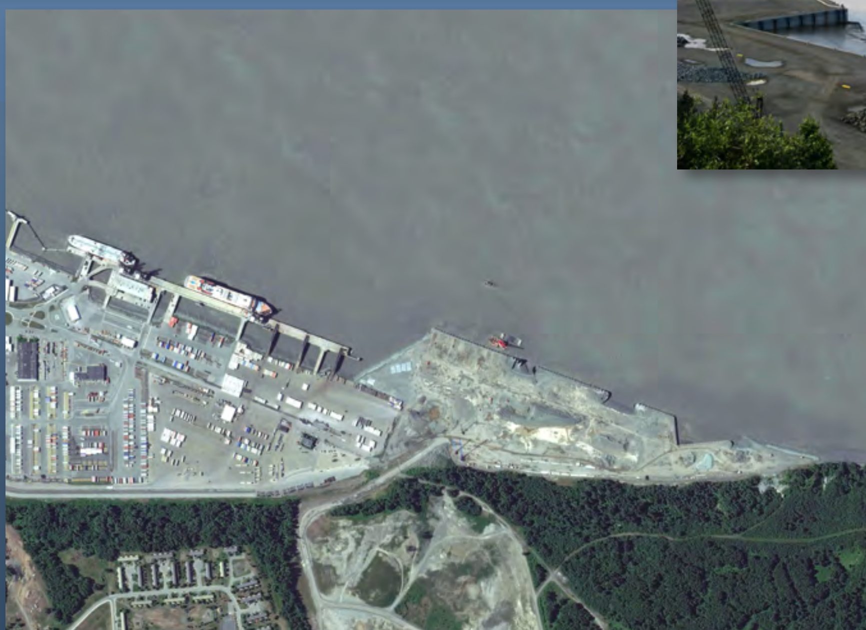
FINAL **SUMMARY REPORT**

Port of Anchorage
Intermodal Expansion Project
Suitability Study

February 14, 2013



Prepared by

CH2MHILL®

Prepared for

PORT OF ANCHORAGE
Alaska's Port • Alaska's Future

US Army Corps of Engineers®

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Port of Anchorage Intermodal Expansion Project Suitability Study

Prepared for

U.S. Army Corps of Engineers, Alaska District

February 14, 2013

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Acronyms and Abbreviations

2D	two-dimensional
3D	three-dimensional
AASHTO	American Association of State Highway and Transportation Officials
ADCP	acoustic Doppler current profiler
AIC	Alaska Interstate Constructors LLC
ALWC	accelerated low water corrosion
ASCE	American Society of Civil Engineers
BCF	Bootlegger Cove formation
BPF	blows per foot
CIUC	consolidated isotropically undrained compression
CIUE	consolidated isotropically undrained extension
CL	construction load
CLE	contingency level earthquake
cm	centimeter
cm/sec	centimeters per second
CPT	cone penetration test
CRR	cyclic resistance ratio
CSR	cyclic stress ratio
CU	consolidated undrained
cy	cubic yard
CyDSS	cyclic direct simple shear
D	dead load (CH2M HILL)
DL	dead load (PND)
DOF	degree of freedom
DOWL	DOWL HKM
DSHA	deterministic seismic hazard analysis
DSM	deep soil mixing
DSS	direct simple shear
E	earth pressure load
EA	environmental assessment
EAFB	Elmendorf Air Force Base
EOR	Engineer of Record
EPS	extruded polystyrene

EQ	earthquake load
ERDC	U.S. Army Engineer Research and Development Center
FLAC^{3D}	Fast Lagrangian Analysis of Continua in 3 Dimensions
FS	factor of safety
ft^3	cubic feet
ft/s	feet per second
g	acceleration of gravity
GAC	Municipality of Anchorage Geotechnical Advisory Commission
GLE	generalized limit equilibrium
GMPE	ground motion predictive equation
HS	hydrostatic load
IBC	International Building Code
ICRC	Integrated Concepts & Research Corporation, Inc.
ICE	ice load
IHA	Incidental Harassment Authorization
ITB	Invitation to Bid
JBER	Joint Base Elmendorf-Richardson
k	permeability
kcf	kips per cubic foot
km	kilometer
kPa	kilopascal
ksf	kips per square foot
ksi	kips per square inch
L	live load
LDCC	low-density cellular concrete
LL	liquid limit; live load (PND abbreviation)
LPT	large penetration test
LWL	landside water level
m	meter
M	earthquake magnitude
MARAD	U.S. Department of Transportation Maritime Administration
MC	Mohr-Coulomb
MCE	maximum considered earthquake
MHEA	maximum horizontal earthquake acceleration
MKB	MKB Constructors

ML	mudline
MLLW	mean lower low water
Mm^3	million cubic meters
MOA	Municipality of Anchorage
MOTEMS	Marine Oil Terminal Engineering and Maintenance Standards
mph	miles per hour
MPT	modified penetration test
mpy	mils per year
m/s	meters per second
MSE	mechanically stabilized earth
MSF	magnitude scaling factor MTR Marine Terminal Redevelopment
M_w	moment magnitude
NMFS	National Marine Fisheries Service
NE	North Extension
NEHRP	National Earthquake Hazard Reduction Program
NEPA	National Environmental Protection Act
NGA	Next Generation Attenuation
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
OCR	overconsolidation ratio
OCSP®	open-cell sheet pile
OLE	operating level earthquake
pcf	pounds per cubic foot
PGA	peak ground acceleration
PH	phreatic load
PHGA	peak horizontal ground acceleration
PI	plasticity index
PIANC	The World Association for Waterborne Transport Infrastructure
PIEP	Port of Anchorage Intermodal Expansion Project
PND	PND Engineers, Inc.
POA	Port of Anchorage
PSD	pile-supported deck
psf	pounds per square foot
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch

QA	quality assurance
QAP	Quality Asphalt Paving
S-waves	shear-waves
SCPT	seismic cone penetration test
SHANSEP	Stress History and Normalized Soil Engineering Properties
sig4	FLAC^{3D} sigmoidal model
SLD	service load design
SPT	standard penetration test
SWL	seaside water level
TOTE	Totem Ocean Trailer Express
URS	URS Greiner Corporation
USACE	United States Army Corps of Engineers
V_s	shear wave velocity
WBB	Wet Barge Berth
West	West Construction
W_{HD}	hydro-dynamic water load
W_p	phreatic water load

Executive Summary

Background

The Port of Anchorage (POA) provides critical infrastructure for the citizens of Anchorage and most of Alaska, since the majority of goods shipped to and from the state pass through the POA. The existing infrastructure at the POA was largely built in the 1960s and is reaching the end of its useful life. The POA Intermodal Expansion Project (PIEP) is intended to provide new berthing facilities for the shipping companies calling at the POA.

An Open Cell Sheet Pile® (OCSP®) design was selected for the project based on representations by the designer, PND Engineers, Inc. (PND). However, as the project moved into the construction phase, the suitability of this OCSP® system for the POA was called into question, primarily because of problems encountered while installing the OCSP®.

CH2M HILL was contracted in 2011 by the United States Army Corps of Engineers (USACE) in partnership with the Maritime Administration (MARAD), the POA, and the Municipality of Anchorage (MOA) to conduct an independent suitability study of the OCSP® system used in the North Expansion area of the ongoing PIEP. For this suitability study, CH2M HILL reviewed the project information for the PIEP and conducted independent engineering analyses to evaluate the performance of the OCSP® system. The primary intent of this independent review was to determine whether the basis of design for the OCSP® system was consistent with standard design requirements at the time, whether the as-built OCSP® system will meet design requirements for gravity and seismic loading, and whether alternative design or construction methods need to be considered.

The engineering analyses for this suitability study included assessments of seismic hazards, hydrological considerations, geotechnical conditions, structural performance, and constructability. Geotechnical and structural evaluations included analyses based on conventional design methods and on numerical modeling of the soil-structure system.

Overview of Constructed Facilities

Figure ES-1 shows the limits of the projects constructed to date for the PIEP. Collectively they are referred to as the North Expansion Projects, and individually they are referred to as the Dry Barge Berth (DBB), Wet Barge Berth (WBB), North Extension 1 (NE1), and North Extension 2 (NE2).

Each of these projects involves facilities with different wall heights because of varying planned dredge depths along the alignment. The different wall heights are important to understand, as the results of this suitability study are directly related to the wall heights. Figure ES-2 shows the wall heights for the DBB, WBB, NE1, and NE2, with the wall height defined as the distance from the sea bottom elevation to the finished ground elevation. The actual sheet piling lengths differ from the wall height to account for sheet piling embedment depths at the bottom and a steel or concrete cap at the top. Figure ES-3 shows the cell numbering scheme and the berth limits by cell number in the North Expansion area.

The suitability of the OCSP® system depends on the internal and global stability of the OCSP® system for static (or gravity) and seismic loading. It is important to understand the difference between “internal” and “global” stability of the OCSP® system. Figure ES-4 from PND shows the system mechanics of the OCSP® system. Internal stability is the stability of the structure itself and the material contained therein. Global stability takes into consideration factors physically located outside of the structure, but that will have an impact on its stability. For example, an analysis of global stability includes the Bootlegger Cove Formation (BCF) clay that lies outside of and under the actual structure, where an internal stability analysis would not. Static loading is generally described as the gravity loads encountered under normal working conditions, as well as during construction. Seismic loading is the additional loading that occurs during an earthquake.

A key issue for the PIEP is confirming acceptable performance under future seismic events. A multi-level seismic design approach is used for the OCSP® system. The multi-level approach involves either two levels or three levels of seismic ground shaking, as follows:

- **Operational Level Earthquake (OLE):** All facilities are to be designed for OLE ground motions. These ground motions are defined as having a 50 percent probability of being exceeded in 50 years. This design ground motion corresponds to an average return period of 72 years.
- **Contingency Level Earthquake (CLE):** All facilities are also to be designed for the CLE ground motions. These ground motions are defined as having a 10 percent probability of being exceeded in 50 years. This design ground motion corresponds to an average return period of 475 years.
- **Maximum Considered Earthquake (MCE):** The NE1 and WBB berths (essential post-earthquake facilities for emergency point of entry for receiving relief supplies and goods) are designed for the MCE ground motions. These ground motions are defined as having a 2 percent probability of being exceeded in 50 years (or ground motions with an average return period of 2,475 years).

The use of the MCE for design of a portion of the OCSP® system is based on a resolution from the Port of Anchorage Seismic Design Committee on June 29, 2004. The panel recommended examining and evaluating the physical and economic feasibility of “designing at a minimum, one berth to withstand a seismic event greater in scope than a Level 2 Contingency Level Earthquake in order to provide an emergency point of entry for goods and supplies necessary to support the community.” Based on this recommendation, the MOA and the POA agreed that two areas (WBB and NE1) within the North Expansion should maintain services under a seismic event “greater in scope than the CLE,” in keeping with lifeline earthquake engineering concepts. The MCE was selected by PND as an appropriate event to meet this requirement. The spectral acceleration values from the MCE event would be roughly 60 percent higher than those for the CLE, and more than three times the OLE. The areas identified for the Essential Facility are shown in Figure ES-1.

Geotechnical Design Concerns

The focus of the suitability study involves assessing the stability of the OCSP® system during static and seismic loading. This assessment is made in terms of factors-of-safety (FS), which represent the margin of safety that the system has during static and seismic loading. A facility is judged as acceptable if it meets or exceeds FS values set or generally accepted by building codes, regulatory agencies, or standards of engineering practice.

Based on the geotechnical analysis carried out during the suitability study, a series of graphs were developed that compared the original designer’s FS calculations with those developed by CH2M HILL in the suitability study. These graphs were presented to the MOA Assembly and the MOA Geotechnical Advisory Commission on November 9, 2012. The graphs show not only the original designers FS determinations, but they also illustrate the effect of changing input design values as determined by this study.

The following observations can be made from these graphs for the existing North Expansion projects regarding their existing conditions and global stability:

- **Dry Barge Berth (DBB).** The DBB has been successfully constructed and has adequate FS values for structural and global stability. Figure ES-5 shows the original designer’s FS=2.06 as compared with the study value of 1.49. The design criteria under static loading require a minimum FS=1.5. The difference between the original designer’s FS and the study FS is attributed to the difference in the shape of the slip surface, ground water table, 3-zone soil model, and live load. The calculations were performed with each variable isolated so that the sensitivity to FS for each variable could be readily observed. Although the study determined the FS is much lower than the original designer’s, it meets the minimum FS. The DBB is currently in use by the POA for transfer of barge cargo, and no further action is required.
- **Wet Barge Berth (WBB).** The WBB currently has major defects in the installed sheet piling. Most of the problems stem from encountering large rock and stiff clay during sheet piling installation. Many of the sheets

are damaged beyond repair. Additionally, the FS for static global stability is not adequate for the WBB. Figure ES-6 shows the original designer's FS=1.64 as compared with the study value of 1.22. The design criteria under static loading require a minimum FS=1.5. Again, the difference between the original designer's FS and the study FS is attributed to the difference in the shape of the slip surface, ground water table, 3-zone soil model, and live load. Like the evaluations for the DBB, the calculations were performed with each variable isolated so that the sensitivity to FS for each variable could be readily observed. The 63-foot wall height in the WBB is over twice the 26-foot height of the DBB, and this contributes directly to the lower factors of safety.

- **North Extension 1 (NE1).** The NE1 had some damaged sheet piles and defects that have been repaired according to the original designer, as referenced in a letter to ICRC from PND on September 23, 2011. However, this section of the OCSP® is about three times as high as the DBB as shown in Figure ES-2 and has an even lower FS for static global stability than the WBB. The FS for static global stability is 1.1, which is significantly below the required FS=1.5. Again, the difference between the original designer's FS and the study FS is attributed to the difference in the shape of the slip surface, ground water table, 3-zone soil model, and live load.
- **North Extension 2 (NE2).** Only about 800 feet of the NE2 was constructed prior to suspension of construction. The NE2 has had the most dramatic construction defects, consisting of large sinkholes opening behind the sheet piling. The cause of the sinkholes is linked to sheet piles "out-of-interlock" below the water line, creating an opening for saturated backfill to easily pass through the openings. Underwater inspections and forensic explorations have documented the broken interlocks. The FS for static global stability is 1.13, which is significantly below the required FS=1.5. Figure ES-7 shows the original designer's calculated FS=1.51 as compared with the study value of 1.13. Again, the difference between the original designer's FS and the study FS is attributed to the difference in the shape of the slip surface, ground water table, 3-zone soil model, and live load. The calculations were performed with each variable isolated so that the sensitivity to FS for each variable could be readily observed. This section has an 89-foot wall height as shown in Figure ES-8, which is over three times the height of the DBB.

The discussion above focused on static global stability for simplicity. As would be expected with low FS for static conditions, the FS calculated in the study for seismic loading is even lower.

1964 Alaska Earthquake and the Bootlegger Cove Formation Clays

Any vital infrastructure construction project in the Anchorage area should be cognizant of the large-scale damage that was triggered by the 1964 Alaska earthquake. There were major ground movements at Turnagain Heights, L Street, Government Hill, Fourth Avenue, and other areas in Anchorage. A great deal of effort in this suitability study was focused on understanding the 1964 Alaska earthquake and how lessons learned from it should be applied at the POA. The 1964 Alaska earthquake also provides a useful reference point, as many Anchorage citizens remember this event.

The ground movements in Anchorage during the 1964 Alaska earthquake varied from a few inches to as much as 18 feet in the case of Fourth Avenue. The earthquake totaled 4 to 6 minutes of strong ground shaking, and the level of ground shaking was estimated to be 0.15 to 0.20 times gravity (0.15g-0.20g). This is particularly relevant to the POA because the design criteria require essential facilities at the POA to be able to receive freight goods after a maximum considered earthquake (MCE), which has an estimated peak ground acceleration of 0.39g. Note that the MCE is twice as large as the ground accelerations estimated to have been experienced in the 1964 Alaska earthquake.

In contrast to the major slides in the higher areas around Anchorage that generally occurred along bluffs and steep slopes, the POA facilities survived the 1964 Alaska earthquake with minor damage. The facilities were almost new at the time, having been constructed in the early 1960s, and consisted of a pile-supported wharf and relatively shallow fills to create the backlands. Both the existing pile-supported wharf and the shallow backlands fill are substantially lighter in weight than the OCSP® bulkhead fill that has been built for the North Expansion; recall that the wall height of the NE2 section is 89 feet.

In addition to bluff height and steep slopes as drivers of major slides, the large ground movements during the 1964 Alaska earthquake are attributed to the BCF clay that underlies the Anchorage area, including Fourth Avenue and the POA. These clays experienced large strength loss and underwent large movements during ground shaking. To discover whether the BCF clays under the POA could have the same behavior as the clays under Fourth Avenue during the 1964 Alaska earthquake, BCF clay samples were taken from the POA North Expansion area and sent to the University of Illinois for testing in a ring shear apparatus that was previously used to characterize the Fourth Avenue BCF clays. The results of the testing indicate that the BCF clay at the POA could behave similar to the BCF clay underlying Fourth Avenue. This is a concern for the North Expansion because the wall height is similar to the height of Fourth Avenue above the clay formation. Figure ES-9 shows an overlay of the OCSP® wall on a cross section of Fourth Avenue. This overlay illustrates the general scale of the OCSP® wall in relation to the similar geology of Fourth Avenue.

Structural and Life-cycle Analyses

The long-term structural integrity of the PIEP is another component of the evaluation. In this case, the integrity deals with the strength of the structure and the potential changes as the structure ages, mainly as a result of corrosion. Structural analyses based on conventional methods were conducted for the as-built condition, without consideration for construction deviations such as broken interlocks. The results show that the FS values for cell internal stability (that is, interlock strength and tailwall pullout) are satisfactory for both static and seismic load cases, assuming that the OCSP® system is constructed without defects. However, when the “global stability” is taken into consideration, only the DBB meets the static and seismic criteria initially established for the project.

Additional conclusions from the structural evaluation are as follows:

- Damage occurred during OCSP® sheet pile installation. This damage resulted in sheets being out of interlock and sheets with inadequate penetration. This damage affects the stability of some sections of the OCSP® system, and it results in zones of weakness that could lead to extensive damage during a large seismic event. These failures could be particularly significant if they occur in a facewall sheet or at the forward segment of the tailwall near the wye connection.

- Life-cycle performance relies on the OCSP® corrosion protection system. This system includes galvanizing and an impressed current cathodic protection system. Although protected, corrosion will still occur with time, and this corrosion will result in loss of structural capacity. Estimates of the reduction in wall thickness after 50 years suggest reduction of structure thickness. FS values for tension of the sheets at the highest wall sections of the North Extension could result in reduction of the FS to 15 percent under the design criteria near the end of the 50-year design life. Where the wall heights are lower, such as the Dry Barge Berth, the FS is adequate at 50 years. Regular inspection and maintenance will be essential for maximizing service life. Accelerated low water corrosion (ALWC) is present at the existing POA facilities, and it could further reduce the FS in localized areas if not controlled by cathodic protection.

Hydrological Review

Hydrological issues were studied including currents, sedimentation, scour, and ice forces that will impact the PIEP during construction and operations. Conclusions from the hydrological review are as follows:

- The location of the OCSP® system in Knik Arm could result in accelerated sedimentation in some locations. The POA and USACE will need to consider this potential when planning future maintenance dredging operations

- A potential exists for localized scour. This scour could occur at the base of fender piles and could be on the order of 5 to 7 feet. Propeller wash could also increase scour depths in localized areas. This potential needs to be monitored as part of future maintenance operations, and where needed, mitigated through use of scour protection systems.

- The location of the new berths will put the vessels in faster currents requiring additional mooring lines and likely requiring the use of larger or more tugs.
- Ice loading is an important design consideration because ice loads result in localized impact loads to the face of the OCSP® system, and they result in increased mooring forces to vessels. Ice loading on the moored vessels will be significantly greater than at the existing berths. Additional mooring lines will be required when large ice floes are present.

Constructability Assessment

The OCSP® system represents a creative approach to wharf construction and has been found to be very successful at numerous locations. However, the POA site has a number of environmental and geological conditions that make it a particularly difficult site. A number of issues related to the construction of the OCSP® system were identified as potentially contributing to the construction problems that have been observed. These observations include the approach taken to install the OCSP® walls, as well as the difficult environmental conditions such as restricted work hours and extreme tidal conditions. These also contributed to the past difficulties in constructing the OCSP® system at the POA.

Additional conclusions are as follows:

- Constructing the taller sections of the OCSP® structure from the dike on the land side of the wall appears to have been one of the main causes of construction problems. The fundamental issue lies with unbalanced soil pressure on the OCSP® wall, which prevents the sheet piling from driving straight (Figure ES-10).
- The use of large rock to stabilize the dike slopes in earlier phases of the project appears to have resulted in subsurface obstructions that were encountered during driving of the sheet piling, even when the contractor was required to remove these rocks. Evidence clearly shows that the sheet piling cannot be driven on alignment through rocks. This was particularly evident in the WBB.
- The listing of the Cook Inlet beluga whales and the associated permit conditions severely limited the time available for pile-driving. These permit conditions will hamper any future pile-driving operations at the POA. OCSP® construction is pile-driving-intensive, and any future construction of the system should evaluate whether there is sufficient time allowed to complete the work.
- Construction of an OCSP® system in this extreme environment requires finesse and experience. To lower the risk of installation problems, any additional OCSP® work at the POA must include a contractor selection process that ensures that a very experienced OCSP® construction contractor is selected. However, even with an experienced contractor there is still the risk that interlocks will be damaged
- The design plans for future OCSP® work must include a workable method of construction for the contractor to perform. Any future methods for construction must be thoroughly vetted with the construction industry to ensure they are constructable prior to the tendering of the contract.

Overall Conclusions

This suitability study led to the following overall conclusions regarding suitability and conditions of the existing structures and recommended next steps:

- **Suitability.** Installed properly, only the DBB meets the original FS criteria established for the project. The WBB, NE1, and NE2 do not meet the original static or seismic criteria when the overall global stability of the structure is taken into consideration.

- **Condition of Existing Structures.** The installation has numerous defects. The WBB and NE2 are irreparable from a construction perspective, and if they could be repaired they would not have the necessary FS for global stability. NE1 has been repaired to acceptable construction conditions, according to the original designer; however, it too does not have the necessary FS values originally established for the project.

In essence, then, with the exception of the DBB, which is currently being used by the POA, the North Expansion projects need to be reconstructed using a suitable method.

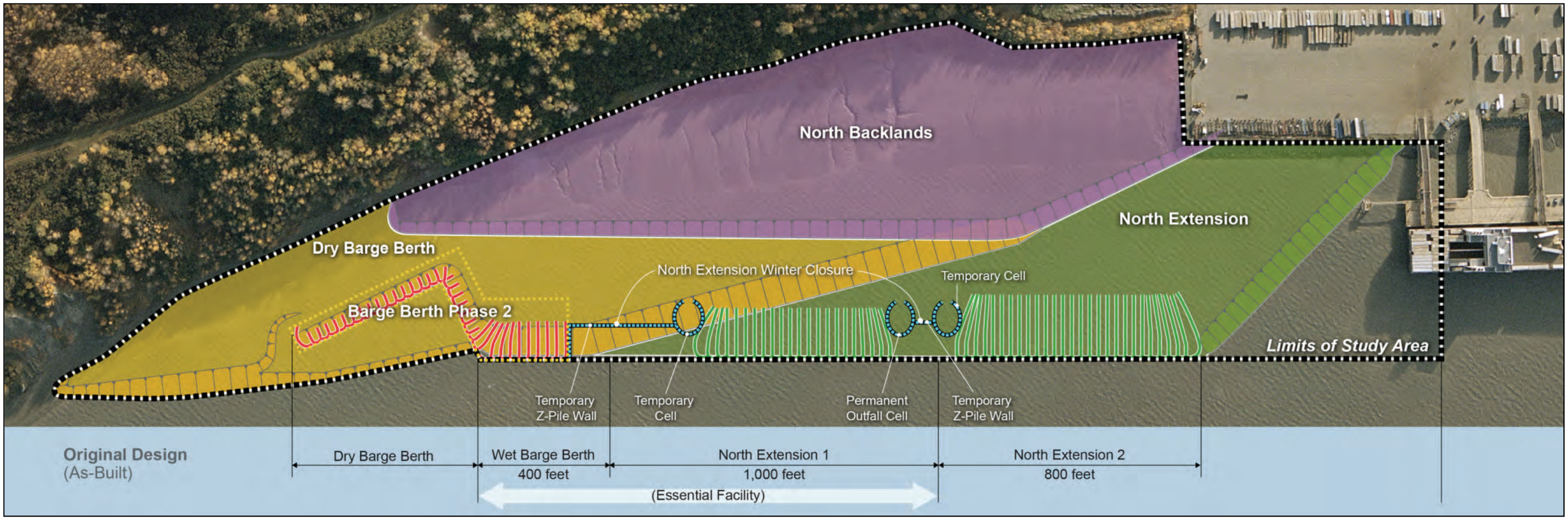


Figure ES-1. North Expansion Projects

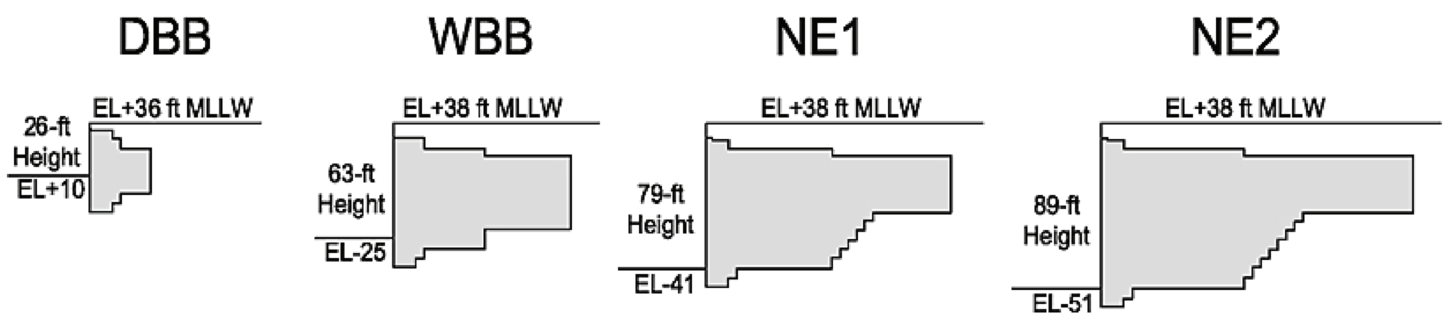


Figure ES-2. Wall Heights

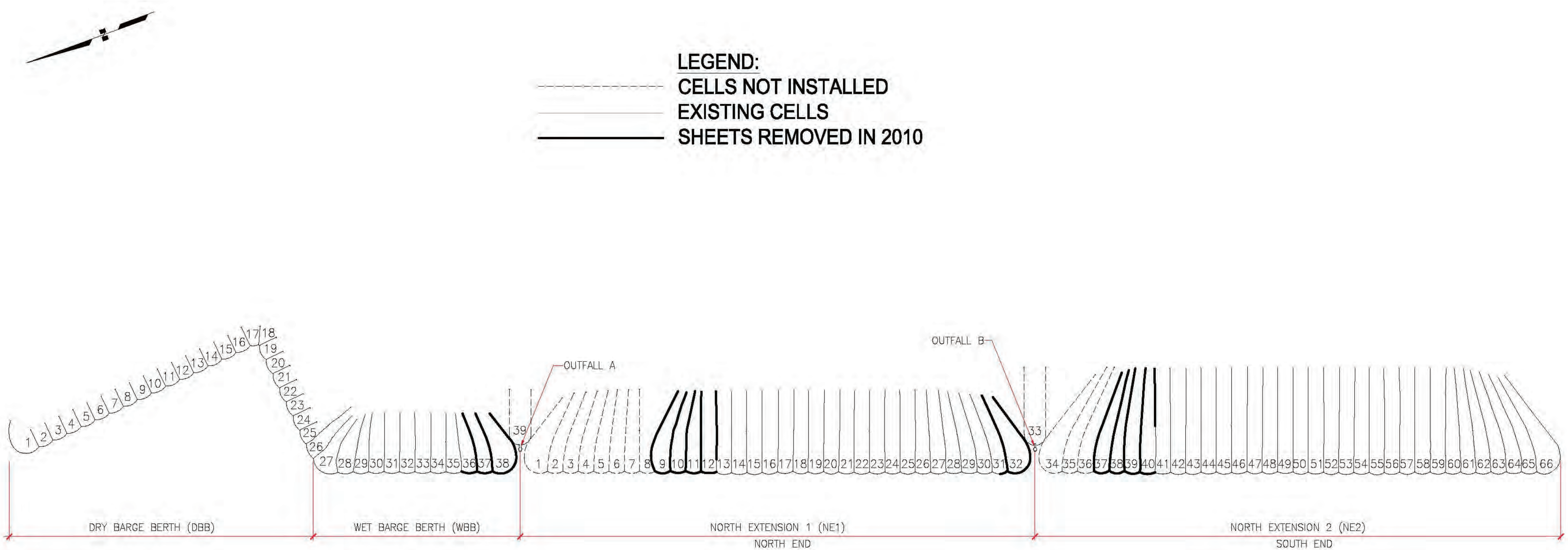


Figure ES-3. Cell Numbers in Barge Berth and North Extension Areas (Source: ICRC, 2010)

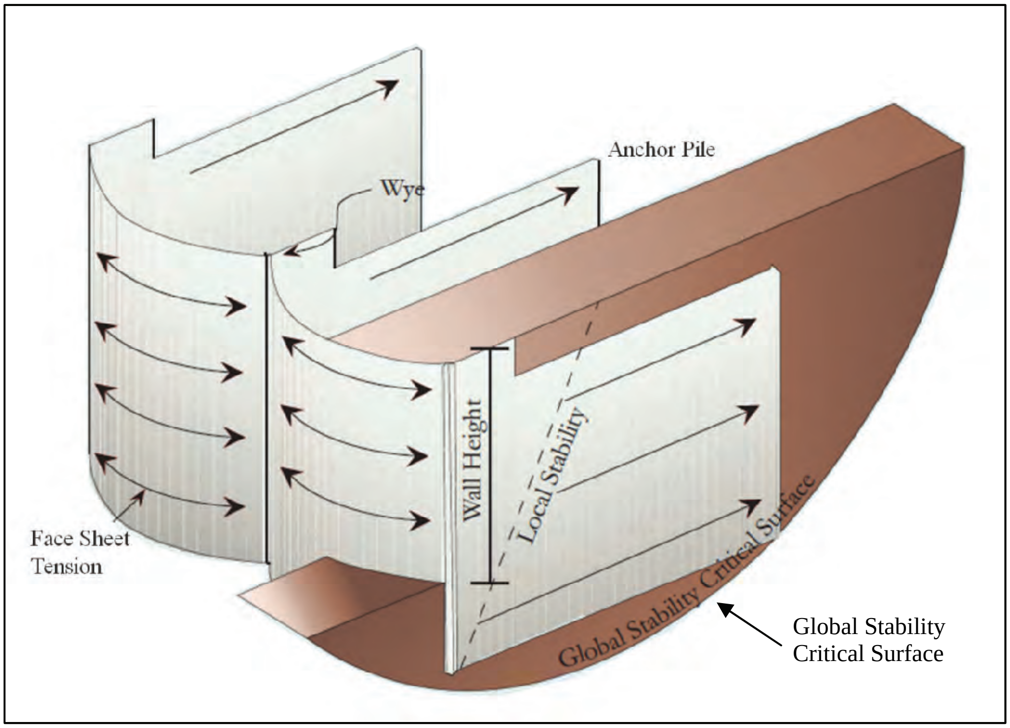
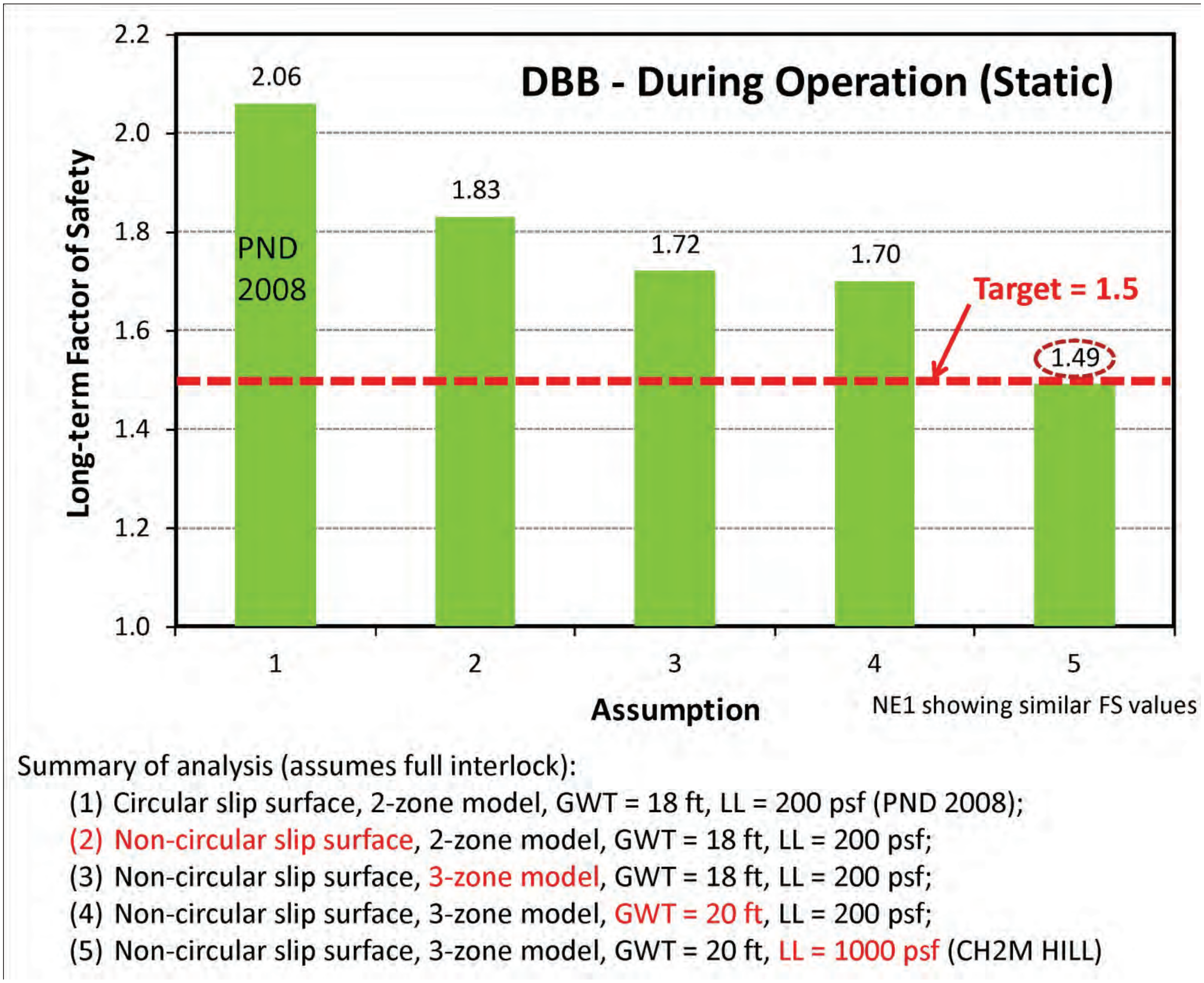
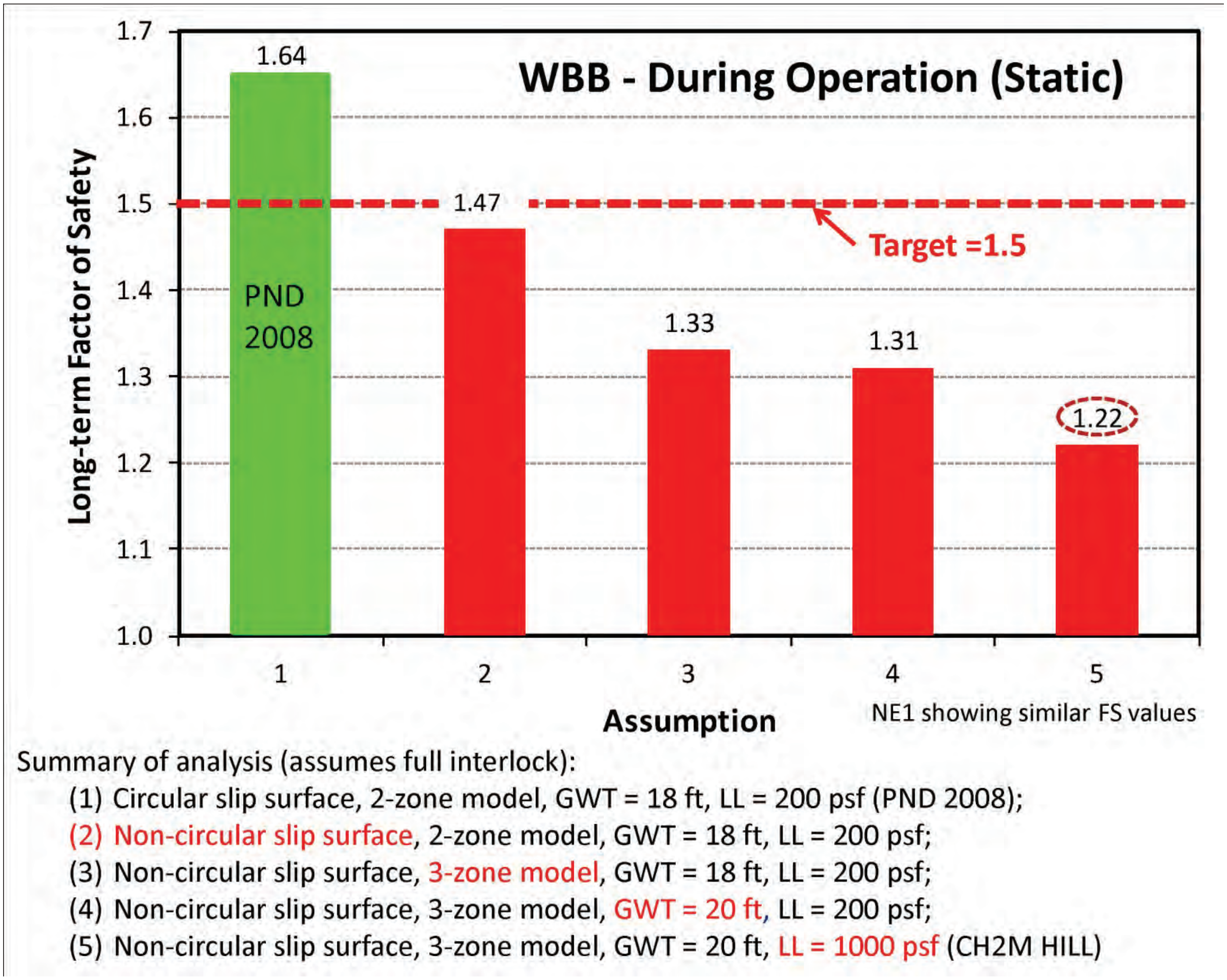


Figure ES-4. OPEN CELL® Sheet Pile (OCSP®) System Mechanics (PND, 2012)



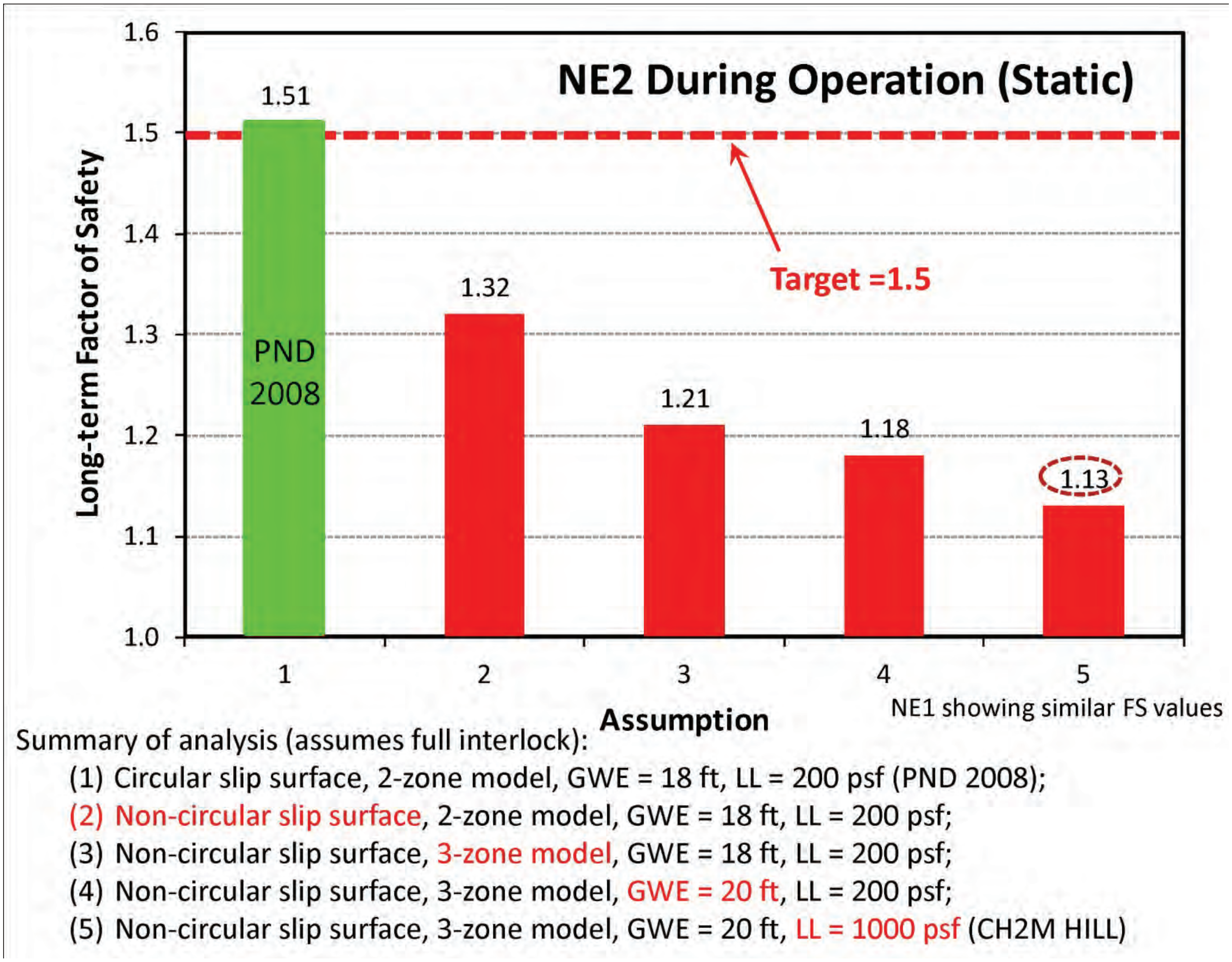
GWT = ground water table
LL = live load
psf = pounds per square foot

Figure ES-5. Factor of Safety of Dry Barge Berth during Operation



GWT = ground water table
LL = live load
psf = pounds per square foot

Figure ES-6. Factor of Safety of Wet Barge Berth during Operation



GWE = ground water elevation
LL = live load
psf = pounds per square foot

Figure ES-7. Factor of Safety of North Extension 2 during Operation

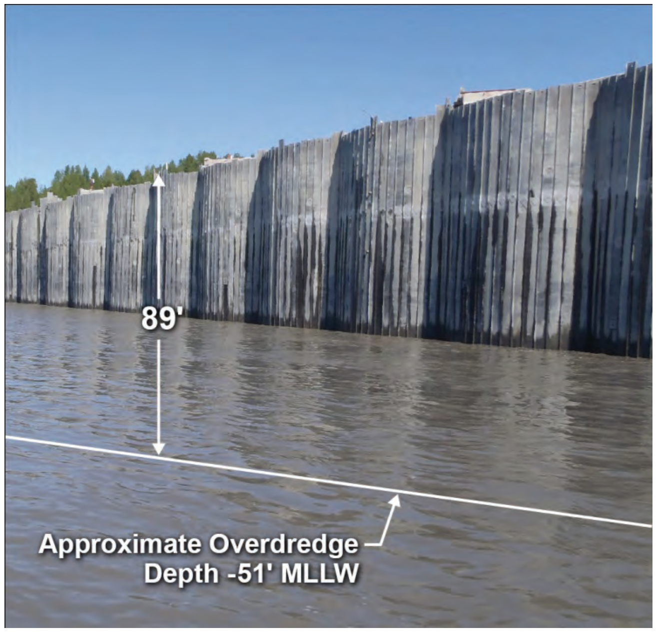


Figure ES-8. North Extension 2 Wall Height

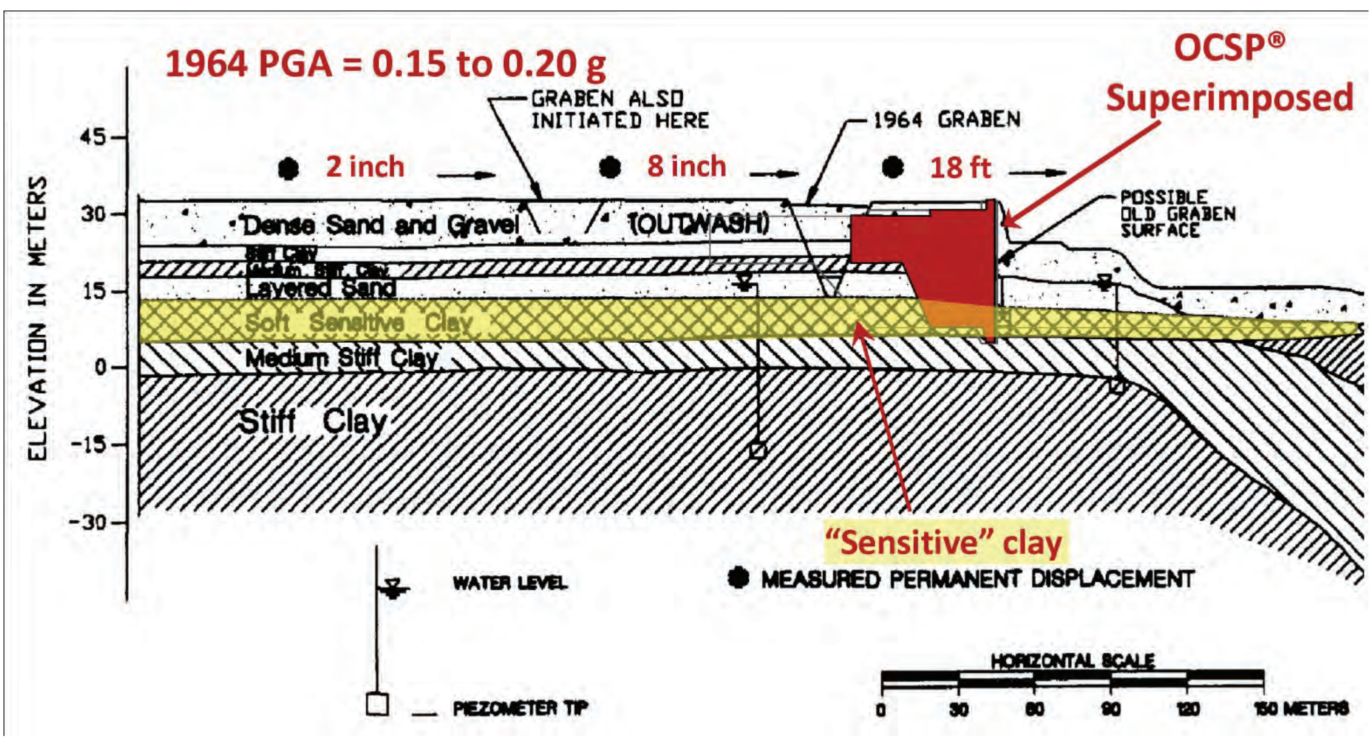


Figure ES-9. Comparison of Fourth Avenue and North Expansion Area Heights above BCF Clay (Adapted from Shannon and Wilson, 1964)

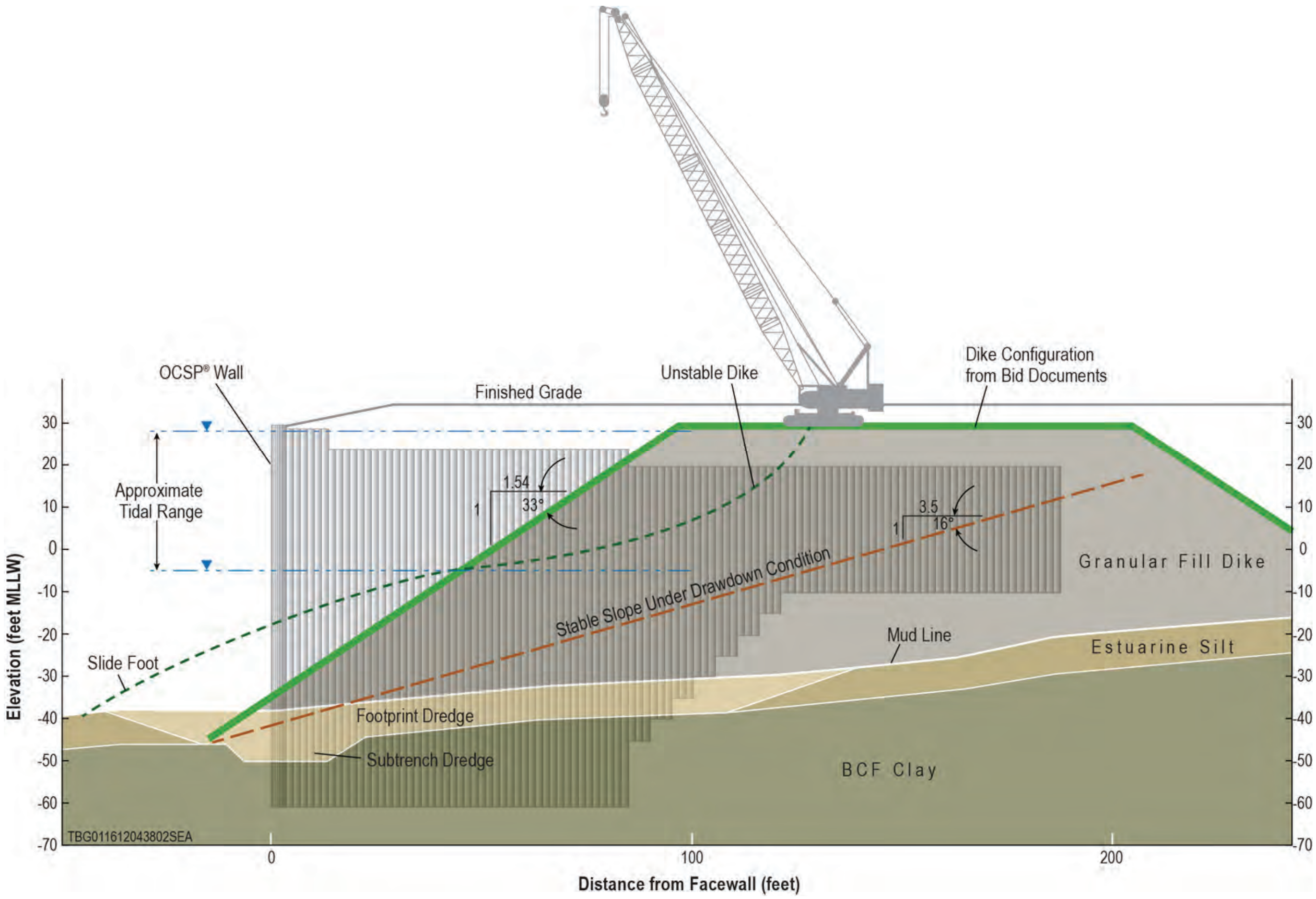


Figure ES-10. Source of Unbalanced Soil Pressure on the OCSP® Wall (Section Similar to Section G-G)

SECTION 1

Introduction

CH2M HILL was contracted by the United States Army Corps of Engineers (USACE) to conduct an independent suitability study of the OPEN CELL® sheet pile (OCSP®) system used in the North Expansion area of the ongoing Port of Anchorage (POA) Intermodal Expansion Project (PIEP). The suitability study included an analysis of the existing seismic, hydrologic, geotechnical, structural, and design conditions used in the current design of the OCSP® system and an independent assessment of the expected performance of the OCSP® at the current site based on the review of existing site conditions and anticipated operational and environmental loads. Constructability of the present design was also evaluated. This report provides background information for the assessment of the OSCP® system, summarizes the results of the CH2M HILL's analyses, and then gives recommendations regarding the general suitability of the OCSP® system for the site and loading conditions, changes to contracting methods and construction practices, and future testing and analysis efforts that should be considered to address the study limitations. This report is not intended to be a design document, nor is it an effort to provide an analysis of the project for purposes of supporting potential litigation, pursuant with the terms of the USACE contract. The reader is hereby notified that this document along with the supporting analysis should not be considered adequate for purposes beyond those expressly defined herein.

1.1 Project Authorization, Scope, and Approach

This report was prepared under the terms of the contract between CH2M HILL and USACE dated November 14, 2011. The contract authorizes CH2M HILL to provide geotechnical, civil, structural, corrosion, coastal, and construction engineering services associated with the PIEP in general accordance with the scope defined in USACE Delivery W912PP-09-D-0016 Task Order No. ZJ02. Appendix A provides a summary of the scope of work for Phases 1 and 2 of the project. The relationship of this study to the design and construction efforts is shown in Figure 1.1-1.

The initial scope of work described above was limited to use of information that had been collected by the original design team during design and construction of the OCSP® system. This existing information was used by CH2M HILL to assess stability of the OCSP® system for gravity and operational loads and for seismic loading. This initial phase of the project included meetings with POA and USACE staff, reconnaissance of the project site, and meetings with contractors performing work on the project to better understand the technical and administrative constraints that could have affected the approach taken by the original design team.

The approach taken for this suitability study included very limited interaction with the original designer. Meetings were held on two occasions to confirm that all documents prepared by the original designer of the PIEP had been received by CH2M HILL. During these meetings, the organization of project documentation was reviewed. The contacts with the original designers were limited to these two meetings and a transfer of original design documents to CH2M HILL in an effort to develop an independent opinion regarding project suitability. The USACE wanted CH2M HILL to take this approach to maintain as much transparency as possible in the review process.

During CH2M HILL's project review and evaluation, a need was identified for additional as-built information on the granular backfill used to construct the OCSP® system and on the Bootlegger Cove Formation (BCF) clay soils below the granular backfill. This need was discussed with the USACE, POA, and U.S. Department of Transportation Maritime Administration (MARAD), and led to two additions to the original scope of services:

- **USACE Delivery W912PP-09-D-0016 Task Order No. ZJ01 - Due Diligence 1**. This program included a limited post-construction field exploration program to evaluate backfill conditions as well as the elevation of the transition from backfill to the underlying native soils. This program was initiated in February of 2012. Laboratory tests on samples collected during the exploration program were completed in March 2012. Summary reports for the field exploration and laboratory testing programs are included in Appendix D.

- **USACE Delivery W912PP-09-D-0016 Task Order No. ZJ01 - Due Diligence 2**. This program included an additional post-construction field exploration program to collect high-quality intact soil samples from the BCF for subsequent specialized static, cyclic, and large displacement laboratory testing. The program was initiated in May of 2012. Laboratory tests on samples collected during the exploration program were completed in September of 2012. Summary reports for the field exploration and laboratory testing programs are included in Appendix D.

1.2 Project Description and Objectives

The project site is the North Expansion area, located at the POA north of downtown Anchorage, Alaska. The center for the project site is at approximate longitude -149.883 degrees west and approximate latitude 61.249 degrees north. Figure 1.2-1 shows the project location. The Phasing Plan for the PIEP is shown in Figure 1.2-2. The North Expansion area consists of a number of different projects, which are the focus of this study, as defined in Figure 1.2-3. The following subsections provide background information about the PIEP, including a brief summary of the project history, a description of the OCSP® design concept, information about use of the OCSP® system at the POA, and objectives of the suitability study.

1.2.1 Project History

The Port of Anchorage provides critical infrastructure for the citizens of Anchorage and all of Alaska, since the majority of goods shipped to and from the entire state pass through the POA. Uses include supplying fuel to Joint Base Elmendorf-Richardson and Ted Stevens Anchorage International Airport.

The existing POA infrastructure was largely built in the 1960s and is reaching the end of its useful life. The Port of Anchorage Master Plan (VZM Transystems, 1999) outlined steps that the POA must take to meet increasing demands, including the possibility of doubling its capacity (circa 2002) by the end of the year 2020. Recognizing the importance of the POA to the Anchorage area and the state, studies were initiated in the early 2000s (such as Tryck Nyman Hayes, Inc., 2002) for replacement or upgrading of the existing facilities.

Key findings from the POA Intermodal Marine Facility design study report (TNH, 2002) included the need for replacement or upgrade of the existing POA to provide the following:

- A new multipurpose dock designed to support existing users, accommodate underserved users, stimulate local military and regional development, and provide overall transportation system improvements
- A dock that is wider than the existing dock, configured for new and larger cranes, and designed for an increase from -35 to -45 foot draft
- Infrastructure and facilities that would keep pace with increasing commercial ship sizes and changing cargo-handling technology
- Potential use by cruise ships industry
- Use by the Department of Defense for rapid deployment of personnel and equipment to Joint Base Elmendorf-Richardson as well as transit to the Pacific Rim region and elsewhere, requiring the use of about 2,000 feet of dock face, about 16.5 acres of upland staging, and access to rail cars

The 2002 Intermodal Marine Facility study identified a new pile-supported wharf located south of existing Terminal 1. The wharf would have a length of 1,350 feet, a width of 120 feet, and be capable of servicing container cargo, dry bulk cargo (primarily cement), the military, project-specific cargo, and cruise ships. Provisions for 100-foot-gauge cranes were included in the plans. The initial dredge depth was -35 feet with a planned dredge depth of -45 feet and provisions for a maximum dredge depth of -55 feet. The plans also included a 1,550-foot-long petroleum, oil, and lubricants (POL) extension with two POL terminals. A seasonal smallcraft float was included in the concept, and a cellular sheet pile structure, behind the pile-supported cargo wharf, would provide a transition from the marginal wharf to the backlands.

Subsequent to the completion of the TNH study in 2002, an alternate concept involving an Open Cell® Sheet Pile (OCSP®) concept was proposed by PND Engineers (PND) of Anchorage, Alaska. The OCSP® is a propriety system patented by PND. It involves a sheet pile concept backfilled with granular soil, rather than a pile-supported wharf. Flat sheet piles are driven on a radius to form the face of the wharf, and tailwalls extend normal to the face at approximately 30-foot intervals to provide support for earth forces developed on the face. The OCSP® system had been used at over 70 other locations in Alaska, and had recently been constructed at Port MacKenzie, almost directly across Knik Arm from the POA. Although the OCSP® system proposed for the PIEP would require sheet pile lengths of over 90 feet in some areas, exceeding the height of the OCSP® system at Port MacKenzie by nearly 50 percent, the concept was represented as having significant cost savings to the POA, as the construction was limited to the installation of sheets piles and extensive use of locally available fill.

Beginning in 2002, a series of geotechnical field explorations, laboratory tests, and geotechnical and structural design studies was initiated to evaluate the feasibility of the OCSP® system at the POA. These studies were conducted by Terracon Consulting Engineers and Scientists (Terracon), PND, Lachel & Associates, and Moffatt & Nichol. The geotechnical work included the following:

- Drilling and sampling of multiple borings and cone penetrometer test (CPT) soundings along the proposed pierhead line
- Cyclic and static laboratory tests to quantify the behavior of the primary soil unit (BCF clay) beneath the OCSP® system under gravity and seismic loading
- Studies to quantify likely seismic loading conditions
- Detailed limit-equilibrium and numerical modeling of the OCSP® system to demonstrate that the OCSP® system would meet performance expectations during seismic events

Beginning in 2003, work on the PIEP Marine Terminal Development Environmental Assessment (EA) was started. The EA was prepared under the direction of MARAD and in cooperation with the POA. The EA considered and evaluated three alternatives for expansion of the POA:

A. Sheet pile construction with fill
B. Pile-supported dock construction with fill
C. A combination of both methods

The Draft EA was published in August 2004 (MARAD and POA, 2004) and the Final EA was completed in March of 2005 (MARAD and POA, 2005). Based on the analysis in the EA, the POA and MARAD identified Alternative A as the preferred alternative.

1.2.2 OCSP® Design Concept

The PIEP uses the OCSP® system for the wharf and the berthing area. The OCSP® system was developed and patented by PND in 2004 (Patent No. US-6, 715, 964 B2) and has been used since the early 1980s to construct wharf and berthing structures for a number of other projects in Alaska and conterminous United States, as well as in the Middle East. It has been found to be a fast and economical method of developing a wharf in many locations. However, these other OCSP® systems were typically in less severe environments relative to the combination of ice, tide, and seismic loading, and dredge depths were typically not as deep. As noted above, the closest OCSP® facility to the planned POA OCSP® development, in terms of environmental conditions, is located across Knik Arm at Port MacKenzie, where a similar OCSP® system was built in early 2000. Environmental conditions for the Port MacKenzie facility are generally similar to those at the PIEP; however, the dredge depth and sheet pile wall height are significantly less and the length of the overall development is smaller.

The mechanics of the OCSP® system are shown in Figure 1.2-4a, and a photograph of OCSP® construction is provided in Figure 1.2-4b. These figures show that the OCSP® bulkhead is a variation of a modified diaphragm cofferdam (U.S. Steel, 1984). The system relies on (1) the horizontal hoop tension developed in the arched face sheets to retain soil at the front face and (2) interface friction and bearing resistance of the tailwall and anchor pile(s) in the backfill soil to provide pullout resistance. The tailwalls serve as anchors for the face sheets, preventing excessive horizontal deformation and local instability of the face sheets.

The structural integrity of the OCSP® system relies entirely on development of the interlock tension between the sheet piles.[1] Therefore, it is critical that the interlocks remain fully engaged during and after installation because loss of interlocks makes the wall system vulnerable to structural failure. The pullout resistance of the OCSP® system results from friction between sheet piles and the soil, which depends on properties of the backfill. Backfill must have sufficient frictional resistance and be properly placed and adequately compacted to ensure that tailwalls do not pull out during gravity and seismic loading. PND also uses a proprietary design method that accounts for passive pressure at the sheet pile knuckles when estimating pullout capacity of the tailwall. This method results in a higher capacity of the tailwall than can be developed from sliding friction alone. An independent assessment was made of this mechanism by CH2M HILL as part of this suitability study and is included in Section 7. For all calculations performed by CH2M HILL in this study, the proprietary PND method was not used.

The OCSP® system uses a relatively light-weight sheet pile to provide face support and tailwall resistance. Sheet piles have a nominal thickness of 0.4 to 0.5 inch and widths of approximately 20 inches. Lengths of individual sheets can be in excess of 70 feet, although the original PND patent describes the sheet pile lengths as being up to about 65 feet in length. Tailwalls are typically 30 to 50 feet apart and can be over 100 feet in length measured from the wye pile in the direction away from the face sheets.

As with the design of any earth retention structure, both the local (or internal) stability and global stability must be verified. The critical global slip surface (that is, where the global factor of safety is lowest) shown in Figure 1.2-4a extends under the sheet piles and behind the tailwall. For a specific project, the critical slip surface depends on the final wall geometry, tailwall configuration, and the subsurface conditions at the project site. Global stability calculations are usually conducted using limit-equilibrium (conventional slope stability or retaining wall analysis) or phi-c reduction (numerical analysis) methods. The factors of safety and estimated deformation during gravity and seismic loading are compared with the established design criteria to determine suitability of the OCSP® design.

1.2.3 Construction of OCSP® System at Port of Anchorage

Construction for the port expansion began in 2006 with the North Backlands area being created. For the Dry Barge Berth project, soft or loose deposits of silts and sands were dredged, and a construction access berm constructed from granular fill. Installation of the OCSP® system began in 2008. The distance between the tailwalls is approximately 27.5 feet, and the face sheets were installed on a radius of 27.5 feet. Lengths of the sheet piles range from less than 50 feet to as much as about 90 feet. Both vibratory and impact methods were used to install the sheet piles. Environmental restrictions resulting from listing of Cook Inlet beluga whale as an endangered species resulted in restricted and intermittent construction durations.

After the installation of the sheet piles for the OCSP® system, the cells were filled with granular fill material to provide backland area. The upper layers of backfill were compacted with surface vibratory compaction equipment after the granular material was above water. Once the granular fill was near the final grade, the filled area between the cell walls was densified using vibracompaction procedures. Vibracompaction consisted of advancing a vibrating steel H-pile probe down to a target elevation, defined in the construction drawings for the specific project. The probe was withdrawn and then re-advanced into the ground with fill being added to the hole by mounding the material at the ground surface. With successive strokes (penetration and withdrawal of the probe from the ground), the advancement of the probe was progressively less deep, until the stroke was within about 10 feet of the ground surface. The intent of the vibracompaction was to assure that soil within the cell was in a very dense condition that would provide stability during gravity and seismic loading.

[1] Interlock refers to joining of two sheet pile sections by a “knuckle and thumb” arrangement. If the sheets are out of interlock, tension stresses are not transferred from one sheet to the next. Sheets that are out-of-interlock represent one of the most serious types of damage to sheet piles, as they can lead to overall structural failure.

The Wet and Dry Barge Berths and North Extension have been built. During construction, various difficulties were reported regarding the installation of face sheet piles for the OCSP® system, including very hard driving conditions and the formation of sinkholes behind the sheet pile face following inspection dredging, particularly in the southern area of construction where the sheet pile face is taller. Subsequent underwater inspections found that a number of sheet piles had come out of interlock during installation, which allowed soil to erode through the face during tidal cycles and form sinkholes at the ground surface behind the sheet pile wall. Pictures of representative sheet pile damage are shown in Section 6 of this report. Sections of OCSP® face and tailwall system were also exposed by removing the fill behind the sheet pile wall in a few areas to evaluate the extent and cause of the construction issues. Further details regarding construction sequencing and issues are discussed in Section 8 of this report. Based on this investigation work, PND recommended in a letter to ICRC dated September 23, 2011 (see Item H31 in Appendix H), that sections of the OCSP® be removed and reconstructed.

Prior to continued use of the OCSP® to complete the North Expansion of the PIEP, CH2M HILL was contracted by the USACE to evaluate the suitability of using the OCSP® system at the POA and, as appropriate, to recommend methods to optimize design and its construction. The suitability study included an assessment of use of the OCSP® system at locations where the existing sheet pile will have to be removed, as well as an assessment of the suitability of portions of the OCSP® that PND recommends accepting.

1.2.4 Project Objectives

The general objective of the suitability study is to identify any major deficiencies with the existing design approach, to determine whether the design can be constructed at this site (including risks of it being constructed as designed), and to identify whether or not a more significant change in direction of the PIEP is warranted. To support such conclusions and recommendations, the following specific objectives were used in this study:

- Evaluate the project design criteria.
- Evaluate the seismic conditions at the POA site with emphasis on seismic design criteria and the impact of design spectra on OCSP® design.
- Evaluate the hydrological conditions with emphasis on sedimentation and scour processes, and also ice forces on the bulkhead and moored ships.
- Check the internal and external stabilities of the OCSP® system using state-of-the-practice engineering analysis methods.
- Evaluate the soil-structure interaction of the constructed OCSP® system using numerical analysis. Perform independent numerical modeling to provide estimated performance of the OCSP® system and allow for comparison with performance criteria.
- Review the constructability of the design OCSP® bulkhead at the POA. The constructability assessment considers construction risks, means and methods, the contractor selection process, and construction inspection procedures.
- Consider the life-cycle costs of the OCSP® with regard to long-term performance considering corrosion.

CH2M HILL addressed these objectives by conducting detailed design studies for two separate cases: As-Designed and As-Built Design. These cases involve different assumptions regarding soil conditions, external loading conditions, and OCSP® geometry. In addition to these studies, a set of independent design studies was also conducted. The independent design studies were conducted using the same OCSP® geometry, site soil conditions, and loading, but the analyses were modified from the as-designed and as-built studies to include mitigation measures necessary to meet project design criteria. These three cases are summarized as follows:

- **As-Designed.** For this case, analyses were conducted using assumptions and methodologies similar to those used by the original design team. No independent assessments of soil properties, final constructed soil or sheet pile conditions, or external loadings were made. The intent of these analyses was to determine whether or not the original approach used by the designers was reasonable. Changes that occurred during construction were not considered.

- **As-Built Design.** For this case, analyses were performed using conditions interpreted by CH2M HILL from the available geotechnical and construction data based on post-construction conditions. Soil properties were changed to represent likely post-construction conditions based on additional soil testing. New assumptions were made regarding the external loading (e.g., more representative tidal conditions, groundwater locations, and live load during seismic design), and the geometry after construction, including sheet pile lengths and scour depths, based on CH2M HILL's interpretation of construction records.
- **Independent Open-Cell Design.** For this case, the basic configuration of the OCSP® system was maintained (i.e., type and configuration of sheet piles including face radius, dredge depths, and soil profile); however, mitigation measures were introduced with the intent that the OCSP® would meet design criteria relative to factors of safety and allowable displacements for gravity and operational loads and during the seismic events. These mitigation measures included consideration of light-weight fills within the backfill zone and addition of ground improvement within the BCF formation. The independent open-cell design did not include alternate types of wharf systems, such as a pile-supported marginal wharf, a concrete caisson wharf, or cellular cofferdam wharf, as agreed to during scoping studies with the USACE, POA, and MARAD.

Analyses conducted for this suitability study are applicable to conditions at the Dry Barge Berth, Wet Barge Berth, and North Extension 1 and 2 within the North Expansion area. No design development has occurred for areas outside of these limits, and the results or conclusions in this suitability study should not be deemed appropriate for the remainder of the PIEP.

1.3 Project Organization

The organization chart for design development of the PIEP is provided in Figure 1.1-1. As shown, MARAD is the lead federal agency for the PIEP. Integrated Concepts & Research Corporation, Inc. (ICRC) was retained by MARAD to administer the project. The USACE was contracted by MARAD in partnership with the MOA/POA to review and make recommendations regarding the work accomplished to date within the North Expansion. In support of that review, CH2M HILL was contracted in 2011 under USACE to perform the suitability study.

Other key groups involved in the engineering design or review of the PIEP were as follows:

- Starting in 2003, ICRC subcontracted with Terracon of Charlotte, North Carolina, to execute a geotechnical investigation for evaluating the site foundation conditions. Gregg Drilling and Testing, Inc. of Long Beach, California, performed the field testing and sampling. Laboratory strength and classifications tests were conducted on samples recovered from the project area by Terracon and numerous others during the work, including PND, MEG Technical Services, DOWL HKM Testlab, and Geotesting Express.
- In June of 2003, the proposed geotechnical program was presented by Terracon and PND to the Municipality of Anchorage (MOA) and the Municipality of Anchorage Geotechnical Advisory Commission (GAC). The GAC recommended that the Mayor appoint a "Seismic Design Committee" of subject-matter experts and governing officials to establish design criteria for the POA PIEP. Recommendations for design criteria were presented and adopted for the project. Resolutions adopted in 2004 by the GAC and the Seismic Design Committee, designated as the Port of Anchorage Seismic Design Committee, are provided in Appendix B.
- Also in 2003, ICRC retained an Independent Advisory Committee to review the geotechnical program performed by Terracon and its consultants and to advise ICRC and MARAD on geotechnical issues. The Independent Advisory Committee was composed of three geotechnical experts:
    - Dr. Paul Mayne, Professor at Georgia Institute of Technology, who is a specialist in the interpretation of soil properties from data collected by in situ testing methods.

  - Dr. Peter Robertson, formerly Professor at the University of Alberta and now an independent consultant, who is a specialist in the use of the cone penetrometer test (CPT) method for assessing performance of the soil during seismic loading.
  - Dr. Youseff Hashash, Professor at the University of Illinois Urbana-Champaign, who is a specialist in the estimation and evaluation of ground motions during seismic loading.
- In 2004 Terracon prepared a report presenting results of a preliminary geotechnical exploration program. This program considered the suitability of the OCSP® system, as well as a pile-supported wharf. They concluded from their work to date that the OCSP® system appeared to be more economical to construct and to potentially have lower life-cycle costs than a pile-supported wharf. In 2004, MARAD completed an environmental assessment in accordance with the National Environmental Protection Act (NEPA). An earth-filled bulkhead structure was selected as the preferred alternative for expanding the port facility.
- In 2006 Terracon completed a feasibility analysis of the earth-filled bulkhead. The Terracon analysis addressed OCSP® settlement and stability under gravity and seismic loads. The scope of their analyses included three-dimensional (3D) numerical modeling. Only limited evaluation of constructability was included in the 2006 Terracon report.
- Starting in 2006, ICRC subcontracted with PND and GeoEngineers of Bellevue, Washington, together comprising the final design team to complete the design of the OCSP® at the POA. PND was the licensed engineer-of-record. This design team independently re-evaluated available soils information and seismic parameters, developed final designs for OCSP® structures, and analyzed the proposed design using limit-equilibrium and two-dimensional (2D) finite element methods. Sensitivity analyses were conducted. The PND geotechnical report was then independently reviewed by Terracon. PND and Terracon worked independently under subcontract to ICRC. In 2006 PND also contracted with VECO Corporation of Anchorage, Alaska, to conduct global stability analyses to confirm that the OCSP® system would satisfy minimum factor-of-safety requirements for gravity and seismic loading. This work was completed in March 2007.

1.4 CH2M HILL Study Overview

The suitability study performed by CH2M HILL included an analysis of the existing seismic, hydrologic, geotechnical, and structural conditions used in the current design of the OCSP® system; an assessment of the suitability of the OCSP® system at the current site based on the review of existing conditions; and an independent evaluation of potential changes to the OCSP® system design intended to meet project design criteria. To accomplish these objectives, eight evaluation tasks and one reporting task were established, as follows:

- **Review of Definition of Design.** CH2M HILL reviewed the multi-level seismic, performance, and design criteria used for the current, as-specified design for the PIEP. The design and performance criteria were compared to other applicable port facility design criteria (e.g., Marine Oil Terminals Engineering and Maintenance Standards [MOTEMS], The World Association for Waterborne Transport Infrastructure [PIANC], American Society of Civil Engineers [ASCE] Seismic Guidelines for Ports, etc.) and recommendations were made on either the suitability of or changes that should be made to design and performance criteria for future POA expansion design efforts. These recommendations are provided in Section 2 of this report.
- **Seismic Hazard Assessment.** This assessment involved a general review of the seismic conditions associated with the project location, with the focus on evaluating the as-built conditions of the OCSP® system. Specifically, the seismic hazard assessment involved a review of firm-ground input motion used as a basis of design and site-specific ground response analyses to provide an independent estimate of input ground motions appropriate for seismic design. Results from this assessment and review are provided in Section 3 of this report.

- **Hydrological Analysis.** This evaluation consisted of both sedimentation and scour analyses, the results of which were used to develop design assumptions about design scour elevations in front of the OCSP® face sheets. Additionally, a review of the design assumptions and calculations of the PND designers was conducted to assess whether the ice forces on the dock structure and moored ships have been adequately addressed. Results from this evaluation and review are provided in Section 4 of this report.
- **Geotechnical Engineering Analysis.** Using state-of-the-practice methods, the subsurface conditions were reviewed and then independently characterized using geotechnical investigation data collected during earlier stages of the design process of the PIEP. Characterization of subsurface conditions included:
    - Development of soil profiles
    - Estimation of soil engineering properties
    - Review of groundwater conditions within the OCSP® backfill
    - Characterization of as-built backfill characteristics
    - Assessment of liquefaction susceptibility

  Two supplemental field exploration and laboratory testing programs were conducted in 2012 as part of this task to investigate (1) the characteristics of the granular backfill and the transition between the backfill and the underlying BCF clay and (2) the strength characteristics of BCF clay for static, cyclic and post-cyclic, and large displacement loading conditions. Results from this characterization effort are provided in Section 5 of this report.
- **Stability Analysis.** Implementing the results of the subsurface condition assessment, the internal and external stabilities of the as-built OCSP® wall were assessed. Internal stability addressed sheet pile interlock tension exceedance and tailwall pullout resistance. External stability addressed wall sliding, global stability, and estimates of seismic-induced permanent deformation. A sensitivity evaluation was conducted using the as-built condition to identify the most important design variables affecting internal and external stabilities. The sensitivity evaluation was also used to verify stability with respect to parameters with higher degrees of uncertainty and to serve as a check on detailed numerical modeling with the three-dimensional computer program, FLAC^{3D}. Also included in this task was a set of independent design studies where options necessary to meet project design criteria were evaluated. Results from the as-built stability evaluations are provided in Section 5 of this report; results of the independent design are included in Section 9.
- **Structural Analysis.** Structural analysis was conducted in addition to stability calculations. These evaluations defined the effect of construction defects (e.g., damaged sheets, inadequate pile embedment) on structural stability of the OCSP® system. Additionally, a life-cycle analysis focusing on potential corrosion of the OCSP® system was conducted. Results from this structural analysis are provided in Section 6 of this report.
- **Numerical Modeling.** The soil-structure interaction of the OCSP® system was evaluated using numerical analysis methods. Specifically, FLAC^{3D} analyses were conducted to provide insight into soil-structure interaction at different stages of construction and operation (e.g., end of construction, after dredging but before operation, long-term operational load with container loads, and during various levels of seismic loading) and to compare estimated performance parameters (e.g., wall deflection) to performance design criteria. Separate numerical analyses were conducted for As-Designed, As-Built, and Independent Design configurations for the OCSP®. Static and dynamic analyses were conducted. Results from the numerical analyses for the As-Designed and As-Built cases are provided in Section 7 of this report; results of the Independent Design case are included in Section 9.
- **Constructability Assessment.** The constructability assessment was conducted as a parallel task to the engineering analyses to assess the constructability of the OCSP® system at the POA site and to determine whether or not the issues encountered during construction of the early phases of the expansion project were, in part, a result of the construction methods. The results of the construction review were also used to help in

the interpretation of post-construction conditions used in the As-Built Design. The constructability assessment addressed:

- o Constructability of the OCSP® design
- o Contractor selection criteria
- o Evaluation of construction methods (for example, equipment and sequence) and records (for example, quality assurance)

Results from this constructability assessment are provided in Section 8 of this report.

- **Reporting.** Results of these tasks are documented in this summary report. The documentation summarizes anticipated performance of the As-Designed, As-Built, and Independent Design of the OCSP® structures, including comparisons to existing design and performance criteria. The report also includes conclusions regarding the suitability of the OCSP® system for the North Expansion at the POA with recommendations for improvement.

  Included within the reporting task was a series of public presentations. The intent of these presentations was to provide the public with information on the study and allow for submission of verbal and written comments. The presentations included materials and displays necessary to graphically depict the project information.

1.5 Study Methodology

This suitability study for the PIEP addresses a number of subjects that are interrelated to varying degrees. To accomplish the work, CH2M HILL assembled an integrated, multi-discipline team comprised of experts in each subject matter. These engineers conducted separate analyses to achieve the previously discussed objectives. Where subjects overlapped (for example, ground motions from site-specific ground response analyses as input excitations for numerical analyses), collaborative relationships were established. The intent of this study methodology was to provide conclusions and recommendations that consider all aspects of the project and are not biased toward any one particular aspect of the project.

The methodologies for individual analyses are discussed separately in the relevant sections of the report. Industry-accepted, state-of-the-practice methods were used for all engineering analyses. Some non-traditional analysis procedures were also used, and these methods are discussed in advance of the analysis results. As noted earlier in this introduction, contacts with the original design team were limited to transfer of information in an effort to satisfy the overall goal of providing an independent suitability study. In view of the investment that had been made prior to the suitability study, the independent approach was essential for maintaining as much project transparency as possible.

1.6 Organization of Report

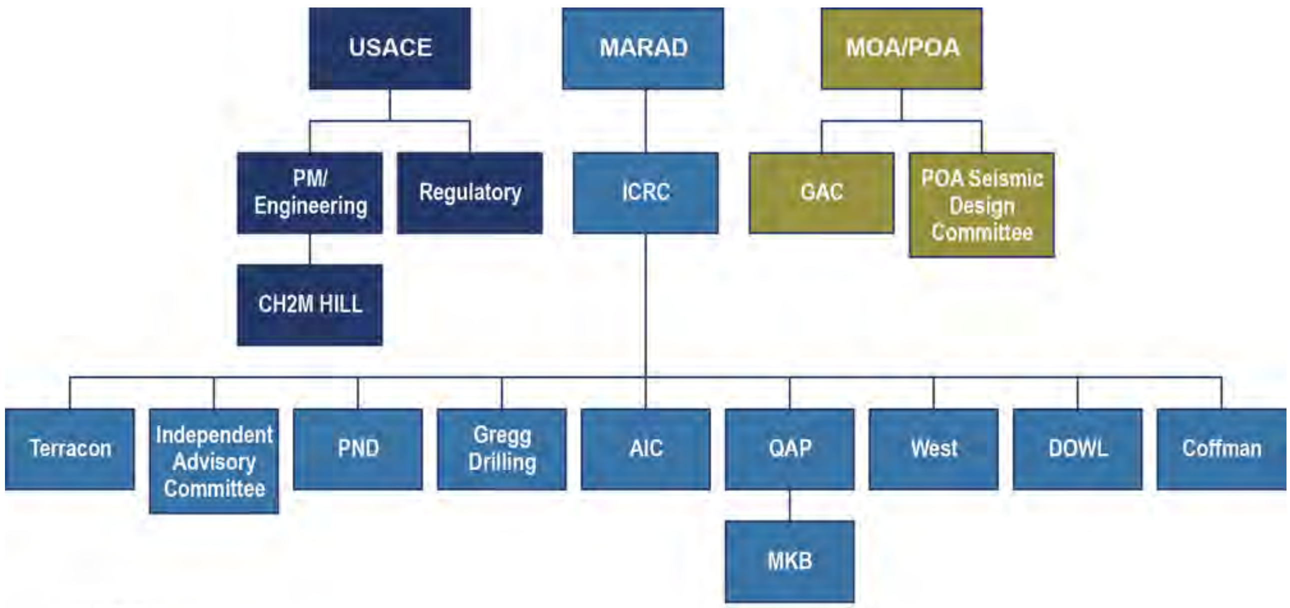
This report is organized in twelve sections and eight appendixes. The first volume includes the following report sections:

- Section 1 is the introduction.
- Section 2 discusses the definition of design.
- Section 3 summarizes results of the seismic hazard assessment and includes results of the site-specific ground response analyses.
- Section 4 provides results of the hydrological analysis, which are used as the basis for recommending tidal heights, scour depths, and ice loading conditions for design.
- Section 5 covers the conventional geotechnical analyses using state-of-the-practice methods. These results show the effects of parametric variations on performance and were used to guide cases evaluated with numerical modeling methods with FLAC^{3D}.

- Section 6 summarizes results of the structural analysis and life-cycle performance evaluation.
- Section 7 describes the FLAC^{3D} numerical modeling for both general and local loading cases, and includes global stability assessments and internal structural stress evaluations.
- Section 8 discusses the constructability review and its conclusions.
- Section 9 provides results of the independent design studies.
- Section 10 presents final conclusions and recommendations from the analyses discussed in the preceding sections.
- Section 11 discusses limitations of the work.
- Section 12 lists references that were used during the course of this work.

Appendices A through G are included in the second volume of the report, and Appendix H is provided in the third volume. These appendices provide supporting data to the field investigations, laboratory testing, and engineering analyses that have been carried out for the project:

- Appendix A provides the summary scope of work with the USACE.
- Appendix B contains the Geotechnical Advisory Commission and Seismic Design Committee resolutions.
- Appendix C presents additional earthquake time history plots (discussed in Section 3.2).
- Appendix D contains results of field and laboratory investigations that were performed for the project. It includes:
  - Appendix D1 is a memorandum describing the results of field investigations and laboratory testing programs conducted as part of the supplemental work to characterize as-built granular backfill properties.
  - Appendix D2 is a report summarizing results of field investigations and laboratory testing programs conducted as part of the supplemental work to characterize BCF clay.
- Appendix E summarizes the results of the SLOPE/W analysis of cross section F.
- Appendix F contains results from global FLAC^{3D} analyses.
- Appendix G presents results from local FLAC^{3D} analyses.
- Appendix H includes construction records documents cited in Section 8.



LEGEND

USACE	US Army Corps of Engineers	PND	PND Engineers, Inc.
MARAD	US Department of Transportation Maritime Administration	AIC	Alaska Interstate Constructors LLC
MOA	Municipality of Anchorage	QAP	Quality Asphalt Paving
POA	Port of Anchorage	West	West Construction
CH2M HILL	CH2M HILL, Inc.	DOWL	DOWL HKM
ICRC	Integrated Concepts & Research Corporation, Inc.	Coffman	Coffman Engineers
GAC	Municipality of Anchorage Geotechnical Advisory Commission	MKB	MKB Constructors
Terracon	Terracon Consultants, Inc.		

FIGURE 1.1-1. Design, Construction, and Suitability Study Organization Chart

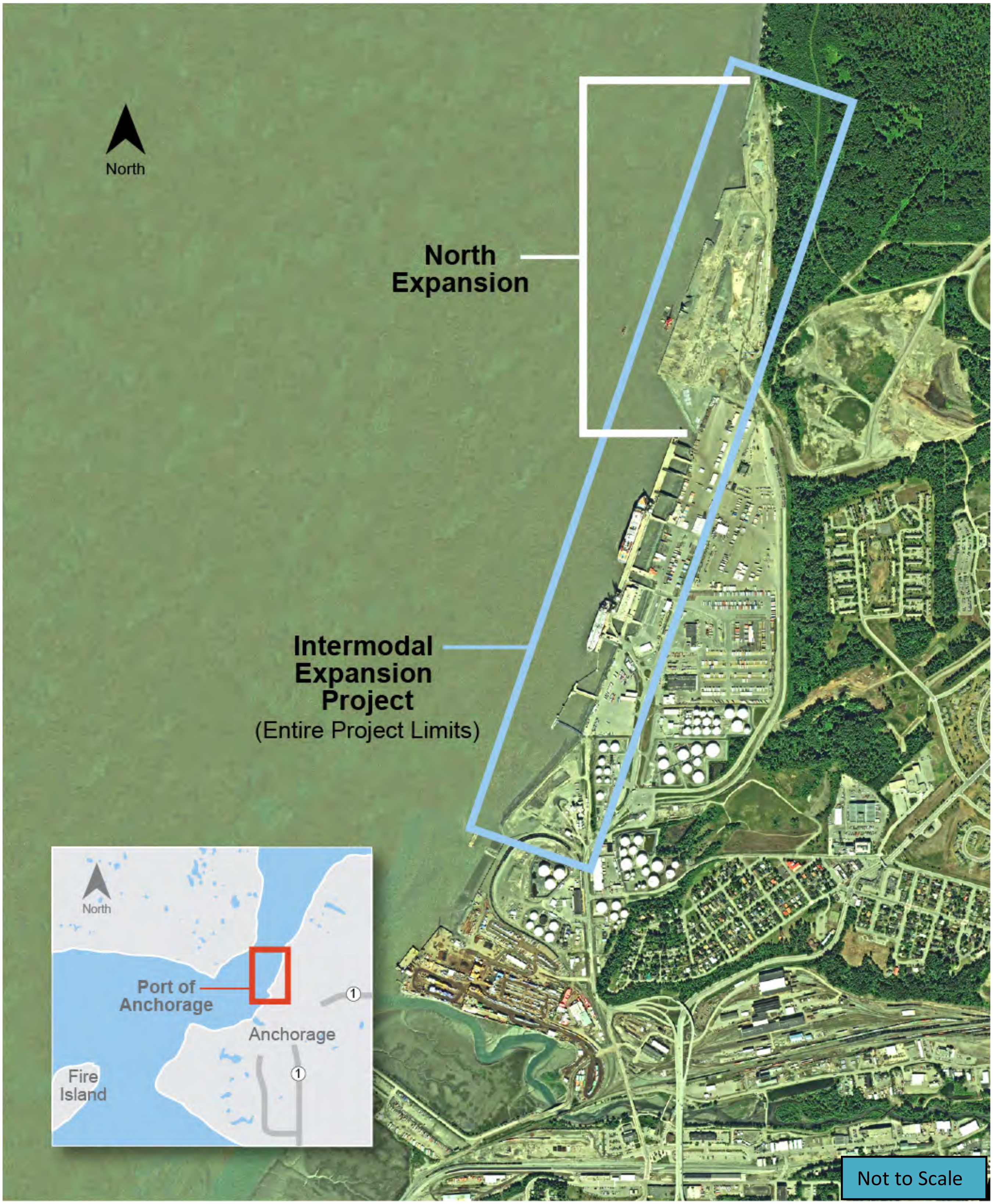


FIGURE 1.2-1. Project Location and Vicinity at Port of Anchorage in Alaska

FIGURE 1.2-2. Port of Anchorage Intermodal Expansion Project Phasing Plan

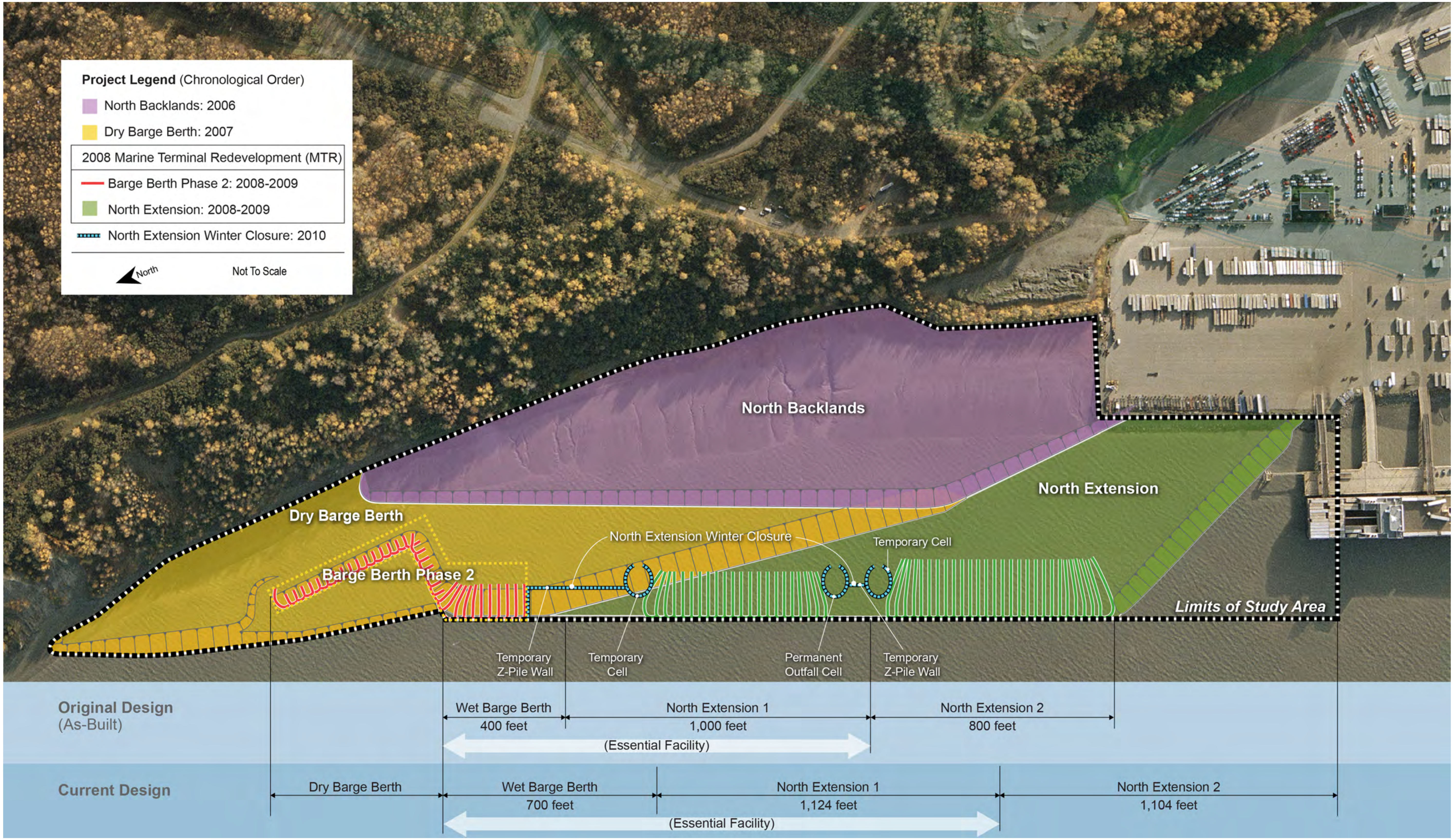


FIGURE 1.2-3. Port of Anchorage Intermodal Expansion – North Expansion Projects Definition

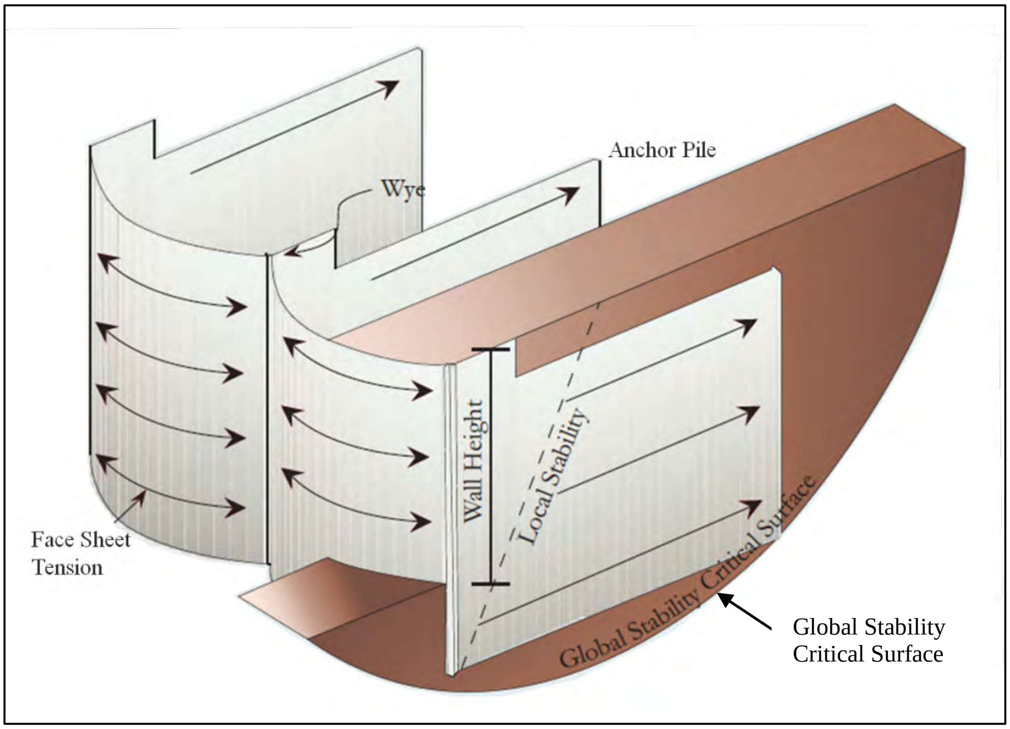


FIGURE 1.2-4a. OPEN CELL® Sheet Pile (OCSP®) System Mechanics (PND, 2012)

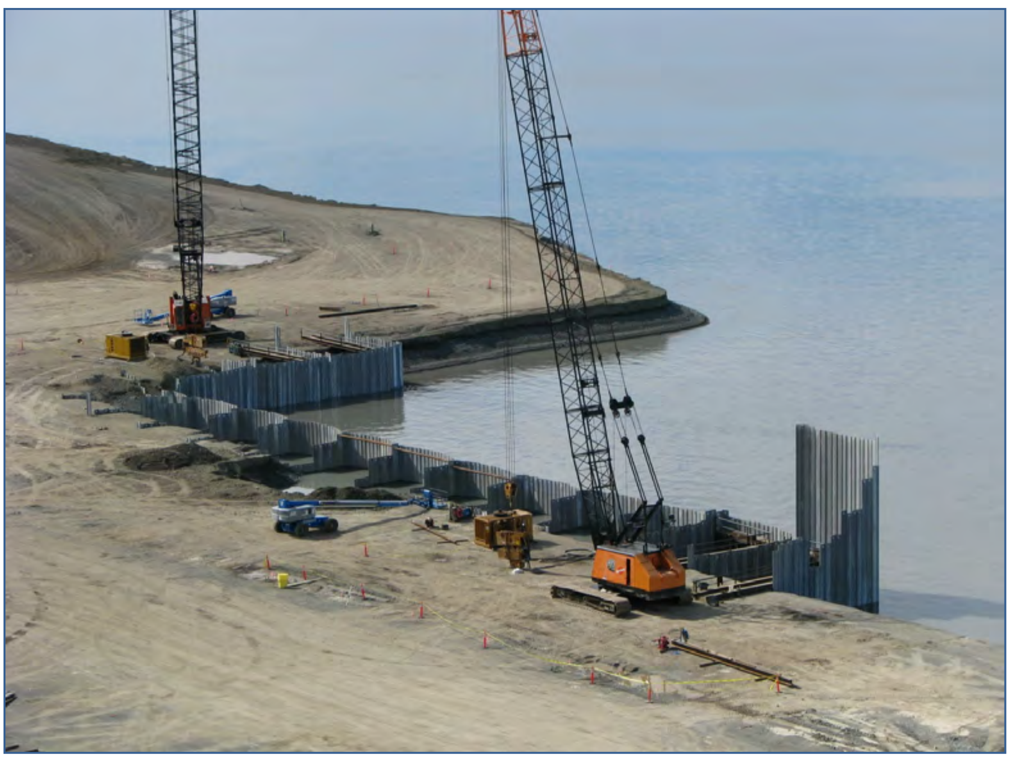


FIGURE 1.2-4b. OPEN-CELL® Sheet Pile (OCSP®) System Construction (Photo Taken 04/07/08)

SECTION 2

Definition of Design

This section of the report discusses the basis of design for the OCSP® system, first by summarizing what was used by the PND design team, the original designer, and then by providing recommended changes to the basis of design. The recommended changes are intended to either improve the long-term OCSP® system performance during operations, or provide higher confidence in acceptable performance during seismic loading. The basis of design is discussed in the following three sections:

- Section 2.1 summarizes the design criteria used by PND for the original design of the OCSP® system.
- Section 2.2 reviews the performance criteria recommended by PND for the OCSP® system at the POA.
- Section 2.3 provides a summary of the findings and recommendations from the suitability study.

2.1 Design Criteria Review

The existing PND design criteria are presented in various design criteria documents, as-built plans, and design reports and calculations. The primary source documents used to review the design criteria for the PIEP included the following:

- Howlett, Garth, and Mageau, Dan. 2008. *Geotechnical Advisory Commission Presentation*. Port of Anchorage Expansion Project.
- ICRC. 2005. *Wharf Design Report, Port of Anchorage, Alaska*.
- ICRC. 2008. *Port of Anchorage Intermodal Expansion Project, Marine Terminal Redevelopment, Geotechnical Report,* Volumes I and II.
- ICRC. 2009. *Port of Anchorage Intermodal Expansion Project, 100% Submittal Plans - General Notes*.
- ICRC. 2011a. [Port of Anchorage Intermodal Expansion Project] *PIEP Design Criteria (Draft)*.
- MARAD and ICRC. 2012. *Port of Anchorage Intermodal Expansion Project (PIEP) Budgetary Cost Estimate Report, Appendix B: PIEP Design Criteria Summary*.
- PND. 2006b. *Port of Anchorage Expansion Project – Open Cell Sheet Pile (OCSP) 35% Design Criteria*.
- PND. 2006c. *Open Cell Bulkhead Component Evaluation*.
- PND. 2006f. *Design Report and Calculations, Barge Berths, Port of Anchorage Expansion Project*.
- PND. 2007a. *Port of Anchorage Expansion Project, Dry Barge Berth Conformed Drawings – General Notes*.
- PND. 2007c. *Port of Anchorage Expansion Project, North Extension Drawings – General Notes*.
- PND. 2008a. *Port of Anchorage Expansion Project, Barge Berth Phase 2 As-Built Drawings – General Notes.*
- PND. 2009a. *Port of Anchorage Expansion Project, North Extension Cap Project Drawings – General Notes*.
- PND. 2009b. *Port of Anchorage Expansion Project, Wet Barge Berth Cap Project Drawings – General Notes*.
- PND. 2009d. *Port of Anchorage Expansion, Design Study Report*.
- PND. 2010b. *Port of Anchorage Expansion Project, North Extension As-Built Drawings – General Notes.*
- PND. 2010c. *Port of Anchorage Expansion Project, North Extension Bulkhead Project Drawings – General Notes.*
- PND. 2011a. *Port of Anchorage Marine Terminal Redevelopment, Extended Wet Barge Berth Design Manual*.

- PND. 2011b. *Port of Anchorage Marine Terminal Redevelopment, Extended Wet Barge Berth Bulkhead 100% Plans – General Notes*.
- Port of Anchorage Seismic Design Committee. 2004. *Design Approach and Seismic Parameter Selection Resolution.* Municipality of Anchorage.

The extent of CH2M HILL's review also included relevant design codes, service life, design loads (for example, dead, live, earth pressure, earthquake, hydrodynamic, wind, current and wave, impact, ice, and berthing and mooring), material properties used in design, and scour in front of the OCSP® system.

2.1.1 Design Codes and Guidelines

There are currently no over-arching codes or design guidelines that apply to waterfront structures. To address this, some ports such as the Port of Los Angeles and the Port of Long Beach have developed their own codes or design guidelines. Most ports adopt a combination of codes and design guidelines, such as the International Building Code (IBC), the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications, Customary U.S. Units* (AASHTO, 2012), and the Marine Oil Terminal Engineering and Maintenance Standard (MOTEMS).

PND acknowledges this absence of specific codes in their discussion of design criteria and states that for the PIEP there was collaboration among "appropriate engineers, the Port of Anchorage consultants and management, and the codes/guidelines" that resulted in the design criteria developed for the PIEP. Mentioned within PND's discussion of design criteria are the American Society of Civil Engineers (ASCE) guidelines for determining appropriate response spectra and design level ground motions from earthquakes and buildings and other structures in SEI/ASCE 7-02, and ASCE's Technical Council on Lifeline Engineering guidelines for the design of waterfront structures in seismic areas (ASCE, 1998). PND also mentions the involvement of local business leaders and national experts (Port of Anchorage Seismic Design Committee, see Appendix B) in the development of the two-level approach to seismic design, as well as recommending that at least one berth be designed above the two-level approach for higher earthquake resistance as an emergency point for receiving relief supplies and goods.

Based on these discussions and a review of available literature, the document *PIEP Design Criteria Summary* (MARAD and ICRC, 2012) was developed by ICRC in conjunction with MARAD, the Port of Anchorage, PND Engineers, and other consultants for the PIEP. The document lists the following codes and criteria as applicable to the design of bulkhead structure and fenders:

- IBC (International Building Code), 2006
- AASHTO LRFD Bridge Design Specifications, 2006
- Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS), 2003
- American Society of Civil Engineers (ASCE) 7-02, Minimum Design Loads for Buildings and Other Structures, 2002
- USACE Design & Engineering Manual, 1989
- MOA Criteria as Specific and General Recommendations

CH2M HILL's review identified a number of other relevant design codes and criteria that, although available at the time, were not included in the *PIEP Design Criteria Summary* listed above or discussed by PND. These additional design codes and criteria are as follows:[1]

- Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures (USACE, 1989)
- Seismic Design of Waterfront Retaining Structures (USACE, 1992)

[1] Only those references available in 2011 are listed. More recent documents, such as AASHTO (2012), have now been published but these documents were not available when MARAD and ICRC prepared their document.

- Seismic Guidelines for Ports (ASCE, 1998)
- Unified Facilities Criteria (4-151-10) – General Criteria for Waterfront Construction (UFC, 2001)
- Seismic Design Guidelines for Port Structures (PIANC, 2001)
- Unified Facilities Criteria (4-152-01) – Design: Piers and Wharves (UFC, 2005)
- Wharf Design Criteria v2.0 (POLB, 2009)
- International Building Code (ICC, 2009
- Municipality of Anchorage Local Amendments to IBC 2009 (MOA, 2009)
- ASCE 7-10 Minimum Design Loads for Buildings and Other Structures (ASCE, 2010)
- Marine Oil Terminal Engineering and Maintenance Standards (California Building Code, 2010)
- Code for Seismic Design, Upgrade and Repair of Container Wharves (POLA, 2010)
- AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 5th Edition (AASHTO, 2010)
- Coastal Engineering Manual (USACE, 2011)

It is likely that PND considered some or all of the above codes, manuals, and guidance documents at the time of their design work, either directly or indirectly; however, these additional references are not listed in a basis of design document. In the future the POA should require preparation of a basis of design document during the initial phase of project development to avoid any future questions regarding design methodologies or approaches.

Based on CH2M HILL's review of OCSP® system design and CH2M HILL's experience in designing other marine structures in seismically active areas, a combination of the above codes will have to be used to establish a basis of design. The general concepts required for seismic design can be taken from some of the newer design codes, such as the *Seismic Design of Piers and Wharves* (ASCE, 2011) and *Marine Oil Terminal Engineering and Maintenance Standards* (California Building Code, 2010); however, these newer codes need to be interpreted relative to more specific design guidelines for cellular sheet pile structures, such as those found in *Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures* (USACE, 1989). Some other newer codes, such as *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units,* 6th Edition (AASHTO, 2012) provide methodologies that should be considered for estimating earth pressures on retaining walls.

While there were additional codes and standards that CH2M HILL expected to see documented, PND's general approach to design seemed reasonable, and in most cases design criteria were appropriate. However, as noted in the next subsections, some design and environmental loading conditions were not explicitly addressed in the PND project documentation. It is unclear whether these were considered and concluded as not being necessary or possibly overlooked. In this regard and as noted above, a basis of design document that listed all loading conditions would have helped identify which load cases were considered and not considered. It is also possible that the process of identifying these codes, particularly some of the newer codes, would have triggered consideration of load cases that appear to have been overlooked.

2.1.2 Service Life

Waterfront structures are typically designed for 25 to 50 years of service life. Based on the *PIEP Design Criteria Summary* (MARAD and ICRC, 2012), the service life of the PIEP bulkhead structure is 50 years. A 50-year service life is consistent with what is used in other ports around the United States (for example, Ports of Los Angeles and Long Beach, Ports of Seattle and Tacoma, and Port of Houston). This service life is also consistent with the exposure period used for evaluation of seismic hazards for most buildings. Current AASHTO *LRFD Bridge Design Specifications* (2012) increased the service life to 75 years for normal bridges, and 100 years is used for special bridges.

CH2M HILL considers the 50-year service life to have been an appropriate service life for the PIEP OCSP® system at the time the PIEP was being planned. However, with the increased service life being used in standard AASHTO bridge design, it also makes sense to consider a 75-year service life for future facility development, particularly where capital costs are high and replacement is difficult.

One of the main factors affecting service life of waterfront structures, especially those fabricated from steel, is corrosion. Corrosion will gradually reduce the capacity of the members exposed to the environment to the extent that they are rendered structurally unsound. Areas most vulnerable to corrosion are within the zone of fluctuating tidal elevation. This would be most significant for the facewalls and the wyes within the OCSP® system, where tidal elevation fluctuates between elevations -5 feet mean lower low water (MLLW) and +34.5 feet MLLW.

Methods of corrosion protection generally fall into the following categories:

- Structural design
- Protective coating
- Cathodic protection

PND proposes to achieve the design service life of 50 years with a combination of coating/galvanization, corrosion allowance, and a cathodic protection system.

Additional discussion of corrosion potential of the OCSP® system at the PIEP is provided in Section 6, as part of the life-cycle discussions.

2.1.3 Design Loads

The following subsections provide a review of the loads that were used in the design of the OCSP® system and CH2M HILL's recommendations regarding the use or modification of these loads. The loads include:

- Dead loads resulting from materials used in construction of the OCSP® system
- Live loads that occur during operation of the OCSP® system
- Earth pressure loads that develop on the face and sides of the sheet piles
- Earthquake loads that have various likelihoods of occurrence
- Hydrostatic loads resulting from tides in front of the cell and groundwater behind the OCSP® system
- Hydrodynamic loads resulting from inertial response of groundwater behind the sheet pile wall and of sea level in front of the wall during earthquake loading

In addition to these loads, a number of other loads associated with a normal port facility were reviewed, including loads from wind, current and wave, impact, ice, and berthing and mooring of ships.

2.1.3.1 Dead Loads

Dead loads for the OCSP® system are the cumulative weight of the entire structure, including the weight of the soil, pavement, steel sheet piles, concrete utilidor/cap, and other permanent attachments. The existing design criteria did not clearly specify the unit weights used during the design. In the absence of these unit weights, there is a potential for dead-load variations, depending on the unit weight assumptions.

Because there can be some variation between designers and agencies in what would be considered the appropriate unit weights, unit weight values based on industry standards and actual field conditions are normally stated in a basis of design document. In the absence of this information in the PND, MARAD, and ICRC design documentation, CH2M HILL used the standard unit weight values shown in Table 2.1-1 (California Building Code, 2010) for this suitability study.

Dead load from the container crane was not included in the dead load used for design of the OCSP®. Concrete beams for supporting container cranes are currently planned to be installed behind the OCSP® system at the southern end of the North Expansion project (Figure 2.1-1 is a wharf cross-section showing crane rail beams). The waterside crane rail beam will be supported on piling embedded in the Bootlegger Cove Formation (BCF), and the landside crane rail beams will be supported on piling founded in the granular backfill. The lengths of the piling will be such that the cranes and supporting beams will not exert additional dead (axial) load to the OCSP® system.

TABLE 2.1-1
Recommended Standard Unit Weights

Material	Unit Weight (pcf)
Steel or cast steel	490
Cast Iron	450
Aluminum alloys	175
Timber (untreated)	40-50
Timber (treated)	45-60
Concrete, reinforced (normal weight)	145-160
Concrete, reinforced (lightweight)	90-120
Asphalt paving	150

Note: Values taken from Table 31F-3-1 of *Marine Oil Terminal Engineering and Maintenance Standards* (California Building Code, 2010).

2.1.3.2 Live Loads

The live loads considered for the OCSP® system design include uniform loading from container storage, mobile cranes, forklifts, and truck loading. The design uniform loading was changed during the course of the project. Based on the General Notes on the as-built plans for the North Extension project (PND, 2010b), the original design uniform loading was 200 psf within 200 feet of the bulkhead control line, and 1,000 psf beyond 200 feet. CH2M HILL understands that the OCSP® system constructed to date was designed to this uniform live loading. In 2011, the design uniform loading was increased to 1,000 psf in the *Extended Wet Barge Berth Design Manual* (PND, 2011) and the *PIEP Design Criteria Summary* (MARAD and ICRC, 2012).

Based on project plans, rail-mounted container cranes will operate in the southern end of the North Expansion project. The weight of the crane will be supported by concrete crane beams, which in turn will be supported on piling embedded in either the BCF soil below the granular fill (waterside rail) or the granular fill (landside). As a result, rail-mounted cranes will not exert additional vertical live load to the bulkhead structure.

Other live loads listed in the *PIEP Design Criteria Summary* (MARAD and ICRC, 2012) include AASHTO HS-25 truck, Taylor 950L forklift, top-pick (80,000 lbs), and 275-ton crawler crane/truck crane. These live loads are considered to be consistent with the operational requirements of the PIEP. Since these live loads typically only have a localized effect on the pavement and utilidor design, they will not control the design of the bulkhead.

CH2M HILL has the following observations regarding the live loads:

- The proposed live load of 1,000 psf is consistent with the typical uniform loading requirements for container piers (UFC, 2005 and POLB, 2009). This load applies for long-term operational (gravity) loading. The design performed by Terracon used the lower live load criteria (that is, 200 psf for 200 feet and then 1,000 psf), which is lower than this design. It appears that PND repeated the internal and external stability analysis using higher live load for some sections at the North Extension and Wet Barge Berth (PND, 2011a). However, it was not clear whether all the other analyses were repeated using the higher live load. These loads are higher than the original and could result in larger movements or lower factors of safety.
- During seismic loading, PND used a live load factor of zero, which means that no contribution to live load was considered during seismic design. There is no consensus within the earthquake engineering community on the live load factor that should be applied for waterfront facilities. Some ports, such as the Port of Long Beach and the Port of Los Angeles, use a live load factor of 0.1, meaning that a live load of 100 psf is used for seismic loading studies (that is, 0.1 × 1,000 psf = 100 psf). Other guidelines recommend live load factors between zero (California Building Code, 2010) to 0.2 (PIANC, 2001).

- Given the importance of the PIEP to the city of Anchorage and the state of Alaska, CH2M HILL recommends that a live load factor of 1.0 be used for operational loading and a live load factor of 0.1 to 0.2 be used for seismic loading. The live load factor of 1.0 for operational loading means that the design should apply 1,000 psf to all areas behind the face of the OCSP® system during long-term, global stability evaluations associated with, for example, tidal fluctuations. In all likelihood the entire area behind the OCSP® structure face will not be loaded to a uniform 1,000 psf live load, making any stability analyses using this live load conservative to some degree. For seismic loading the use of a live load factor of 0.1 to 0.2 results in 100 to 200 psf of live load. This reduced load accounts for the low probability that the area behind the face of the OCSP® structure will be fully loaded and that the load is not fully coupled during a seismic event. In the subsequent analysis of the suitability study, the higher factor of 0.2 was used. Use of the higher live load is equivalent to approximately an extra foot of soil fill at the site and therefore is not a significant increase.
- Another potential source of earthquake loading that was identified during review was additional lateral loading to the face of the OCSP® structure from the seismic response of the crane system. In this case, the inertial response of the crane imposes a lateral load to the piles supporting the crane rails, which in turn develops additional force on the OCSP® facewall. These forces would typically develop in the upper 5 to 10 pile diameters. The combination of soil reaction to lateral pile response, as well as distance between the crane-rail piles and the OCSP® facewall, should be sufficient to minimize this additional loading to the face. Further, the facewall has sufficient flexibility and an adequate margin of capacity to resist such loads. Relative to the overall mass of soil, the lateral pile forces will be small, and therefore will not affect the global stability of the OCSP® structure. For this reason CH2M HILL believes that the lateral pile force can be neglected.

2.1.3.3 Earth Pressure Load

The function of the OCSP® system depends on the development and transfer of earth pressure forces. At the face of the cellular structure the magnitude of earth pressures for long-term operations will depend on the type, consistency, and weight of soil, groundwater and tidal elevations, exterior live loads, and the flexibility (or compliance) of the retaining wall system. During seismic loading, an increment of earth pressure develops as the soil behind the face of the OCSP® structure exerts additional inertial force, which will vary according to the level of earthquake shaking and soil characteristics. These face forces are resisted by interface friction that develops along the tailwalls. The interface friction will vary according to the soil type and density, and the constraint imposed during placement of fill behind the tailwall.

The general approach used by the PND design team is not fully documented and, therefore, it is not completely clear how the earth pressure forces used for design compare to those from conventional analyses. Available design information suggests that PND used conventional earth pressure theory to estimate pressure at the face of the OCSP® structure; however, proprietary methods were used to estimate the reaction developed by the tailwall. The proprietary methods account for the interface friction along the face of the tailwall, as well as additional reaction developed by the knuckle at the sheet pile interlock, as it is pulled through the soil. It is understood from discussions with the USACE that PND may have conducted computer modeling studies of the tailwall reaction at the University of Alaska Anchorage to demonstrate that the OCSP® tailwall will develop a larger reaction than would be developed by interface friction alone.

In the absence of specific information regarding the facewall and tailwall design, CH2M HILL conducted a series of analyses to investigate the level of earth pressures that could develop on the OCSP® structure, and the effects of material property and earthquake loading variations on these pressures. These independent analyses included an evaluation of additional force contributed by the knuckle at the interlock. This allowed the internal stability of the cellular structure to be assessed independently. Additional discussion of earth pressure used in design, as well as independent evaluations of these pressures made as part of this project, are included in Section 5.2. Results of tailwall interlock modeling are presented in Section 7.5.

2.1.3.4 Earthquake Loads

This section provides an overview of the three levels of seismic loading used during design of the PIEP. The discussion also summarizes performance expectations for the three levels of loading, the method used to determine these levels of shaking, and the risk associated with the earthquake loads. Further discussion of earthquake loading is also provided in Section 3 of this suitability study.

Seismic Hazard Levels

A multi-level approach was used for the seismic design of the OCSP® system. The multi-level approach involved either two levels or three levels of seismic ground shaking, depending on the specific North Expansion berth:

- **Operational Level Earthquake (OLE).** All facilities were designed for OLE seismic ground motions. These ground motions are defined as having a 50 percent probability of being exceeded in 50 years. This design ground motion corresponds to an average return period of 72 years.
- **Contingency Level Earthquake (CLE).** All facilities were also designed for the CLE seismic ground motions. These ground motions were defined as having a 10 percent probability of being exceeded in 50 years. This design ground motion corresponds to an average return period of 475 years.
- **Maximum Considered Earthquake (MCE).** The North Extension 1 and Wet Barge Berth were designed for the MCE seismic ground motions. These two facilities were classified as essential post-earthquake facilities that would serve as emergency points of entry for receiving relief supplies and goods. The MCE ground motion was defined as having a 2 percent probability of being exceeded in 50 years. This ground motion corresponds to an average return period of 2,475 years.

The levels of ground shaking for the OLE and the CLE are consistent with recommendations in the ASCE (1998) manual *Seismic Guidelines for Ports* and serves as the level of design for other major ports, such as the Port of Los Angeles and the Port of Long Beach. The use of the MCE for design of a portion of the OCSP® system was based on a resolution from the Port of Anchorage Seismic Design Committee on June 29, 2004 (see Appendix B). The panel recommended examining and evaluating the physical and economic feasibility of "designing at a minimum, one berth to withstand a seismic event greater in scope than a Level 2 Contingency Level Earthquake in order to provide an emergency point of entry for goods and supplies necessary to support the community."

Based on the Seismic Design Committee's recommendation, the MOA and the POA agreed that two areas within the North Expansion should maintain services under a seismic event "greater in scope than the CLE," in keeping with lifeline earthquake engineering concepts. The MCE was selected by PND as an appropriate event to meet this requirement. The spectral acceleration values from the MCE event would be roughly 60 percent higher than those for the CLE, and more than three times the OLE. The areas identified for the Essential Facilities are identified in Figure 1.2-3.

Performance Expectations for Seismic Loading

The minimum performance goal for these three levels of earthquake loading is that damage will not endanger the life safety of occupants or users of the structure. This is a common precept of all codes used in the United States. This requirement means that the post-earthquake structure should continue to support gravity loading and should not prevent egress.

In addition to these minimum requirements, ICRC (2008) identified additional performance goals for each of the selected levels of ground shaking. These performance goals are listed in Table 2.2-1 in this section. Below is a summary of the goals, as well as CH2M HILL's interpretation of the goals relative to the OCSP® system:

1. **Serviceability during the OLE.** Provide for "very little additional bulkhead movement beyond static loading condition – damage repairable in a short time period and no interruption to wharf operations" as a result of the earthquake. This requirement normally implies only minor or negligible cosmetic damage that is visually observable and/or accessible for repairs in a short period of time. To meet this objective, the behavior of the

structural elements should usually be nearly elastic with very little or no residual deformation. Most port structures are designed to meet this serviceability requirement.

For the OCSP® facewall, this serviceability requirement means that the above-water portion of the facewall could undergo small permanent deformations, as long as the sheet piles do not come out of interlock. The below-ground component of the facewall should be designed such that no damage occurs, since this section cannot be inspected. To meet this requirement, deformations would have to remain within elastic limits. In the case of the tailwall, practical methods of inspecting the tailwall section are not available other than to excavate alongside the tailwall, which is impractical for an operating port facility. This means that for the tailwall, no damage should occur within the sheet pile, particularly since there is limited redundancy in the tailwall system. In this context, redundancy refers to the ability of adjacent structural components to carry load if a structural member fails under seismic loading, potentially resulting in loss of reaction for the facewall. Permanent movement of the tailwall could occur, as long as the tailwall stresses remain within elastic limits and movement does not result in loss of containment of the face.

2. **Controlled and Repairable Damage during CLE.** Provide for "small additional bulkhead movement beyond static loading condition – damage repairable with minimal interruption to wharf operations." For most CLE designs, damage is acceptable, but it should be controlled, economically repairable within a few months, and not be a threat to life safety. This requirement usually means that damage is in locations that are visually observable and accessible for repair and that the structures should respond in a ductile manner and experience limited inelastic deformations. Most port structures are designed to meet these controlled and repairable damage requirements.

   For the OCSP® facewall, this serviceability requirement suggests that limited tilting and/or yielding of sheet piles above the water could occur, including limited plastic deformation of the interlock. However, no loss of interlock should occur, since the failure could be sudden with little warning and repair would be very difficult. The tailwall would be expected to remain undamaged during the CLE (that is, remain essentially elastic), since the tailwall cannot be inspected and repair would be very difficult. Permanent slip of the tailwall towards the water would be permissible, as long as it did not result in facewall failure. This criterion means that the factor of safety for permanent slip should be less than the factor of safety for sheet pile yield to assure that slip occurs before structural yield.

3. **Continued Operation of Essential Facility after MCE.** Provide for "Moderate additional bulkhead movement beyond static loading condition – moderate damage but economically repairable with some significant interruption to damaged portions of wharf operation." The parts of the OCSP® system designed as an Essential Facility are expected to be operational almost immediately after the MCE. For the Essential Facility, damage is expected to be limited and critical services can be quickly restored in a matter of days to allow relief operations. Cranes would not necessarily have to be functioning; however, the extent of damage to the facility would not preclude mobile cranes from unloading emergency food and fuel.[2] The effects of the MCE are expected to involve more extensive wharf damage and disruption of services for the non-Essential Facilities.

   For this "continued operations" requirement, the facewall of the OCSP® system could experience extensive yielding and tilting, and plastic deformation of the interlock could be more extensive than the CLE condition; however, the amount of damage would be limited. Since the serviceability requirement includes being economically repairable, below-ground sections of the facewall should remain relatively undamaged, although structural yielding of the sheet would be permissible. The tailwall would be expected to remain undamaged (that is remain essentially elastic), since the tailwall cannot be inspected and repair would be very difficult. Permanent slip of the tailwall towards the water would be permissible as long as it did not result in facewall failure or loss of contained backfill. This criterion also requires that the factor of safety for permanent

[2] Given that cranes may not be functional following the MCE, it would be advisable for the POA to develop an Emergency Action Plan (EAP) for the essential facility. This EAP should identify the location of mobile equipment suitable for use in unloading vessels and emergency contracting mechanisms for obtaining equipment and operators necessary to provide emergency services following an MCE.

slip should be less than the factor of safety for sheet pile yield to assure that slip occurs before structural yield.

These performance expectations were defined by PND in terms of expected factors of safety and allowable displacements for the OCSP® system, as discussed in Section 2.2 of this suitability study.

Method of Determining Seismic Design Levels

The three levels of earthquake loading were initially established for the PIEP based on information published by the United States Geological Survey (USGS) for Alaska in 1996. At the recommendation of the Municipality of Anchorage Geotechnical Advisory Commission and the PIEP Independent Advisory Committee, a site-specific probabilistic seismic hazard analysis (PSHA) was completed by URS for ICRC to define the levels of ground shaking for each of the above cases (URS, 2008). The 2008 PSHA used more up-to-date assumptions regarding earthquake sources, frequency of seismic activity, ground motion attenuation, and uncertainty to estimate ground motions at the POA site. Results of the PSHA established the peak horizontal ground acceleration (PHGA), as well as spectral accelerations at periods of 0.3 and 2.0 seconds, for different probabilities of exceedance. Table 2.1-2 summarizes the level of PHGA and spectral accelerations for the three levels of ground shaking. The PHGA and spectral accelerations define response spectra ordinates that were used by URS for fitting acceleration time histories and in design. Depending on the stiffness of the structure, the period of fundamental response changes and these changes can lead to higher levels of seismic shaking. For the evaluation of most geotechnical response mechanisms, the PHGA, which defines the acceleration at zero period, is used.

TABLE 2.1-2

Levels of Earthquake Ground Shaking at Firm-Ground Level from URS PSHA

Earthquake	Return Period (years)	Peak Horizontal Ground Acceleration (PHGA)	Spectral Acceleration at 0.3 sec (g)	Spectral Acceleration at 2.0 sec (g)
OLE.	72	0.16g	0.26	0.1
CLE	475	0.34g	0.59	0.24
MCE	2,475	0.58g	1.02	0.44

Note: Firm-ground level occurs at depths of nearly 500 feet below the ground surface.

OLE = operational level earthquake
CLE = contingency level earthquake
MCE = maximum considered earthquake
g = acceleration of gravity (32.2 ft/sec^2)

The PHGA values described above were estimated by URS for a soft rock outcrop site (USGS B/C site class boundary, where the B/C refers to the boundary between Site Class B [soft rock] and Site Class C soils [stiff soil]). The firm-ground level was defined by URS as being nearly 500 feet below the ground surface, based on site-specific geologic and seismic studies. Soils in the upper 100 feet on the landward side of the OCSP® system generally are characterized by shear wave velocities (V_s) from 1,000 to 1,200 feet per second (ft/s), placing them in site class D, as defined in the IBC. Soils on the seaward side of the OCSP® wall system have V_s values from 600 to 1,000 ft/s, also classified as site class D. Ground motions for soils in this site class could be either amplified or de-amplified by local geology as they propagate through the soil profile to the ground surface, depending on the level of firm-ground PHGA and the site class.

Site-specific ground response analyses were conducted by Professor Youssef Hashash for Terracon to evaluate the changes in ground motion as they propagate from the USGS B/C boundary to the ground surface. These evaluations are discussed in Section 3 of this report. Table 2.1-3 lists eight representative earthquake time histories for the horizontal motions used in the seismic analyses performed in the previous investigations (URS, 2008; Hashash, 2008). These earthquake time histories were selected based primarily on their sources/types (that is, crustal or subduction earthquake), magnitudes (that is, strong shaking duration), and distances to rupture energy that are consistent with the controlling earthquakes. They were also modified by URS so that their spectral acceleration values match closely to the levels of design ground motion (OLE, CLE, and MCE). The appropriate selection of time histories (in terms of type, magnitude, and distance) and spectral modifications (including PHGA) are important because earthquake duration and spectral values (in addition to PHGA) affect the dynamic responses and deformations of port facilities during earthquakes.

TABLE 2.1-3
Earthquake Time Histories

Earthquake	Earthquake, Year	Moment Magnitude (M_w)
Operating Level Earthquake (OLE)	Puget Sound, 1965	6.5
	Peru Coast, 1974	6.3
	Michoacan, Mexico, May 22, 1997	6.6
Contingency Level Earthquake (CLE)	Western Washington, 1949	7.1
	Michoacan, Mexico, January 11, 1997	7.1
	Nisqually, 2001	6.8
Maximum Considered Earthquake (MCE)	Nisqually, 2001	6.8
	Michoacan, Mexico, 1997	7.1
	Western Washington, 1949	7.1
	Cascadia Megathrust Synthetic (ALL005)	9
	Cascadia Megathrust Synthetic (ALL009)	9

Seismic Risk Considerations

This multi-level approach to seismic design recommended for the PIEP is used in the evaluation and design of most modern-day port facilities in order to provide for continued operation, minimized economic losses, controlled seismic damage, and to provide for life safety during seismic events. The inherent uncertainties in the occurrence of earthquakes, in combination with the costs of protecting port facilities from different levels of shaking, have led to this multi-level approach to seismic loading. With the multi-level approach, the OCSP® system is design to meet various performance goals for each level of earthquake loading. Use of an OLE and CLE with return periods of 72 years and 475 years, respectively, is consistent with what is used for many ports on the West Coast of the United States. The idea of using a longer return period (that is, 2,475 years) for design of the Essential Facility is also consistent with the approach used for other critical lifeline systems. Values identified by the design team for the OLE, CLE, and MCE are discussed further in Section 3.1 of this report.

Although this multi-level approach to seismic design is consistent with other ports along the West Coast of the United States, there have been questions about whether the PIEP should all be designed to a higher seismic hazard level than identified above, given the importance of the POA to Alaska. Since the POA is the only major port in the area, loss of use could have a significant effect on the region due to the lack of back-up port facilities. However, in the absence of a formal risk analysis that evaluates the consequences of complete or partial shutdown of the POA on the economy of the area, CH2M HILL believes that the approach of designing the North Extension 1 and the Wet Barge Berth as essential facilities is appropriate based on current and projected services levels to the POA and capital costs necessary to design to higher seismic hazards. As future developments are planned by the POA, consideration should be given to conducting a formal risk study involving the potential consequences of a large earthquake on the POA and the regional economy. Results of such a risk study could be used to identify the level of service that will be required, in consideration of population growth, the risk of long-term shut-down of portions of the POA, and the cost of development.

2.1.3.5 Hydrostatic (Phreatic) Water Loads

Hydrostatic loading occurs at the face of the OCSP® system, resulting in a net landward force when the tidal height exceeds the groundwater elevation behind the OCSP® system and a net seaward force during periods when the tidal height is lower than the groundwater condition. The tidal variation at the POA is large, with typical daily variations in the order of 30 feet and extreme daily variations of approximately 40 feet. Groundwater conditions behind the OCSP® structure face result from a combination of water inflow through and under the cellular structure during periods of high tide, as well as from surface water infiltration during rainfall and from runoff from slopes above the backlands.

The design criteria for hydrostatic loading were set by PND based primarily on National Oceanic and Atmospheric Administration (NOAA) tidal information and fluctuations and groundwater observations for a similar OCSP® system at Port MacKenzie, located across Knik Arm from the POA and where similar tidal conditions occur. The PND observations of groundwater behind the OCSP® structure at Port MacKenzie suggest that the groundwater behind the OCSP® structure does not fluctuate with tidal changes but remains relatively constant, indicating little flow through the interlocks. Results of groundwater measurements at Port MacKenzie also showed relatively low salinity, again suggesting limited in-flow of tidal water. Based on this information and experience, PND assumed that the hydrostatic loading conditions for the POA would be as shown in Table 2.1-4 for static and seismic loading. As part of the PIEP, an automated groundwater monitoring system was installed by Terracon during construction of the face and tailwall of the OCSP® system and within the backfill used to fill the OCSP® structure. The instrumentation system provided groundwater measurements between September of 2009 and June of 2011. Results of the monitoring were time averaged by Terracon to show long-term changes in groundwater elevations. Individual 30-minute readings for piezometers installed in backfill in Cells 15 and 45 were also obtained and reviewed. It was concluded from this review that groundwater elevations in the backfill typically range from elevation +15 to +20 feet MLLW and show a limited correlation to the tidal cycle. This is further discussed in Section 5.1.4.

TABLE 2.1-4
Hydrostatic Loading Assumptions Assumed by PND (ICRC, 2008)

	Limit Equilibrium		Numerical Modeling	
Loading Case	**Water Elevation In Front of Wall (feet)**	**Water Elevation Behind Wall (feet)**	**Water Elevation In Front of Wall (feet)**	**Water Elevation Behind Wall (feet)**
Static	-5	18	-5	18
Seismic	16.5	16.5	11.5	16.5

Notes:
Elevation datum is mean lower low water (MLLW).
Sensitivity analyses were also performed by PND for all three seismic loading cases (OLE/CLE/MCE) using limit-equilibrium methods and for the CLE using numerical modeling methods.

The CH2M HILL design team reviewed the available information provided in reports prepared by PND and Terracon to determine whether the information presented in Table 2.1-4 provided an adequate basis of design. The conclusion from this review was that the hydrostatic loading conditions identified in Table 2.1-4 are generally appropriate for static design; however, available data supported raising the static groundwater elevation from +18 feet MLLW to +20 feet MLLW. There is also a concern that the groundwater values used for seismic design are insufficient for both limit equilibrium and numerical modeling. The tidal fluctuation ranges from -5 feet MLLW to +34.5 feet MLLW. The design values used by PND for limit-equilibrium and numerical analyses have probabilities of occurrence of 50 percent at elevations below +16.5 MLLW and approximately 35 percent at elevations below +11.5 MLLW. Although the tidal elevation for seismic design is often assumed to be the mean sea level in areas with more moderate tides, the large fluctuations in tidal elevation in Anchorage occur twice a day, and the resulting 35 or 50 percent probability of a design seismic event occurring when tidal elevation is lower appears to result in too much risk.

Based on further evaluation and review, CH2M HILL believes that the tidal elevation for seismic cases is ideally evaluated with an analysis that considers the full tidal range. This can be accomplished with the combination of sensitivity analysis using the 2D Static Limit Equilibrium method over the full tide range and a FLAC 3D analysis at a single tide elevation for verification and correlation. The elevation chosen for the FLAC 3D modeling is represented by the mean minus one standard deviation tidal elevation. The resulting elevation for the mean minus one standard deviation is + 7.5 feet MLLW. The actual tidal elevation has a 25 percent probability of being lower than this elevation. The net difference in the tidal elevation at +7.5 feet MLLW and the recommended groundwater elevation at +20 feet MLLW results in a significant increase in hydrostatic pressure for seismic design relative to what was used by PND for design. The sensitivity analysis in Section 5 considers the tidal elevation range from -5 feet MLLW to +34 feet MLLW.

It is important to note that even use of +7.5 feet MLLW as the basis of design for seismic loading is not the most severe (or bounding) load case. Each day there will be a 25 percent probability that tidal level will be lower than +7.5 feet MLLW. If the design seismic event occurs when the tide is below +7.5 feet MLLW, increased demand will occur on the OCSP® system, resulting in lower factors of safety for global stability and higher seismic deformations. These effects are discussed further in Section 5 of this suitability study report. The bounding tidal elevation of -5 feet MLLW was not selected as a basis of design in view of the very low likelihood that the earthquake and low tidal elevation would occur at the same time. Also, as the tidal elevation is decreased to the bounding value of -5 feet MLLW, the cost of meeting stability and displacement requirements for seismic loading increases. The value of +7.5 feet MLLW is believed to represent a reasonable compromise between the likelihood of occurrence and design conservatism.

For the static cases, the tidal elevation of -5 feet MLLW is considered appropriate for the End-of-Construction case and the Long-term Static-Undrained case because both cases assume a quick failure mode (refer to Section 2.1.6 for detailed discussion of loading cases); hence a potential for instability during the tidal state exists. The Long-term Static-Drained case assumes a slow failure mode, and the likelihood of tidal elevation remaining below -5 feet MLLW throughout the duration of failure is considered very low. A more appropriate risk level for the Long-term Static-Drained case is considered to be a tidal elevation of +7.5 feet MLLW. Again, the actual tidal elevation has a 25 percent probability of being lower than this elevation. However, because of the slow rate of shear that would be associated with the drained failure mechanism—possibly occurring over a period of hours or days—the tidal elevation at +7.5 feet MLLW is considered a reasonable bounding case. CH2M HILL's recommended hydrostatic loading assumptions are summarized in Table 2.1-5.

TABLE 2.1-5
Recommended Hydrostatic Loading Assumptions

	Limit Equilibrium		Numerical Modeling	
Loading Case	**Water Elevation In Front of Wall (feet)[a]**	**Water Elevation Behind Wall (feet)[a]**	**Water Elevation In Front of Wall (feet)**	**Water Elevation Behind Wall (feet)**
Static (Construction/Long-Term Undrained/Post-Earthquake)	-5	20	-5	20
Static (Drained)	7.5	20	7.5	20
Seismic	7.5	20	7.5	20

Note: Elevation datum is MLLW.
[a] Baseline modeling elevation, supplemented with sensitivity analysis in Chapter 5.

2.1.3.6 Hydrodynamic Water Loads

The presence of water can play a strong role in determining the loads on waterfront structures both during and after earthquakes, as discussed by Kramer (1996) and Ebeling and Morrison (USACE, 1992). Water outboard of a retaining wall can exert a temporary dynamic pressure on the face of the wall; water within the backfill can also affect the dynamic pressures that act on the back of a wall through either increased inertial forces or hydrodynamic effects. These hydrodynamic water pressures, which can act to stabilize or destabilize the structure, were not addressed in the PND and Terracon design documentation.

CH2M HILL's recommended design approach is to account for hydrodynamic pressure as follows:

- **Sea Side of Wall.** The hydrodynamic water pressure force on the outboard side of the OCSP® structure should be determined by using the Westergaard equation. The critical case for the bulkhead design is when this hydrodynamic pressure is acting away from the wall, i.e., seaward. In this condition the hydrodynamic water pressure is subtracted from the hydrostatic water pressure. The magnitude of the hydrodynamic water pressure force is given by the following equation:

  $$P_w = \frac{7}{12} K_h \gamma_w H^2$$

  Where k_h is the horizontal seismic coefficient, γ_w is the unit weight of water, and H is the water depth. This force is applied at 0.4H above the dredge line. It is a temporary force during the seismic event, and therefore, use of this force results in design conservatism.

- **Land Side of Wall.** The additional force in the saturated soil behind the OCSP® structure will depend on the relative movement between the backfill soil particles and the porewater that surrounds the particles, as discussed by Kramer (1996). If the permeability (k) of the soil is small enough (for example, $k \leq 10^{-3}$ centimeters per second [cm/sec]), then the porewater moves with the soil during the earthquake (that is, no relative movement of soil and water), which is referred to as a restrained porewater condition. In this case, the inertial forces will be proportional to the total unit weight of the soil. If the soil has a high permeability, the water moves independent of the wall and produces an added force defined by the Westergaard equation given above. Based on the fines content within the backfill material, CH2M HILL believes that it is reasonable to assume that the porewater moves with the soil, and restrained porewater conditions exist.

In summary, the recommended design criteria for hydrodynamic loads is to include the Westergaard pressure on the outboard side of the OCSP® system and to use total unit weight to estimate seismic earth pressures on the land side of the face of the OCSP® structure. These criteria increase load demands on the OCSP® system, relative to neglecting the hydrodynamic components, and as the demands increase, the tendency for lower factors of safety for global stability and larger seismic displacement increases. As discussed in Section 5, the impact of these hydrodynamic effects on factor of safety for global stability at POA was generally small.

2.1.3.7 Wind Loads

Environmental loads such as wind, wave, and currents generally do not directly control the design of the sheet pile bulkhead structures. They do, however, affect mooring and berthing of the ships and crane operations. Based on Chapter 6 of ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), the basic wind speed (3-second gust) in the vicinity of the POA area is approximately 100 miles per hour (mph). Adjusted to a 30-second duration, the design wind speed to be used in the mooring analysis is approximately 95 mph. This is roughly consistent with the wind load specified in the *PIEP Design Criteria Summary* (MARAD and ICRC, 2012), where the Non-Operational Wind Speed is set at 100 mph. The *PIEP Design Criteria Summary* also specified two Operational Wind Speed levels (45 mph and 70 mph). These wind speeds are only related to crane operations and do not affect the design of the bulkhead structure.

2.1.3.8 Current and Wave Loads

Knik Arm experiences large tidal currents, and expansion of the POA further into Knik Arm will increase the exposure of the OCSP® system to these currents. The POA, and particularly the North Extension and Barge Berth locations, is in the area south of Cairn Point in which an eddy or gyre forms during ebb tide as a result of the sheltering effect of Cairn Point. During flood tides, the site will see high currents paralleling the shore. During ebb tides, because of the formation of the gyre, nearshore currents are significantly reduced at the POA and can actually run counter compared to the currents in the main portion of Knik Arm. Modeling has shown that expanding the shoreline further into Knik Arm will squeeze the ebb tide gyre and result in higher nearshore currents during both phases of the tidal cycle.

Because currents will parallel the OCSP® system, current loads directly on the structure will be minimal. However, loads can be transmitted to the structure through their effects on ship mooring loads, through impacts of objects, and through ice carried by the currents. Currents will also result in drag forces on associated elements such as fender piling, which can be exacerbated during times of ice accumulation on the structure. Although drag forces on the fender piles are not a direct affect on the OCSP® system, the ability of these elements to withstand current loads will affect the project success. High currents can also result in additional scour at the site.

Design criteria specified in the *PIEP Design Criteria Summary* for currents are given as follows:

- Ebb Tide Average: 2.5 knots, SW
- Flood Tide Average: 1.5 knots, NE
- Ebb Maximum Average: 3 knots, SW
- Flood Maximum Average: 2.9 knots, NE

Flood tide current speeds appear reasonable based on available modeling results. Ebb tide flows appear high and in the opposite direction than would be expected because of the Cairn Point gyre.

Waves can affect loads on the structure either directly from the unbroken waves reflecting off the OCSP® system, or from wave action on ships at berth. Waves affecting the OCSP® system will affect the pressure distribution on the seaward face of the structure by adding a dynamic component from water accelerations within the waves to the hydrostatic component as a result of the weight of the water. This direct wave load effect on OCSP® bulkhead was not addressed in the previous evaluations.

The dynamic pressure from the wave can be calculated by using the Sainflou formula. The dynamic wave pressure will be additive to the seaside hydrostatic water pressure (i.e., acting towards the wall) when the crest of the wave reflects off the wall. The dynamic wave pressure will be subtractive to the seaside hydrostatic water pressure (i.e., acting seaward, away from the wall) when the trough is at the face. For bulkhead walls with saturated backfill, the dynamic wave pressure associated with the wave trough is often the critical case because the total seaside phreatic pressure is at its minimum. CH2M HILL believes that this critical case should be included in the internal stability evaluation of the OCSP® bulkhead. Because the reversible nature of the wave pressure and its short duration, dynamic wave pressure is typically not considered in the global stability evaluation.

Waves at the site are largely wind-generated from inside Knik Arm and Upper Cook Inlet. Design wave heights are 3.5 feet with a 4.5-second period from the west or 2 feet with a 3.5 second period from the north. These were based on 1-hour average 50-year return period winds of approximately 23 knots, which corresponded to winds from the west during summer months. Periods in which ice cover would be expected in Knik Arm were not included in the estimations of winds and waves for the site. Review of wind data from the Anchorage International Airport supports the magnitude of winds used. Wave heights are a function of both wind speed and the fetch over which the wind acts, and design wave heights appear reasonable for the wind speed and exposure of the site.

2.1.3.9 Impact Loads

Sheet pile bulkhead structures may be subject to impact damage by solid objects carried by waves, currents, or hurricane-force winds. Design of OCSP® sheet piles to resist impact loads during extreme events is difficult because of the uncertainty associated with impact speed and duration. At the same time, the likelihood of such damage occurring is probably low considering the extensive fender system that will be installed at the exterior of the bulkhead. The existing design criteria did not specify impact loads used during the design. For bulkhead design, CH2M HILL recommends consideration of only impact loads from ship berthing or ice transmitted indirectly through the fender system. A detailed discussion of ice and berthing load is presented in the following sections.

2.1.3.10 Ice Loads

Ice Floe Forces

Lateral ice loads on sheet pile bulkhead structures are primarily caused by dynamic pressure resulting from moving sheets of ice carried by wind or currents. The magnitude of the force will be a result of the driving force (wind or current) acting on the ice floe, the momentum of the floe, or the loads associated with localized failure (either through crushing or bending) of the ice. The crushing or bending strength of an ice floe will be the limiting force on a structure given sufficient driving force or ice floe momentum. Loads from crushing or bending of an ice floe will occur near the water surface and will be a function of both the thickness and strength of the ice.

Because the direction of lateral ice loads will be mostly in the opposite direction of the main forces causing global instability of the sheet pile bulkhead system (that is, forces resulting from static and dynamic earth pressure and phreatic water loads), it is not expected that ice loads will have any appreciable effect on the stability of the bulkhead. Although it would seem that localized damage could occur from moving ice floes, no damage has been reported on OPEN CELL® structures built in similar icy environments (PND, 2006d).

The *PIEP Design Criteria Summary* (MARAD and ICRC, 2012) specifies bending strengths of 25 to 40 psi, crushing strength of 150 psi, and an ice thickness of 24 inches. The *Extended Wet Barge Berth Design Manual* (PND, 2011a) provides similar ice strengths, but with an 18-inch ice thickness. AASHTO's *LRFD Bridge Design Specification, Customary U.S. Units* (AASHTO, 2010) reported effective ice strength up to 57.6 kips per square foot (ksf) (400 psi) have been used in the design of some bridges in Alaska in the past; however, PND suggests 32 ksf (222 psi) as the reasonable upper limit of ice crushing strength. Findings from the *Knik Arm Crossing Final Ice Condition Findings Technical Report* prepared by PND (2006a) indicate that ice in upper Knik Arm will have values of compressive strength in the range of 200 to 300 psi, and structures designed for ice crushing strengths of 300 psi or less have been successful in Cook Inlet.

For bulkhead design, CH2M HILL recommends using 300 psi as ice crushing strength and 24 inches as design ice thickness. Additional discussion is included in Section 4.3.

Ice Abrasion and Ice Collars

Two other potential sources of ice damage to the OCSP® involve abrasion at the face of the OCSP® system as currents in Knik Arm drive ice floes against the OCSP® and ice collars that develop on fender piles at the face of the OCSP. Based on experience at the POA, the abrasion has not been a significant issue because the ice buildup forms collars or cocoons that insulate the piling from ice abrasion. The ice collars can be accounted for in the design of the structures as additional dead load.

2.1.3.11 Berthing and Mooring Loads

Wind, waves, current, ice, tidal variations, and passing vessels all generate forces acting on a moored vessel. These forces are transmitted through mooring lines to the bulkhead. At the same time, berthing loads are defined in terms of kinetic energy needed to bring a moving vessel to rest against the bulkhead. The *PIEP Design Criteria Summary* (MARAD and ICRC, 2012) specified the following values for the berthing and mooring loads:

- Fender Energy: 200 ft-kips
- Maximum Fender Reaction: 172 kips
- Mooring Load on Bollards: 100 tons

These values appear to be generally appropriate for the site condition and the type of vessel to be expected at POA. According to existing PIEP plans, both the mooring bollards and the fenders will be attached to a continuous reinforced concrete utilidor (shown in Figure 2.1-1) connected rigidly to the top of the OCSP® system. Compared with the OCSP® structure, the concrete utilidor is relatively stiff and thus will distribute any mooring and berthing loads to a large section of the bulkhead. Because of this redistribution effect, mooring and berthing loads should not control the OCSP® system design.

2.1.3.12 Summary of Recommended Design Loads

Table 2.1-6 summarizes the design load values for the OCSP® bulkhead used in the original PND design and CH2M HILL's recommended values.

TABLE 2.1-6
Original and Recommended Design Loads for OCSP® Bulkhead

Design Loads	Original Design	CH2M HILL Recommendation
Dead Loads	Not Defined	Table 2.1-1
Live Loads (Static)	200 psf within 200 ft of bulkhead control line, 1,000 psf beyond[a]	1000 psf
Live Loads (Seismic)	0 psf	200 psf
Earthquake Loads	Table 2.1-2 and Table 2.1-3	Same as original design
Hydrostatic Water Loads	Table 2.1-4	Table 2.1-5
Hydrodynamic Water Loads	Not Considered	Westergaard Equation
Wind Loads	100 mph[b]	Same as original design
Wave Loads on Bulkhead	Not Considered	3.5 ft with 4.5 sec period
Ice Loads	• Bending Strength: 25-40 psi • Compression Strength: 150 psi • Ice Thickness: 24 inch	• Bending Strength: 25-40 psi • Compression Strength: 300 psi • Ice Thickness: 24 inch
Berthing and Mooring Loads	• Fender Energy: 200 ft-kips • Maximum Fender Reaction: 172 kips • Mooring Load on Bollards: 100 tons	Same as original design

[a] Changed in 2011 to 1000 psf throughout; it is not clear whether all the original analyses were repeated using higher live load.
[b] Non-operational wind speed.

2.1.4 Material Properties

According to existing PIEP plans, all sheet piles, wyes, anchors and accessories used to construct the OCSP® structure conform to ASTM A328 "Standard Specifications for Sheet Piling" (ASCE, 2007b) and ASTM A6 "General Requirements for Rolled Structural Steel Bars, Plates, Shapes and Sheet Piling" (ASCE, 2011). PIEP plans also specify that steel sheet piling "shall have similar A572 Grade 50 steel chemistry." These requirements are interpreted as meaning that all steel sheet piles, wyes, anchors, and accessories will conform to the chemistry requirement of ASTM A572, Grade 50, and would have a yield strength of 50 kips per square inch (ksi) and a minimum tensile strength of 65 ksi (ASTM, 2007a).

The strength of sheet piling used in cellular cofferdam structures such as the OCSP® system is controlled by both the yielding of the web and failure of the interlock. The existing PIEP plans require "all interlock group tests shall provide a minimum of 20,000 pounds per linear inch ultimate interlock tensile strength." This is in agreement with manufacturer-provided data for the sheet piling (L.B. Foster, 2011). Other sources (USACE, 1989) indicate that minimum interlock strength of up to 28,000 pound/inch (with reduced swing angle) is readily achievable for A572 Grade 50 sheet piling.

Two sheet pile sections are specified on the PIEP plans: PS31, and PS27.5. Their section properties and allowable stress are presented in Figure 2.1-2 and Table 2.1-7. PS31 sheet piles have a web thickness of 0.5- inch and are mostly used on face sheets where the piling is subject to large hoop tension. PS27.5 sheet piles have a web thickness of 0.4-inch and are mainly used for tailwalls. Based on documents provided by PND (PND, 2007c), connection elements, such as wyes and weld-on connections, are shown to have a minimum tensile strength of at least 20,000 lbs/inch during load testing.

TABLE 2.1-7
Dimensions and Ultimate Interlock Strength for PS31/PS27.5 A572 Gr.50 Sheet Piling

	Dimensions		Ultimate Interlock Strength
Section	**Width (in)**	**Web Thickness (in)**	**Horizontal Tension[a] (kip/in)**
PS 27.5	19.69	0.40	20
PS 31	19.69	0.50	20

[a] Including both interlock tension and web yielding.

The sheet pile sections that are being used for the OCSP® system are reasonable as long as proper installation methods are used. However, given the large overall length and exposed height of the sheet piles used in this project (that is, exposed height of the wall exceeds 80 feet at the tallest section) and the slenderness of the sheets, the planned design is expected to be more sensitive to means and methods used during construction than in other locations. In fact, in one of the papers published by Dennis Nottingham, one of the main inventors of the OCSP® system and former president of PND, Inc., it was suggested that wall heights of about 18 meters (60 feet) with 24-meter (80-foot) sheets are a practical limit for this type of construction: “longer sheets are difficult to handle and drive and are not advisable” (Nottingham, 1995). Refer to Section 8 for more detailed discussion of constructability issues.

2.1.5 Scour

Reduced seabed elevations fronting the sheet pile bulkhead can occur as a result of localized scour associated with placement of the structures and operations at the POA, as well as more global changes resulting from global erosion and bed morphology within Knik Arm. The OCSP® sheet pile walls were designed for a seabed elevation of -51 feet MLLW, which provides a 6-foot tolerance beyond the 45-foot nominal dredging depth for over-dredging allowance (2 feet) and storage dredging (4 feet). Available information from the *Extended Wet Barge Berth Design Manual* by PND (2011a) suggests that scour has not been observed in this area and a scour apron can be added if the problem arises.

It appears that scour was not addressed explicitly by the original design team and was assumed to be minor and combined with the dredging tolerance or considered to be an issue that could be dealt with if the need arose. The tips of both the OCSP® sheet piles and the fender piles are at -60 feet MLLW, for a 9-foot burial depth beyond the dredging and scour tolerance assumed.

It is likely that scour will occur around the bases of the 36-inch-diameter fender piling. Scour around piling under strong currents can be on the order of two times the pile diameter or more. Seasonal changes in the bathymetry offshore of the POA have been observed in survey data with decreases in seabed elevations on the order of 5 to 10 feet in some locations over the winter season. However, these changes have largely been offshore from the site of the expansion, and the data that were centered on the first phase of the expansion were historically stable.

If over-dredging to -51 feet MLLW is expected, an additional 5-foot scour allowance needs to be considered to account for scour induced by the fender system piling and other mechanisms, such as propeller wash from ships during berthing. This is believed to be a conservative value for the purpose of design. Sedimentation at the berths that will occur in the spring and summer should mitigate for the potential scour; however, other factors such as increased current velocities at the location of the expanded port, natural variability in suspended sediment concentrations carried to the site (both seasonally and from year to year), and changes in the hydrodynamics of the area can have an effect on whether, and the extent to which, sediment deposition occurs. In view of the potential for additional scour, an assessment of the effects of scour to -55 feet MLLW on global stability was performed, as discussed in Section 5.3. Because of various uncertainties associated with scour occurrence and the potential impact that additional scour could have on the OCSP® system, regular monitoring of scour along the dock face should be performed and scour protection added, if necessary.

2.1.6 Loading Cases and Load Combinations

CH2M HILL has identified the following loading cases, presented in Table 2.1-8, as being considered in the original design. These load combinations are based on review of existing design criteria and design calculations.

TABLE 2.1-8
Loading Cases Considered in the Original Design

	Loading Cases				
	Short-Term Static	**Long-Term Static**	**Seismic (OLE)**	**Seismic (CLE)**	**Seismic (MCE)**
Load Combinations	DL+LL+PH	DL+LL+PH	DL+OLE	DL+CLE	DL+MCE

DL = dead load; soil load and pavement load
LL = live load; 200-psf within 200 feet of face, 1,000 pounds per square foot thereafter
PH = phreatic load; el. +18-ft MLLW inside wall, el. -5-ft MLLW outside wall
OLE = operating level earthquake, average maximum horizontal earthquake acceleration (MHEA) = 0.20g
CLE = contingency level earthquake, average MHEA = 0.27g
MCE = maximum considered earthquake, average MHEA = 0.41g

The original loading cases are rational; however, several criteria and failure modes were not explicitly considered in the original design, including the following:

- The live load was not considered for any of the seismic loading cases.
- The allowable sheet pile tensile and interlock stresses to be used in each loading case were not clearly specified.
- The post-earthquake condition was not considered. When soil liquefaction or cyclic degradation of the BCF clay occurs as a result of seismic shaking, or when large seismic deformations accumulate during shaking and cause shear strength reduction approaching residual strength, the post-earthquake stability of a structure is often checked. The factor of safety under these static conditions must exceed 1.0. The reason for PND's exclusion of this load case appears to involve the assumptions that, for seismic analyses, the in situ dense granular soils within the BCF interbeds will not liquefy, and that the cohesive soil will lose little strength under design earthquake loading.

For the as-built analyses of the OCSP® system, the following seven loading cases are considered appropriate based on CH2M HILL's review of information: (1) End-of-Construction, (2) Long-Term Static-Drained, (3) Long-Term Static-Undrained, 4) Seismic – OLE, (5) Seismic – CLE, (6) Seismic – MCE, and (7) Post-Earthquake. Each of the recommended loading cases involves different assumptions regarding the strength of the soil, the tidal elevation, the seismic shaking hazard, and the amount of live load that is applicable. These assumptions are summarized as follows:

- **End of Construction.** With consideration of the duration of construction and the consolidation coefficient of the BCF clay, the short-term static condition (for example, End-of-Construction) assumed a consolidated, undrained shear strength for the foundation material. The ground elevation behind the face of the OCSP® system for this case is at elevation +35 feet, prior to any surfacing of the POA facility. A uniform live load of 200 psf was used to account for the presence of construction equipment and possible, temporary material stockpiling. A low tidal water elevation of -5 feet is assumed, appropriate for a quick-failure condition.
- **Long-Term Static-Drained.** The long-term static-drained condition assumed a long-duration failure, in which effective shear strengths (that is, without shear-induced porewater pressures) of the BCF clay are operative. Consistent with a long-duration failure, a tidal water elevation of +7.5 feet was assumed. The full design live

load of 1,000 psf was applied to the top of the structure (elevation +38 ft with surfacing) as a uniform pressure.

- **Long-Term Static-Undrained.** The long-term static-undrained case was included in the analyses to account for the potential for a rapid failure to occur during extreme low tide at elevation -5 feet. Justification for the parameters of this case comes from observations of failures of earth structures, including failure of the OCSP® system at the Skagway Marine Terminal (Cornforth, 2005). In the Skagway slide, failure occurred at extreme low tide, and the time for failure was about 11 seconds—too short to allow for any dissipation of excess porewater pressures. For the long-term static-undrained condition, it was assumed that BCF clay was fully consolidated under the weight of the fill and application of live load (with an associated gain in shear strength), but that the soil exhibits undrained shear behavior. The full uniform live load of 1,000 psf was assumed for this long-term static case because the failure could occur at any point throughout the life of the facility.
- **Seismic OLE, CLE, and MCE.** The assumed seismic conditions included a reduction in consolidated-undrained shear strength in the BCF clay to account for potential cyclic degradation of the material strength and strain-softening behavior of the material at large, accumulated displacements. In estimating undrained strength, the live load was included in the effective overburden stress (input into the shear strength model). For all three seismic cases, inertial forces resulting from ground shaking were combined with gravity loading such as dead load and live load. A live load reduction factor of 0.2 was applied to account for the low possibility of concurrent occurrence of full live load with inertia loading, thus resulting in a uniform pressure of 200 psf applied to the surfaced facility at elevation +38 feet. The baseline tide elevation for all seismic cases was assumed to be at elevation +7.5 feet MLLW and supplemented with sensitivity analysis over a tidal elevation range from -5 feet MLLW to +34 feet MLLW.
- **Post-Earthquake Loading.** The post-earthquake condition refers to conditions where the soil strength is reduced from the best-estimate consolidated-undrained shear strength to some lesser strength as a result of seismic shaking or changes in particle orientation occurring under large strains that accumulate during shaking. The post-earthquake shear strength of the BCF clay was estimated as a function of seismic-induced permanent deformation, as discussed in Section 5. This loading case considers only gravity load (that is, static case) with a required factor of safety for global stability equal to 1.0. A live load of 200 psf was included. The low tide elevation of -5 feet MLLW was assumed, consistent with a rapid, undrained failure mode and assuming limited strength recovery in the time the tide drops. When the yield acceleration is observed to drop to 0.00 g during seismic shaking (as a result of large strain-induced strength reduction of clay), the post-earthquake loading condition does not necessarily need to be checked, because it is understood that the factor of safety is less than 1.0.

Load combinations for each of the above loading cases for service load design (SLD) are presented in Table 2.1-9.

2.2 Performance Criteria Review

In the existing design, the performance of the OCSP® system under the design loads was evaluated by PND in terms of factors of safety and deflections, as follows:

- **Sliding**. Sliding was checked by PND using a limit-equilibrium method considering a block-type failure surface to represent the mode of failure. This approach can be unconservative because it includes soil shearing resistance along the back plane. In the CH2M HILL study, sliding was checked using state-of-the-practice calculation methods. The factor of safety requirements are the same as for global stability (presented in Table 2.2-1).
- **Internal Stability**. Internal stability includes sheet pile interlock strength as well as tailwall pullout strength. The required factors of safety stated for the Static, CLE, and MCE cases are 1.5, 1.3, and 1.1, respectively. No criteria were found for the OLE loading case.

TABLE 2.1-9
Service Load Design Load Factors for Load Combinations[a]

Loading Case	D Dead Load	L Live Load	E Earth Pressure Load	W_p Phreatic Water Load	W_{HD} Hydro-dynamic Water Load	EQ Earthquake Load	ICE Ice Load	Allowable Stress
End of Construction	1.0	1.0[b]	1.0	1.0	-	-	-	100%
Long-term Static (Undrained)	1.0	1.0	1.0	1.0	-	-	1.0	100%
Long-term Static (Drained)	1.0	1.0	1.0	1.0	-	-	1.0	100%
Seismic – OLE	$1.0 \pm K_v$[c]	Γ_{LL}[d]	1.0	1.0	1.0	Γ_{EQ}[e]	-	100%
Seismic – CLE	$1.0 \pm K_v$[c]	Γ_{LL}[d]	1.0	1.0	1.0	Γ_{EQ}[e]	-	100%
Seismic – MCE	$1.0 \pm K_v$[c]	Γ_{LL}[d]	1.0	1.0	1.0	Γ_{EQ}[e]	-	100%
Post-Earthquake	1.0	Γ_{LL}[d]	1.0	1.0	-	-	1.0	100%

[a] Load Symbols:
D = dead load
L = live load
E = earth pressure load
W_p = phreatic water load
W_{HD} = hydro-dynamic water load
EQ = earthquake load
ICE = ice load
[b] Construction live loading (200 psf) should be considered in combination with dead load for the construction case.
[c] $K_v = \alpha K_h$, where K_h = 0.5*Peak Ground Acceleration, to account for the effects of vertical component of ground acceleration. Various codes/criteria have recommended different values of α, including zero (POLB, 2009; UFC, 2005), 0.7 (California Building Code, 2010), and 1 (POLA, 2010). We recommend using $\alpha = 0.0$ for the study.
[d] Γ_{LL} – Various codes/criteria have recommended different live load factors when combined with earthquake loads, including zero (California Building Code, 2010), 0.1 (POLA, 2010; POLB, 2009), 0.05-0.25 (ASCE, 1998), and 0.2 (PIANC, 2001).We recommend using Γ_{LL} = 0.2 for the study.
[e] Γ_{EQ} – Again, various codes/criteria have recommended different load factors for earthquake loads, ranging from 0.7 (California Building Code, 2010; ICC, 2009) to 1.0 (POLA, 2010; ASCE, 1998). We recommend using Γ_{EQ} = 1.0 for the study.

- **Bearing Capacity**. Bearing capacity is implicitly checked via the numerical modeling. No failure criteria were stated.
- **Global Stability.** Global stability analyses involve a check against failure along a surface that extends underneath the tailwall and face sheets. Stability and seismic performance criteria are discussed in Section 2.2.1.
- **Liquefaction**. The required factors of safety against liquefaction of granular backfill for the OLE, CLE, and MCE were all 1.1. For the liquefaction analyses documented in Section 5, earthquake moment magnitudes (M_w) of 7.5 and 9 were considered in the Seed-Idriss simplified blow count method to determine the factor of safety. Also, the analyses were repeated using the modal mean magnitudes from probabilistic seismic hazard analyses by URS. The moment magnitudes (M_w) for these analyses were 6.1, 6.3, and 6.6 for OLE, CLE, and MCE events, respectively.

The desired OCSP® system performance (deformation) and global stability factors of safety (FS) criteria are summarized in Table 2.2-1 (ICRC, 2008). The assumed pseudo-static coefficient was equal to one-half the PHGA for each design earthquake condition. No supporting information was provided by ICRC or PND for the FS and displacement values provided in Table 2.2-1.

TABLE 2.2-1
Performance and Global Stability Criteria for OPEN CELL® Wharf (ICRC, 2008)

Loading Condition	Deformation Based Criteria Description	Allowable Bulkhead Deformation (inch)	Global Stability Factor of Safety (FS) Based Criteria
Short-term static	Moderate bulkhead movement without overstressing of structural components	Less than 18	1.3
Long-term static	Moderate bulkhead movement of structural components	Less than 18	1.5
Seismic: OLE	Very little additional bulkhead movement beyond static loading condition – damage repairable in a short time period and no interruption to wharf operations	Less than 3 (permanent)[a]	1.2
Seismic: CLE	Small additional bulkhead movement beyond static loading condition – damage repairable with minimal interruption to wharf operations	Less than 6 (permanent)[a]	1.1
Seismic: MCE[b]	Moderate additional bulkhead movement beyond static loading condition – moderate damage but economically repairable with some significant interruption to damaged portions of wharf operation	Less than 18 (permanent)[a]	1.0

[a] Displacements under seismic conditions are additional to those from static conditions. Temporary wall movements during a seismic event might exceed permanent wall displacements at the end of an earthquake.

[b] MCE only considered for essential facilities in North Extension/barge berths.

Note that neither overturning nor settlement was explicitly considered in the performance criteria:

- **Overturning.** The ICRC (2008) report cites the high aspect ratio (tailwall length to wall height) of the structure as a basis for not performing this analysis. No minimum criteria were presented for this failure mode (for example, factor of safety or deformation limit). CH2M HILL concurs with this assessment.
- **Settlement.** The performance criteria also did not include any specific settlement requirements. In general, the construction duration was expected to be long enough that consolidation settlement would be within tolerable limits by the start of operations. Crane rails will be pile-supported to minimize the effects of long-term settlement resulting from secondary compression of the BCF clay. This approach to settlement is reasonable for the PIEP.

2.2.1 Factors of Safety for Global Stability

The FS for global stability criteria given in Table 2.2-1 refer to stability of the OCSP® system under gravity, seismic, and other forms of loading. Instability occurs when the load imposed on the OCSP® system exceeds those forces resisting movement, which will generally be the strength of the soil. For the OCSP® structure, the form of global instability is a translational or rotation slip of the structure toward the water. The amount of movement can range from a few inches to many feet, depending on the specifics of loading and the strength of the soil.

The standard-of-practice in geotechnical engineering of embankments and retaining structures, such as an OCSP® structure, is to confirm that the system meets global stability requirements under different stages of loading. The stages of loading include immediately after construction, under long-term loading conditions, and during the three seismic loading conditions. These loading cases are associated with different soil resistance conditions, as well as different assumptions regarding the operational loads that exist for the facility. The approach identified in Table 2.2-1 is consistent with this approach.

Global stability values developed by PND and given in Table 2.2-1 are consistent with the standard-of-practice in geotechnical engineering. Loading conditions and soil strength values associated with each loading condition are summarized in Section 5.

2.2.2 Deflection Criteria

Table 2.2-1 includes a summary of allowable bulkhead deformations and was developed by PND as a basis of their design. For each of the loading conditions given in Table 2.2-1, acceptable behavior occurs when the predicted level of displacement is less than the displacement identified by the criteria.

Based on discussions between CH2M HILL and POA staff, CH2M HILL understands that PND established the acceptable levels of bulkhead displacement on the basis of the amount of movement that can be tolerated before disruption of the Port facilities from loss of utilities, restrictions to movement of vehicles and other machinery, and other operations-related issues. Values given in Table 2.2-1 were developed by PND after discussions with the POA and their operations personnel, as well as from PND's stated experience from similar wharf projects on the West Coast.

Values of allowable OCSP® system displacement range from 18 inches during short- and long-term gravity loading to incremental increases in displacement of 3, 6, and 18 inches of permanent movement during seismic loading from the OLE, CLE, and MCE, respectively. The seismic displacements are in addition to displacements that develop during construction of the OCSP® system. The deflection criterion identified by PND for static loading—deflections that are less than 18 inches—is not unreasonable for a sheet pile structure. The outward movement of the sheet pile face is under 3 percent of the wall height, and this amount of movement appears to be acceptable. Strains within the sheets are well within allowable levels for this amount of movement, and it is unlikely that these movements would affect the performance of the OCSP® system.

Incremental deflections given in Table 2.2-1 for seismic loading are also within levels that would likely be acceptable, as long as the deflection is relatively uniform across different portions of the structure. These incremental loadings are in addition to those that occur during construction and operational loading conditions (that is, defection at the start of seismic event was assumed to be zero).

For the OLE, which is likely to occur during the service life (50 years) of the OCSP® structure, incremental displacements of 3 inches should have little effect on the sheet piles. The function of crane rails or utilities associated with the POA should not be affected if the relative displacement between the different parts of the structure (for example, between the two crane rail beams and between the crane rail beams and the utilidor) is small. If significant relative displacement between different parts of the structure exists, the impact of such relative displacement should be considered in the design of the affected structure component.

For the CLE, larger incremental movements are allowed. This additional movement results in additional risk to the sheet piles and port facilities; however, the additional movement is still fairly small, relative to this type of structure. The Port of Los Angeles allows up to 12 inches of movement during the CLE for their pile-supported structures. The Port of Los Angeles criterion is based on strains in prestressed concrete piles. The OCSP® is probably as tolerant of displacements as a pile-supported structure, suggesting that the 6-inch limit may be too conservative. However, this also depends on the type of other backlands facilities that could be affected by the movement.

For the MCE, the Essential Facilities in the North Expansion are allowed to displace up to 18 inches during the MCE. For this amount of movement, PND expects the possibility of significant interruption of operations and damage, but the degree of damage is expected to be economically repairable. Again, the performance requirement appears to be either consistent with or perhaps somewhat conservative. Most likely movement of the OCSP® system will occur as a block, resulting in limited distortion to the facing sheets and the tailwalls for movement up to several feet.

None of these performance requirements and deformation limits appears to represent life safety issues.

2.2.3 Internal Stability

The following failure modes need to be prevented in order to ensure the internal stability of the OCSP® bulkhead:

- Pile interlock tension failure
- Pullout of tailwall

Because of the geometry of the OCSP® system, other failure modes common to typical sheet pile cellular cofferdam structures, such as shear failure within the cell, pullout of outboard sheets, and penetration of inboard sheets, are either not applicable or are unlikely to control the design. The existing design criteria did not distinguish between the two failure modes. In existing design calculations, PND used factors of safety of 1.5, 1.3, and 1.1 for internal stability of static, CLE, and MCE cases, respectively. No criterion was found for the OLE case.

CH2M HILL recommends using different factor of safety values for interlock tension and tailwall pullout because of the fundamental differences between the failure modes. As discussed in Section 1.2.1, pile interlock tension is crucial to the integrity of the OCSP® system. Interlock tension is not only relied on to maintain the arched shape of the face sheets in order to retain soil at the face of the wall, but also needed to transfer the lateral loads acting on the face sheets to the tailwalls and anchors. The loss of interlock tension will result in collapse of one or more cells within the OCSP® system. CH2M HILL believes higher factors of safety (2.0 for long-term static case) are justified for interlock tension. This is also consistent with standard of practice within the industry (USACE, 1989; U.S. Steel, 1984). As discussed earlier in this section, the OCSP® system will likely survive permanent slip of the tailwall as long as failure of the tailwall does not result in a global failure. As a result, lower factors of safety are recommended for pullout than interlock tension, as shown in Table 2.2-2.

TABLE 2.2-2
Required Factor of Safety (FS) for Internal Stability

	Loading Conditions				
Failure Mode	**Short-term Construction**	**Long-term Static/Post-EQ**	**OLE**	**CLE**	**MCE**
Interlock Tension	1.5	2.0	1.5	1.3	1.1
Pullout of Tailwall	1.3	1.5	1.3	1.1	1.0

Note: Factor of safety defined as allowable resistance divided by demand force.

2.3 Summary of Findings and Recommendations

Section 2.1 provides a review of the design criteria used in the original PND design, including design codes, design loads, material properties, scour, and load cases. After reviewing the design codes quoted in the original design criteria, CH2M HILL has identified additional codes or updated versions that should be used to establish a basis of any future design. Although the stated design service life of 50 years appears to be appropriate and is consistent with what is used in other ports around the United States, consideration should be given to use of a design life of 75 years for any future facility.

The design loads assumed by the original design appear to be largely reasonable. However, CH2M HILL has identified some additional items that would normally be included in design. The original design did not account for live loads during seismic analysis. CH2M HILL recommends considering a portion of the live load (200 psf) in seismic analysis. CH2M HILL also recommends revising phreatic water load assumptions that would lead to a significant increase in hydrostatic pressure for seismic design. Table 2.1-6 summarizes design loads used in the original design and CH2M HILL's recommended values.

The specified material properties and section properties for the sheet piles, wye connections, and anchors appear to be generally appropriate. However, the large overall length and exposed height of the sheet piles used in the PIEP means the planned design is expected to be more sensitive to means and methods used during construction than in other locations.

It appears that scour was not addressed explicitly by the original design team and was assumed to be minor and combined with the dredging tolerance or considered to be an issue that could be dealt with if the need arose. Based on CH2M HILL's review, an additional 5-foot scour allowance needs to be considered to account for scour induced by the fender system piling and other mechanisms. Because of various uncertainties associated with scour occurrence and the potential impact that additional scour could have on the OCSP® system, regular monitoring of scour along the dock face should be performed and scour protection added, if necessary.

The original loading cases are rational; however, a few criteria and failure modes were not explicitly considered in the original design, including live load for seismic cases, allowable stress for sheet pile, and post-earthquake condition. CH2M HILL's recommended load cases and load combinations are summarized in Table 2.1-9.

Section 2.2 provides a review of the performance criteria used in the original design, including factors of safety for global and internal stability and deflection criteria for seismic cases. It appears that the global stability and deflection criteria used in the original design, as identified in Table 2.2-1, are reasonable and consistent with the standard-of-practice in geotechnical engineering. Considering the importance of interlock tension to the integrity of the OCSP® system, CH2M HILL recommends higher factors of safety for interlock tension than values used in the original design.

It was clear from the review of the PND project documentation that a number of design decisions had to be made by the design team during development of the PIEP. The PND design team relied on published information from various codes and standards, including other West Coast ports, as a basis of design. Given the unique environmental loading conditions that exist at the POA, CH2M HILL believes that it would be in the best interest of the POA to developing a formal standard for design and performance for future development, including repair of structures. This approach has been taken by other major West Coast ports, such as the Port of Los Angeles and Port of Long Beach. A design standard would avoid any confusion regarding methods allowed for design, particularly in the areas of tidal conditions and seismic loading. One of the requirements in the design standard should be preparation of a basis of design document that would explicitly identify codes and standards that have been used, assumptions on load combinations, load and resistance factors, and the like.

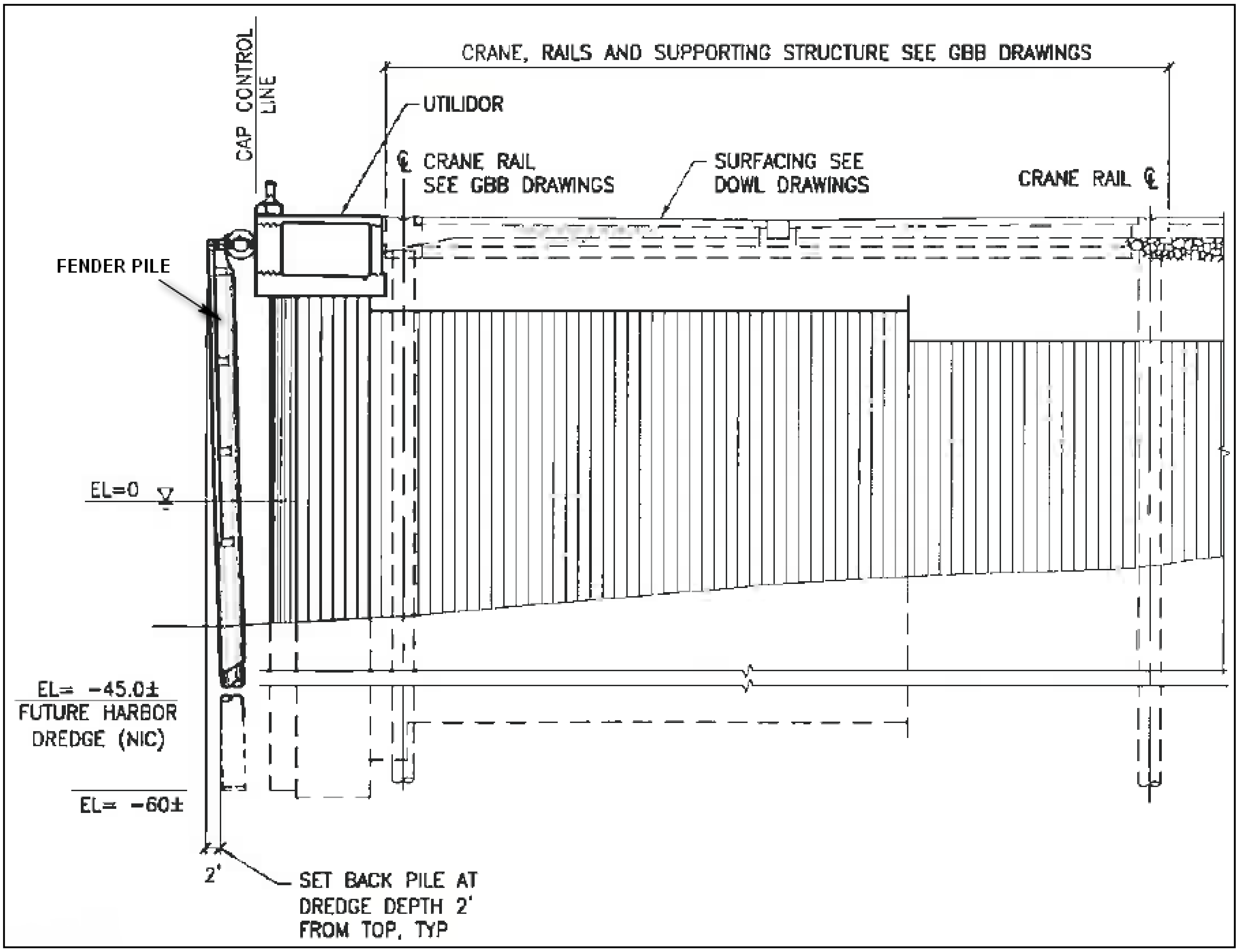


FIGURE 2.1-1. Typical Section of the OCSP® Wharf Showing Fender Pile, Utilidor, and Crane Rails

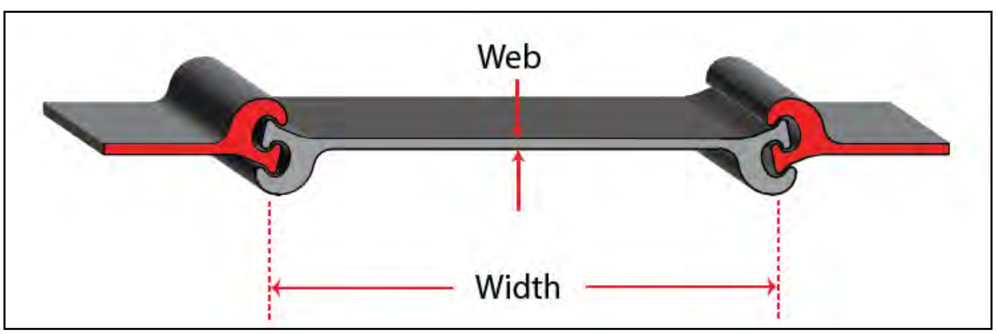


FIGURE 2.1-2. PS31/PS27.5 Flat Sheet Piling (L.B. Foster, 2011)

SECTION 3

Seismic Ground Motion Hazard Assessment

This section discusses results from a review of firm-ground, peak horizontal ground acceleration (PHGA) and response spectra used as input for the seismic ground motion hazard assessment conducted for the PIEP and from results of CH2M HILL's site-specific, one-dimensional ground response analyses conducted to estimate ground motions for CH2M HILL's design evaluation. The review of firm-ground motions was limited to the URS Corporation (URS) probabilistic and deterministic seismic hazard analyses (PSHA and DSHA) conducted in 2008. The PSHA and DSHA information defined input conditions for site-specific ground response analyses conducted by the PND design team and by CH2M HILL for this suitability study. The discussion of ground response analyses reviews the approaches, assumptions, and results of the seismic ground motion hazard assessment for the North Expansion of the PIEP.

This seismic ground motion hazard assessment provides design seismic parameters for the OCSP® global stability analysis and input ground motions needed for the FLAC^{3D} numerical analysis, as discussed in Sections 5 and 7 of this report, respectively.

3.1 Review of Firm-Ground Input Motions

URS conducted PSHA and DSHA for the POA site and provided results of their analysis in 2008 (URS, 2008). The intent of the URS PSHA and DSHA analyses was to develop ground motions that could be used as a basis for site ground response and soil-structure interaction analyses by the PND design team. Previous evaluations of seismic ground shaking hazards at the POA site were based on U.S. Geological Survey (USGS) seismic hazard studies that dated from the 1990s. CH2M HILL's review of the URS report was limited to checks on ground motion predictive equations (GMPEs) and general source characterization. Detailed information regarding the URS seismic model was not presented in their report, and conducting an independent PSHA was beyond the scope of this suitability study.

3.1.1 Overview of URS PSHA and DSHA

The results of URS's PSHA and DSHA analyses established ground motions at different probabilities of occurrence, which were used to define response spectra associated with the OLE, CLE, and MCE. The OLE, CLE, and MCE were defined to have a 50 percent, 10 percent, and 2 percent probability of being exceeded, respectively, during a specified exposure period of 50 years. The corresponding average return periods for these three levels of exceedance are 72, 475, and 2,475 years. Note that although these three design spectra are defined using the results of the PSHA, deterministic response spectra were also considered in developing the CLE and MCE time histories for use in site response analysis (see Section 3.2 below).

The ground motions for the OLE, CLE, and MCE defined the PHGA, as well as spectral accelerations for a 5 percent damped response spectra at selected periods of up to 2.0 seconds. When developing their seismic hazard model for the PSHA and DSHA, URS considered the most recent seismic source characterization, the historical and contemporary seismicity for the area, the ground motion attenuation models, and views on epistemic and aleatory uncertainties. The ground motions were applicable to the overall PIEP, in the sense that separate firm-ground ground motions were not computed for individual facilities in the North Expansion area. The modeling method used by URS does not have the precision to make more site-specific distinctions.

Results of the URS seismic hazard model defined response spectra at the firm-ground level. The firm-ground level was defined by URS based on review of available information as being located approximately 500 feet below the ground surface. URS produced uniform-hazard response spectra for ground motion return periods of 72, 475, and 2,475 years, as well as deterministic response spectra for the mega-thrust earthquakes. A seismic event on the mega-thrust source zone was the cause of the 1964 Alaska earthquake. URS also identified and modified a suite of 15 earthquake records that would be suitable for representing ground motions for the PIEP, and they determined V/H (vertical/horizontal) ratios to assist in the development of vertical response spectra.

URS compared their results to a ground motion seismic hazard map developed by the USGS for Alaska in 2007 and concluded that their site-specific study was approximately 15 to 23 percent lower than computed by the USGS. The difference was attributed to differences in partitioning of hazard contributions and the use of different GMPEs. The URS site-specific study identified the subduction intraslab source that underlies the POA site as the dominant seismic source for the ground motions in the period range of interest, whereas the USGS identified the background seismicity as the dominant seismic source. URS also used the recent Next Generation Attenuation (NGA) relationships to represent crustal and background earthquakes, as summarized in Table 3.1-1; this is consistent with the recent understandings and seismic data (up to 1997 Chi-Chi, Taiwan, and 1997 Kocaeli, Turkey, earthquakes). Note that the NGA models also include the 2008 Idriss relationship, which was not used in the URS study, probably because it was not available at the time of the URS study in 2008.

TABLE 3.1-1
Comparison of Ground Motion Predictive Equations (GMPEs) used by USGS and URS

	USGS (2007)		URS (2008)	
Seismic Source	**Relationship**	**Weight**	**Relationship**	**Weight**
Crustal & Background earthquakes	• Abrahamson & Silva (1997)	25% each	• Chiou & Youngs (2008)	25% each
	• Boore et al. (1997)		• Campbell & Bozorgnia (2007)	
	• Sadigh et al. (1997)		• Abrahamson and Silva (2008)	
	• Campbell & Bozorgnia (2003)		• Boore & Atkinson (2007)	
Interface (Megathrust)	• Youngs et al. (1997)	50% each	• Youngs et al. (1997)	40%
	• Sadigh et al. (1997)		• Atkinson & Boore (2003)	40%
			• Gregor et al. (2002)	20%
Intraslab	• Youngs et al. (1997)	50% each	• Youngs et al. (1997)	50% each
	• Atkinson and Boore (2003)		• Atkinson & Boore (2003)	

URS considered the different GMPEs in their PSHA by assigning different weights to the various relationships. These weights are given in Table 3.1-1. Each weight represents the percentage contribution to the hazard analyses provided by the particular GMPE; this represents URS's judgment on the likelihood of the particular GMPE being correct. CH2M HILL believes that these weightings are reasonable and consistent with values used on other similar projects.

Fifteen new horizontal earthquake time histories were also developed by URS from the eight seed time histories listed in Table 3.1-2 below (taken from the report). The two time histories selected for the interface/megathrust earthquakes are synthetic.

3.1.2 Observations from Review of URS Ground Motions

The methodology followed by URS in developing firm-ground ground motions (that is, 500 feet below the ground surface) for the PIEP is consistent with the approach currently being used for major projects relative to source characterization, earthquake activity, ground motion attenuation, and treatment of uncertainty. In ground motion modeling efforts there is always some variation in approach, usually based on professional judgment or opinion, particularly in the use of GMPEs. Differences in GMPEs that were noted from CH2M HILL's suitability review and their implications are discussed below. Also addressed is the use of synthetic motions that were identified by URS for representing mega-thrust earthquakes.

TABLE 3.1-2
Summary of Seed Time Histories

Station Name	NEHRP Category Based on Vs_{30}	Event Name	Date	Magnitude	Hypocentral Distance (km)	Peak Ground Acceleration (g)
Olympia	D (623 ft/s)	Puget Sound	April 29, 1965	6.5	84.9	H1(176): 0.137
Zarate	?	Peru Coast	January 5, 1974	6.3	73	H1(000): 0.142
Unio	B?	Michoacan, Mexico	May 22, 1997	6.6	107	H1(000): 0.048
Olympia	D (623 ft/s)	Western Washington	April 13, 1949	7.1	74.7	H1(356): 0.165
Cale	B?	Michoacan, Mexico	January 11, 1997	7.1	36.9	H1(180): 0.357
Puyallup East Sherriff Precinct	C (1,445 ft/s)	Nisqually, Washington	February 28, 2001	6.8	62	H1(090): 0.204 H2(000): 0.213
Synthetic ALL005	B	Cascadia	"	9.0		0.217
Synthetic ALL009	B	Cascadia	"	9.0		0.290

? = NEHRP Site Class Category unknown
ft/s = feet per second
g = gravity
km = kilometer
NEHRP = National Earthquake Hazard Reduction Program

GMPEs for Seismic Hazard Analysis

The GMPEs used by URS and summarized in Table 3.1-1 are consistent with GMPEs that were typically used by the scientific community at the time of the URS study. Three additional GMPEs for subduction zone earthquakes were published at the time of the URS study, but they were not used in the URS study. They are the Gregor et al. (2002), Garcia et al. (2005), and Zhao et al. (2006). The decision by URS not to use these GMPEs most likely reflected their views on the acceptance of these GMPEs within the scientific community at the time of their seismic study. Newer GMPE models are being developed all the time. Before adopting these newer models, there is usually a period of vetting, and therefore URS's decision not to use them in the PSHA is not usual.

It is also noted that URS used the Youngs et al. (1997) attenuation equation in their ground motion hazard assessment. While the Youngs et al. (1997) relationship was generally accepted for intraslab subduction zone earthquakes in 2008 and is still being used by USGS in their hazard model, it is no longer used by USGS and most other earthquake engineers because it is judged to be inconsistent with the data from recent earthquakes in Chile and Japan. Dr. Youngs was contacted to confirm that he agreed with deletion of his subduction zone GMPE.

Implications of Different GMPEs

The use of other GMPEs, as well as the change in applicability of the Youngs et al. relationship, could have some bearing on the seismic hazard analysis. The results of analysis performed by URS show that the ground motions in the return period range of interest (< 2,475 years) are dominated by the intraslab source that underlies the POA, and the controlling earthquake magnitude and rupture distance for this dominant source are M_w 6.5 and 50 km, respectively. Figure 3.1-1 shows the comparison of the 5 percent damped acceleration response spectra calculated for an intraslab event with M_w of 6.5 and a rupture distance of 50 km using the relationships of Youngs et al. (1997), Atkinson and Boore (2003), Zhao et al. (2006), and Garcia et al. (2005).

The comparison shows that the Garcia et al. (2005) model predicts much higher ground motions for periods less than about 0.3 to 0.4 seconds and somewhat lower ground motions for periods greater than 0.4 seconds. If the analysis was performed by replacing the Youngs et al. (1997) model with Garcia et al. (2005) and Zhao et al. (2006) models, the predicted ground motions at the POA would likely be lower for T > 0.3 to 0.4 seconds and higher for T < 0.3 second. The period range of interest for the port facilities is between 0.3 to 2 seconds, per the URS report, suggesting that an update that replaces the Youngs et al. model could result in a decrease in ground motions being used as a basis of design at the POA. However, soil response that is related to lower period motions, such as global stability and seismic wall pressures, could be increased using the newer GMPEs.

Figure 3.1-2 compares deterministic response spectra calculated by URS and CH2M HILL for the maximum earthquakes on the subduction intraslab (M7.5; R =37 km) and interface (M9.2; R = 35 km) sources using Youngs et al. (1997) and Atkinson and Boore (2003) ground motion models. The interface source is also referred to as the mega-thrust event because of its very large magnitude. As can be seen from these figures, the deterministic response spectra calculated by URS and CH2M HILL are generally similar.

The response spectra at the firm-ground level are used by both the PND and the CH2M HILL design teams as a basis for site ground response and soil-structure interaction analyses; therefore, the differences in ground motions predictive equations discussed above could have an effect on the ground motions used in both the simplified Newmark slope displacement analyses and the soil-structure interaction analyses using the computer program $FLAC^{3D}$. As noted above, results of CH2M HILL's review suggest that spectral accelerations with periods less than 0.3 seconds, which would include the PHGA, would increase, while spectral accelerations at longer periods will decrease. For the simplified analyses, larger PHGA would result in higher deformation predictions. In the case of $FLAC^{3D}$, the predominant period of response is greater than 0.3 seconds, suggesting that results of $FLAC^{3D}$ analyses may decrease. The effects of variations in PHGA on the results of the simplified analyses are considered in Section 5. Effects of ground motion variation in the $FLAC^{3D}$ analyses are discussed further in Section 7.

CH2M HILL's conclusion from this review is that the firm-ground motions developed by URS were reasonable at the time that they were developed and provided an appropriate basis of seismic design then and for the suitability study. However, changes that have occurred since 2008—mainly to the GMPEs—will warrant further evaluation for future projects carried out at the POA. This evaluation will require conducting new PSHAs and DSHAs similar to those carried out in 2008 by URS.

Use of Synthetic Records for Mega-Thrust Events

As a final comment regarding the URS evaluation, synthetic earthquakes were recommended by URS to represent the mega-thrust seismic event. As implied by its name, synthetic records are not recorded but were created numerically in the absence of recordings for very large (M>9) earthquakes. Recent earthquakes in Chile and Japan now provide records that are representative of large (M>8.8) seismic events. At the time that the CH2M HILL suitability study was performed, records from both earthquakes were still being processed and had not been released to the public. Once these records are processed and available to the public, they will likely provide a better basis of design than synthetic records.

In general, the synthetic records are believed to be very conservative relative to the duration of strong shaking. The consequence of the very long duration of strong shaking is normally expected to cause higher ground displacements than would be recorded during an actual mega-thrust event, suggesting that the results from using these synthetic earthquake records will be conservative. Results of deaggregation analyses conducted by URS during their seismic hazard study showed that a repeat of the 1964 mega-thrust event—with levels of ground shaking in Anchorage similar to those that occurred in 1964—is very unlikely relative to the same levels of ground shaking from an intraslab event (for example, approximately 0.001 chance per annum for the 1964 subduction zone source versus roughly 0.01 chance per annum for the intraslab event). Because of the much lower likelihood of occurrence of the mega-thrust event, the importance of this event to design is significantly reduced, and the focus on design was for the intraslab source. As a result, the use of synthetic records to represent the mega-thrust event was not as critical as appropriate representation of the intraslab event, which meant that the conservatism of using the synthetic record (or the need to use records from mega-thrust events from Chile or Japan) was not considered to be as important.

This limited importance of the mega-thrust event to the overall hazard study does not mean, however, that a seismic event similar in size to the 1964 Alaska earthquake could not occur somewhere on the overall Aleutian mega-thrust zone in the future. In general, the average return period for events on the mega-thrust source is every 500 years; however, the location of the event could be anywhere along the source, leading to greater distances and lower levels of ground shaking. Given the levels of damage observed in the 1964 earthquake, checks on liquefaction and global stability were still made to evaluate response for the PIEP if a repeat of the 1964 Alaska earthquake were to occur.

3.2 Site-Specific Ground Response Analyses

Results of the URS PSHA and DSHA analyses were used by CH2M HILL as a basis for conducting site-specific ground response analyses using the computer program SHAKE2000. These analyses were similar in principle to analyses conducted by Dr. Youssef Hashash (2008) for estimating input ground motions for soil-structure interaction analyses of the OCSP® system, as well as for conducting global stability analyses of the OCSP® system, performed by the PND design team. Dr. Hashash used his own propriety computer program DEEPSOIL. This program is somewhat different than that used in CH2M HILL's independent site-response analyses, in that DEEPSOIL is able to model the nonlinear response of the soil, whereas the modeling method within SHAKE2000 uses an equivalent linear method. These differences need to be recognized when comparing results of the Hashash analyses versus those discussed below.

3.2.1 Soil Model

The subsurface soils along the PIEP North Expansion include the Bootlegger Cove Formation (BCF) and undifferentiated glacial drift. The stiff clayey BCF extends to an elevation of about -200 feet mean lower low water (MLLW) and overlies a dense gravelly and sandy alluvial glacial deposit. The soil borings drilled along the North Expansion also encountered a sand layer from elevation -150 feet to -175 feet MLLW and several feet of loose silty soil near the mudline. Some of this silty soil was dredged and gravelly sandy fill placed behind the OCSP® wall to form the wharf terminal and backland. The depth to bedrock at the project location is unknown; however, it is believed to be greater than 500 feet below the ground surface.

Two idealized soil profiles with their dynamic properties were developed for the analysis: one profile for the seaward side of the OCSP® facewall, and another profile for the landward side of the OCSP® facewall. These two idealized soil profiles and their small-strain, shear-wave velocity (V_s) profiles assigned to the various soil layers are shown in Figure 3.2-1. The same idealized profiles were used throughout the North Expansion area. Although variations in layer thickness occur between the north and south ends of the North Expansion area, these changes were considered small relative to the 450-foot soil model that was used in the ground response models. Sensitivity analyses were conducted to evaluate reasonable variations in V_s. The range of the sensitivity studies was sufficient to cover the possible range in soil layer thicknesses.

The small-strain shear-wave velocities of the various soil deposits/layers were determined by Terracon (PND, 2008, Appendix P) as part of their exploration work for the PIEP. The V_s tests involved use of P-S Suspension logging methods for offshore locations; a cross-hole exploration was conducted at one on-shore location. Terracon supplemented the results of these seismic geophysical tests with results of measurements on similar soils at other nearby locations, as discussed by Terracon.

As shown on Figure 3.2-1, the V_s profile for the BCF on the landward side was defined using the upper end of the data, considering the long-term increase in effective overburden pressure and decrease in overconsolidation ratio (OCR) as a result of new granular fill. The V_s at elevation -43 feet was about 1087 feet per second (ft/s) and increased to about 1405 ft/s at elevation -150 feet – the location of the FLAC^{3D} model base. Similarly, the V_s profile used for the BCF on the seaward side of the wall was taken on the lower side of the data, representing the condition after the proposed sediment removal in front of the wall (proposed dredging to elevation -51 feet MLLW). At the dredge-depth elevation -51 feet, the V_s was 200 ft/s and increased with depth to about 821 ft/s at elevation -150 feet.

The V_s profile in the compacted granular fill was estimated using the following relationships (Seed et al., 1984):

$$G_{max} = 1000 * k_{2max} * \sqrt{\sigma'_m}$$

$$Vs = \sqrt{\frac{G_{max}}{\rho}}$$

where

G_{max}	=	maximum shear modulus (in pounds per square foot [psf])
K_{2max}	=	constant parameter (taken as 100 for the granular fill)
σ'_m	=	mean confining effective pressure (in psf)
ρ	=	mass density
V_s	=	small strain shear-wave velocity

An effort was made in the February of 2012 to measure V_s of the compacted granular fill during a field exploration program, as summarized in Appendix D1. The field exploration program was conducted to determine the density of the granular fill after ground improvement and to collect soil samples for large-size (e.g., 12-inch by 12-inch) direct shear testing in the laboratory. Seismic cone penetrometer tests were planned as part of this exploration program. These tests involved pushing a cone penetrometer rod equipped with a vibration-sensing transducer into the compacted granular fill to various test depths below the ground surface. A transient wave was generated at the ground surface for each test, and the time for this impulsive wave to arrive at the cone penetrometer sensor was measured. The ratio of the distance travelled to the time of travel defined the V_s. However, this attempt to measure V_s was unsuccessful. The compacted granular fill proved too difficult to push the cone penetrometer sensor into because of a combination of density and over-sized gravelly soil, even when using the weight of the drill rig to push the cone penetrometer sensor. In addition, the upper 8 feet of granular fill was frozen, which diffused the transient wave to such an extent that the V_s values could not be identified. Since the V_s measurements were a secondary priority of the program, no further effort was made to obtain V_s values, and the Seed et al. equation identified above was used as a basis of estimating V_s for the ground response studies.

Based on the available V_s data, the National Earthquake Hazard Reduction Program (NEHRP) B/C boundary (a firm-ground site with an average shear-wave velocity of 760 meters per second [m/s] or about 2,500 ft/s) was estimated to be at elevation -450 feet MLLW within the glacial drift, which is consistent with that assumed in the previous site-response analysis (Hashash, 2008). A formula similar to the above was used to develop the shear-wave velocity profile in the glacial drift.

For the CH2M HILL study, the shear-modulus reduction (G/G_{max}) and damping versus shear strain curves developed by Darendelli (2001) were used to model the non-linear soil behavior under cyclic loadings. These relationships are functions of plasticity index (PI) and confining pressure, as illustrated in Figures 3.2-2 and 3.2-3 for PI values of zero percent (used for the sandy/gravelly soils) and 15 percent (used for the clayey BCF), respectively.

3.2.2 Input Motions

As discussed in Section 3.1, URS developed the design earthquake ground motions for the project (URS, 2008). Acceleration response spectra and earthquake time histories corresponding to the OLE, CLE, and MCE were developed for a free-field (outcropping) firm-ground site (that is, a site with an average V_{s30m} of 760 m/s or NEHRP B/C boundary).

Fifteen earthquake time histories were developed by URS and used in the CH2M HILL site response analysis: three for the OLE, six for the CLE, and six for the MCE. These time histories were developed by spectrally modifying actual recorded motions during past earthquakes for the intraslab events and synthetic motions for the mega-thrust events. The naturally recorded time histories selected for the intraslab events were matched to the OLE, CLE, and MCE response spectra developed from the PSHA, while the mega-thrust synthetic time histories were matched to the deterministic response spectra computed for the Cascadia mega-thrust events that were scaled to the probabilistic CLE and MCE spectra in the period range between 0.3 and 2.0 seconds. Note that no mega-thrust time histories were developed for the OLE.

The selected earthquake time histories used for matching are listed in Table 3.2-1. Plots of these time histories are provided in Appendix C.

TABLE 3.2-1
SELECTED EARTHQUAKE TIME HISTORIES FOR CH2M HILL SITE RESPONSE ANALYSES

Earthquake	Recording Station/ Orientation	Moment Magnitude, M_w	Recording Distance, km	Fault Mechanism[a]	Focal Depth (km)
Operating Level Earthquake (OLE)					
1965 Puget Sound	USGS Stn. 2101/ 176 deg	6.5	85	Intraslab	59
1974 Peru Coast	Zarate/ 0 deg	6.3	73	Intraslab	82
1997 Michoacan	Unio/ 0 deg	6.6	107	Intraslab	35
Contingency Level Earthquake (CLE)					
2001 Nisqually	PCEP/ 0 deg	6.8	62	Intraslab	52
2001 Nisqually	PCEP/ 90 deg	6.8	62	Intraslab	52
1997 Michoacan	Cale/ 180 deg	7.1	37	Intraslab	35
1949 Western WA	Olympia/ 356 deg	7.1	75	Intraslab	50
Cascadia Megathrust Synthetic (ALL005)	-	9	-	Synthetic	-
Cascadia Megathrust Synthetic (ALL009)	-	9	-	Synthetic	-
Maximum Considered Earthquake (MCE)					
2001 Nisqually	PCEP/ 0 deg	6.8	62	Intraslab	52
2001 Nisqually	PCEP/ 90 deg	6.8	62	Intraslab	52
1997 Michoacan	Cale/ 180 deg	7.1	37	Intraslab	35
1949 Western WA	Olympia/ 356 deg	7.1	75	Intraslab	50
Cascadia Megathrust Synthetic (ALL005)	-	9	-	Synthetic	-
Cascadia Megathrust Synthetic (ALL009)	-	9	-	Synthetic	-

Note: The Cascadia megathrust time histories were matched to the deterministic target spectra in the long-period range.
[a] Intraslab refers to subduction zone earthquake at relatively deep depths below the ground surface. Synthetic refers to an earthquake record that was created numerically to represent an interface event on the subduction zone.
PCEP = Pierce County East Sheriff Precinct

3.2.3 Site-Specific Seismic Ground Response Evaluation

CH2M HILL's dynamic site response analysis was conducted to characterize the effects of BCF and backland fill on earthquake ground motions and to obtain input ground motions for the soil-structural interaction and wall stability analyses. The presence of BCF and placement of fill behind the wall will likely alter the characteristics of seismic ground motions, especially near the ground surface and mudline, where nonlinearity of soils was expected to have more effect.

The approach used for the site response analysis can be summarized in the following steps:

- Developed idealized soil profiles for analysis, including the soil dynamic properties
- Performed one-dimensional ground motion response analyses using the spectrally matched firm ground earthquake acceleration time histories as input motions at the base of the model
- Calculated earthquake time histories and acceleration response spectra at selected elevations within the BCF and wall and at ground surface

3.2.3.1 Site Response Analysis

The analyses were performed using the computer program SHAKE2000, version 8.9.0 (GeoMotions, 2011a). The program is based on one-dimensional wave propagation of vertically propagating, horizontal shear-waves (V_s or S-waves). The soil responses were modeled using the equivalent-linear method, originally proposed by Seed and Idriss (1970), based on the G/G_{max} and damping versus shear stain curves as noted above. The analyses were performed in iterations until the shear modulus and damping values used in the analyses were compatible with the computed shear strain. A shear strain equal to two-thirds of the peak shear strain was used to determine strain compatible properties. Although the SHAKE methodology clearly represents an approximation of the wave propagation mechanism, it is commonly used, and has been shown to provide a reasonable analytical representation of site response at soil sites where soils are relatively stiff, as they are at the POA, and as long as shear strains are less than 1 to 2 percent.

3.2.3.2 Calculated Acceleration Time Histories and Response Spectra

The acceleration time histories and 5 percent damped acceleration response spectra at the ground surface or mudline and at selected depths were calculated for each idealized soil profile and design earthquake. Figures 3.2-4 through 3.2-7 depict the calculated time histories and response spectra for one of the OLE motions and for the landward and seaward soil profiles (see Appendix C for the plots of all the OLE motions). The calculated maximum accelerations, shear strains, and shear stresses are plotted with depths in Figures 3.2-8 and 3.2-9.

Note that the input outcropping motions were deconvolved to within motions at elevation -450 feet prior to propagating upward through the soil profile; the deconvolved or within motions are shown on the figures, and they are lower than the outcropping motions because of the effects of overburden pressure. As can be seen from these figures, the ground motions are amplified through the BCF and granular fill, especially those at periods greater than about 0.1 to 0.3 seconds over the unconsolidated BCF in front of the wall (seaward soil profile). The seismic-induced shear strains are generally small, except near the mudline where the existing BCF is soft.

Similar plots for the CLE and MCE are shown in Figures 3.2-10 through 3.2-21. Appendix C presents the complete plots of all calculated time histories and response spectra for the CLE and MCE. In general, these results indicate dynamic amplification characteristics similar to those observed for the OLE; however, the seaward soil profile tends to filter out more high-frequency motions over the softer unconsolidated BCF. This is a result of the higher ground motions for the CLE and MCE that induce larger strains and soil non-linearity, which dampen high-frequency motions (note that significant de-amplification of high-frequency motions is estimated for the MCE motions).

Table 3.2-2 summarizes the estimated fundamental periods of the soil columns obtained in the one-dimensional SHAKE2000 model for the three design ground motions, as well as the columns prior to earthquake shaking (initial condition). The period was estimated using the average value of the strain-compatible shear-wave velocities (or-shear moduli) calculated within the various soil layers in the soil columns. As expected, longer periods (or-softer soil columns) were estimated for higher ground motions due to larger seismic-induced strains. The difference in fundamental period between the landward and seaward sides for each ground motion level is the result of differences in the average shear-wave velocity value and column length. Although the average shear-wave velocity for the landward soil column (average V_s = 1,806 ft/s) is higher than that for the seaward column (average V_s = 1,616) due to the stiffer granular fill, its fundamental period is longer because of the greater column length (that is, soil column lengths for landward and seaward sides are 485 feet and 400 feet, respectively).

TABLE 3.2-2
Soil Column Fundamental Periods for the Three Design Ground Motions

	Fundamental Periods (in seconds)			
Soil Column	**Initial**	**OLE**	**CLE**	**MCE**
Seaward side of wall	0.99	1.06-1.07	1.11-1.14	1.19-1.23
Landward side of wall	1.07	1.14-1.15	1.19-1.23	1.27—1.32

3.2.4 Sensitivity Analysis

Sensitivity studies were conducted for four variables in the soil model. These variables were:

- **The depth of ground motion input.** The depth of firm-ground motion was assigned by URS at elevation -450 feet; however, this was based on their judgment. An evaluation of this depth was performed to determine the sensitivity of ground motions to changes in this elevation.
- **The elevation of groundwater behind the OCSP® system wall and the finished grade of the wall.** The groundwater variation behind the wall was uncertain when the initial ground response analyses were conducted. Therefore, the water elevation was assumed to be the same as mean sea level. Later evaluation confirmed that a higher water elevation exists behind the wall. The finished grade was also increased to represent the final constructed elevation.
- **The V_s in the backfill.** No direct measurements of V_s were obtained within the backfill. Therefore, the V_s profile was estimated using a typical k_{2max} value for compacted fill, as discussed previously. Sensitivity of the calculated ground motions to variation of V_s in the backfill was performed.
- **The V_s at the original mudline.** The V_s profile used for the current analysis shows an abrupt velocity contrast at the interface between the backfill and BCF. This velocity contrast may affect how the waves and stresses are propagated upward into the fill. Sensitivity of the calculated ground motions to this contrast was performed.

Results of these sensitivity studies are summarized below.

3.2.4.1 Sensitivity of Results to NEHRP B/C Boundary Elevation

Sensitivity of the calculated motions to the assumed ground motion input level, referred to as the NEHRP B/C boundary, was evaluated by inputting the firm ground earthquake time histories at elevations -200 and -670 feet, in addition to elevation -450 feet as described above. For this sensitivity evaluation, only one of the intraslab (2001 Nisqually earthquake at Pierce County East Sheriff Precinct [PCEP] station, 90 degree component) and one of the mega-thrust synthetic (Cascadia ALL005) CLE records were used. The calculated 5 percent damped response spectra at the ground surface or mudline and at elevation -150 feet are compared in Figures 3.2-22 and 3.2-23 and Figures 3.2-24 and 3.2-25 for the landward and seaward soil profiles, respectively.

Results of this sensitivity study show that inputting the ground motion at elevation -200 feet increases the PHGA by about 17 percent as compared to that for the deeper input motion (input motion at elevation -670 feet).

3.2.4.2 Sensitivity of Results to Groundwater and Finished Grade

Sensitivity of the calculated motions to groundwater depth and finished grade elevation was evaluated by raising the groundwater elevation to +20 feet and finished terminal grade to +38 feet on the landward soil profile. Figures 3.2-26 and 3.2-27 compare the calculated response spectra at the ground surface and top of sand layer (elevation -150 feet) for the 2001 Nisqually earthquake (at PCEP station, 90 degree component) and the mega-thrust synthetic record (Cascadia, ALL005).

Results of the evaluation of groundwater and surface elevation changes show that a groundwater elevation of +20 feet and a surface elevation to +38 feet had a relatively small effect on the response spectra at the ground surface and did not affect ground motions at the level of input for the limit equilibrium stability and soil-structure interaction evaluations.

3.2.4.3 Sensitivity of Results to Variation of Backfill Velocity

Sensitivity of the calculated motions to V_s value in the backfill was evaluated by varying the velocity profile by ±20 percent. Figures 3.2-28 and 3.2-29 compare the calculated response spectra at the ground surface and top of sand layer (elevation -150 feet) for the 2001 Nisqually earthquake (at PCEP station, 90 degree component) and the mega-thrust synthetic record (Cascadia, ALL005).

Results of this sensitivity study show that the ground motions (for periods less than about 1 second) at ground surface increase as the backfill becomes softer (that is, smaller V_s value). Reducing the backfill velocity by 20 percent increases the PHGA by about 15 percent. The ground motions at the top of sand layer (Elev. -150 feet) are not sensitive to the relatively small variation in backfill velocity.

3.2.4.4 Sensitivity of Results to Velocity Contrast at Interface between Backfill and BCF

Sensitivity of the calculated motions to velocity contrast was evaluated by smoothing the transition of velocity values within ± 10 feet above and below the interface between the backfill and BCF. Figures 3.2-30 and 3.2-31 compare the calculated response spectra at the ground surface and top of sand layer (elevation -150 feet) for the 2001 Nisqually earthquake (at PCEP station, 90 degree component) and the mega-thrust synthetic record (Cascadia, ALL005).

Results of this sensitivity study show that the calculated ground motions are not sensitive to the velocity contrast at the interface between the backfill and BCF.

3.2.5 Amplification Factors

The ground motion amplification characteristics of the two soil profiles, landside and waterside, were calculated from the results of the above site response analyses. The amplification factors were calculated by dividing the calculated 5 percent damped response spectral values at the ground surface/mudline with the corresponding outcropping firm ground spectral values. The average amplification factors were then estimated from the calculated ratios for the input time histories. Figure 3.2-32 and Figure 3.2-33 present the calculated and average amplification factors at the ground surface/mudline, as a function of vibratory periods, for the two profiles and the three design earthquakes.

The figures show that the outcropping rock motions with periods less than about 0.3 to 0.5 second are de-amplified, and those with long periods are amplified at the project site. The spectral amplifications are more substantial over the soil profile seaside of the wall (seaward soil profile) and for higher ground motions, as expected.

3.2.6 Comparison of Results with Previous Study

The results of the current analysis are compared to those calculated in the previous site-response analysis (Hashash, 2008) in Figures 3.2-34 and 3.2-35. The comparison was made for the 2001 Nisqually earthquake recorded at PCEP station (90 degree component) and the Cascadia megathrust synthetic earthquake (ALL05) at ground surface or mudline for periods of 0.01 (peak ground acceleration [PGA]), 0.2, and 1.0 seconds. As can be seen from these figures, differences exist between the calculated response spectra of the current study and those from the previous analysis (Hashash, 2008). The differences are particularly significant for the seaward soil profile and for higher ground motions (MCE ground motions). Results are summarized as follows:

- **Landward Results**. In general, ground motions from the equivalent linear analyses conducted by CH2M HILL result in higher PHGA and spectral accelerations than those obtained by Hashash (2008). The difference in PHGA ranges from 20 to 100 percent, depending on earthquake records and the method Hashash used in his analyses. At longer periods the difference was much less, often being within 10 to 20 percent for the CLE and within 50 percent for the MCE. These comparisons suggest that the ground motions developed by the CH2M HILL analyses will be somewhat conservative at the ground surface, resulting in higher estimates of ground displacements. Only small variations in ground motions were observed at the elevation used as input for conventional stability studies (Section 5) and for soil-structure interaction analyses (Section 7), suggesting that the conventional stability and FLAC^{3D} analyses would not be affected to any significant extent. This is particularly true for the FLAC^{3D} analyses where little difference exists between the two ground motions. See Sections 5 and 7 for further discussion of this issue.

- **Seaward Results.** In general, the PHGA and the spectral accelerations on the seaward side of the OCSP® system are higher from the Hashash DEEPSOIL analyses than from the analyses carried out for this suitability study. The reason for this observation is not obvious. Normally, equivalent linear analyses would provide higher ground motions, because soil nonlinearities are not modeled as well. This was not the case for this comparison. However, only the landward soil profile is critical to the soil-structure interaction and ground response analyses; therefore, the difference in ground motions for the seaward side of the OCSP® system was not considered further.

The differences noted in these comparisons result from the different numerical modeling methods. Specifically, as noted earlier, Hashash used a nonlinear computer program and this study used an equivalent linear method. Because the CH2M HILL results are higher for the landward side of the OCSP® system than those determined by Hashash, the basis of design used in the CH2M HILL evaluation would be somewhat conservative relative to the previous analyses.

3.2.7 Conclusions and Recommendations

A review of the firm-ground earthquake motions developed by URS Corporation (2008) was conducted. In general, the approaches, assumptions, and results in the URS study are considered reasonable and consistent with the current understandings of the project site seismic environment. Two synthetic earthquake time histories were developed by URS to represent the mega-thrust events in the absence of recordings for very large (M>9) earthquakes. Recent earthquakes in Chile and Japan now provide records that are representative of large (M>8.8) seismic events. Once these records are processed and available to the public, they will likely provide a better basis of design than synthetic records. We recommend that these natural records be considered for any future work at the POA.

An independent site response analysis was also conducted in the current study. The soil profiles and dynamic properties used for the analysis were developed independently from the previous studies, including the study performed by Dr. Youssef Hashash (2008). The results of the analysis suggest that the ground motions developed by CH2M HILL are somewhat conservative, and therefore, may result in higher estimates of ground displacements. On the landward side of the wall, only small variations in ground motions were observed between these studies at the elevation used as input for soil-structure interaction analyses. This suggests that the FLAC^{3D} analyses for the walls would not be affected significantly, regardless of which results are used. Somewhat higher differences were observed at the level used for the conventional stability analyses. These differences could result in lower factors of safety and higher displacements in the limit-equilibrium stability analyses. However, the amount of variation is within the limits of uncertainty in seismic response analyses, and therefore we believe that the motions are appropriate for making design evaluations.

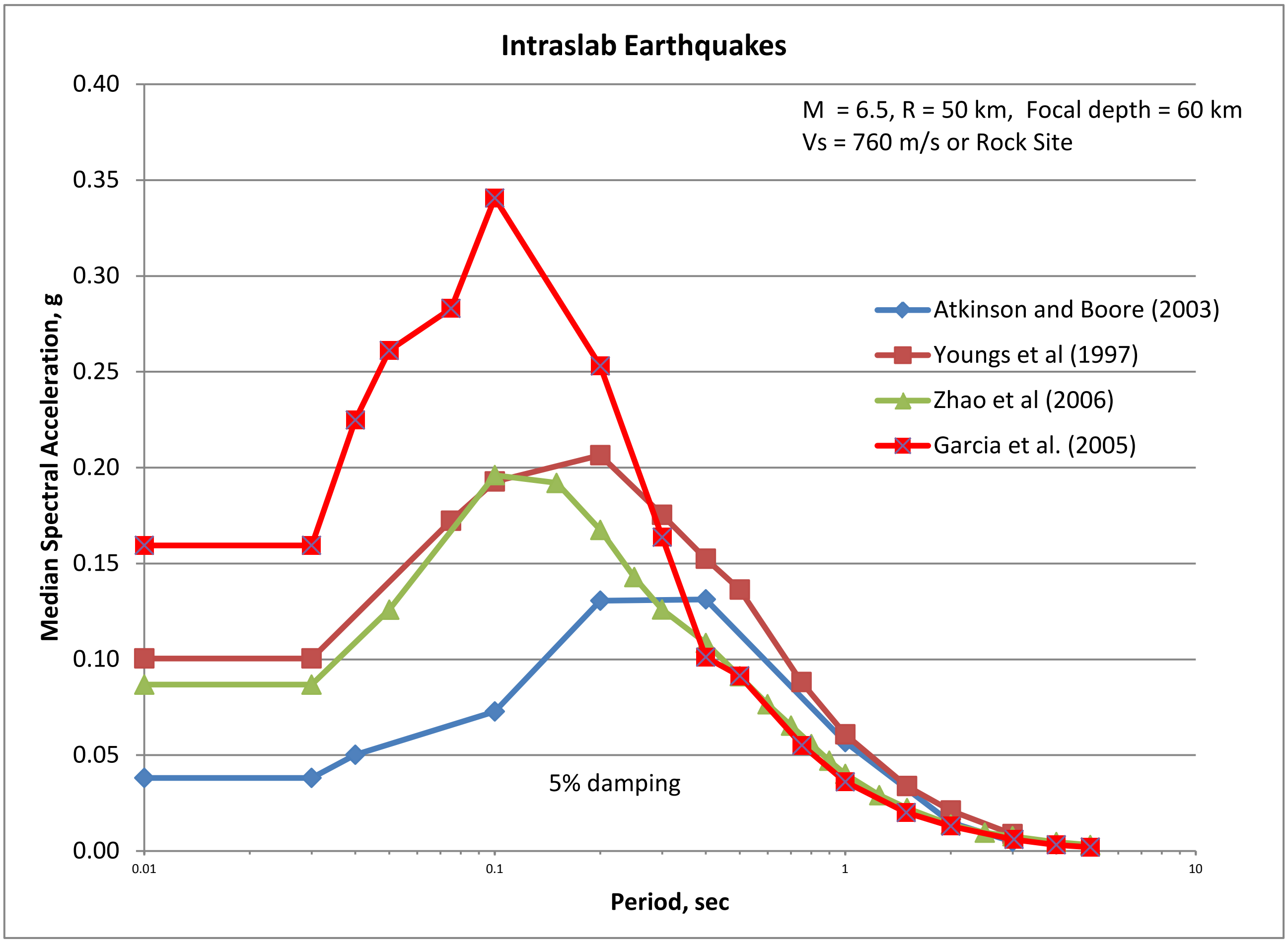
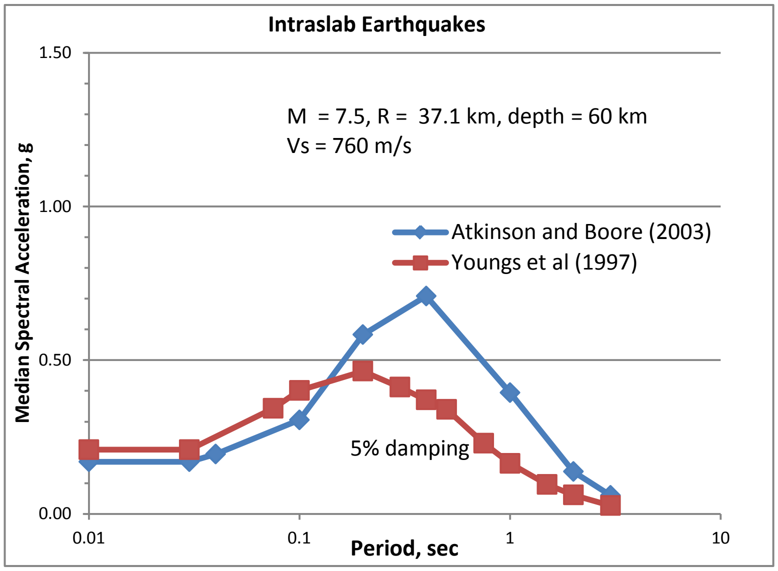
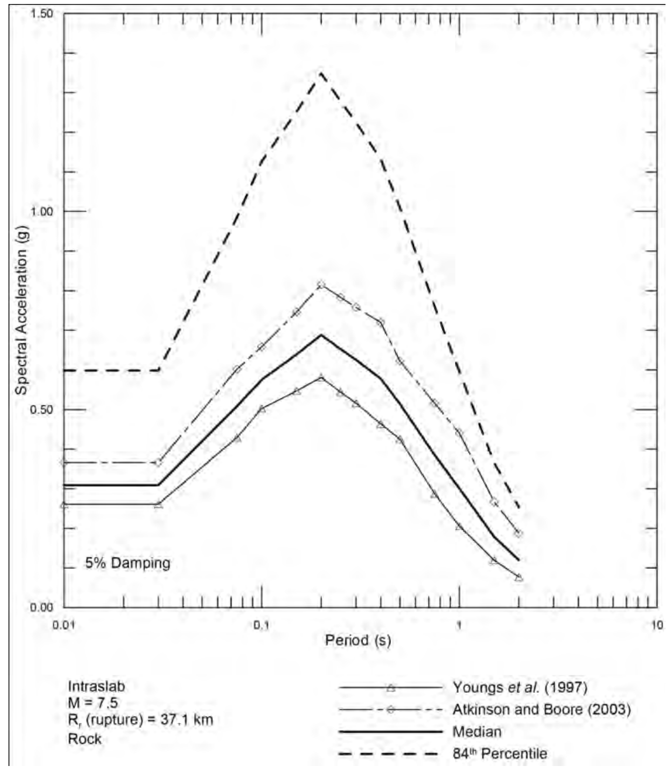


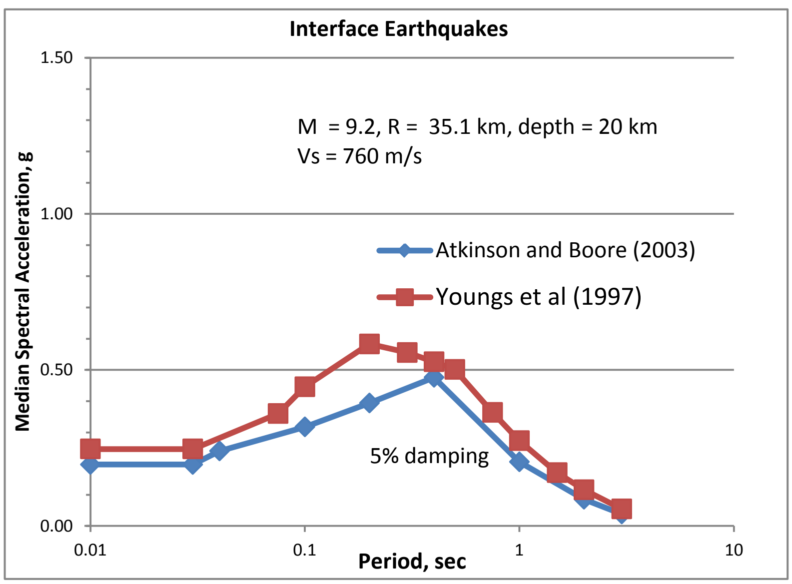
FIGURE 3.1-1. Comparison of Response Spectra for Intraslab Source Mechanism Using Alternate Ground Motion Predictive Equations





CH2M HILL Calculated Spectra

URS Calculated Spectra (Source: URS, 2008)



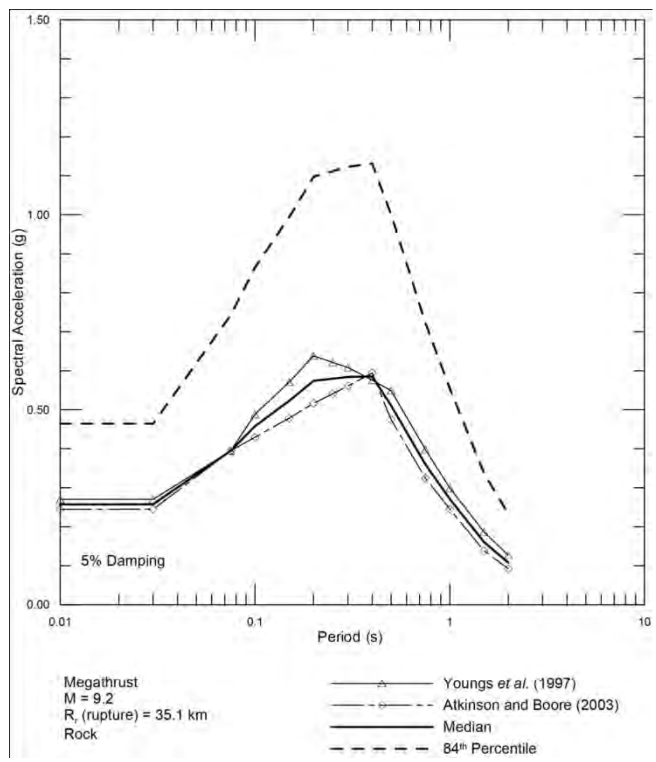


FIGURE 3.1-2. Comparison for Response Spectra Determined by URS and CH2M HILL

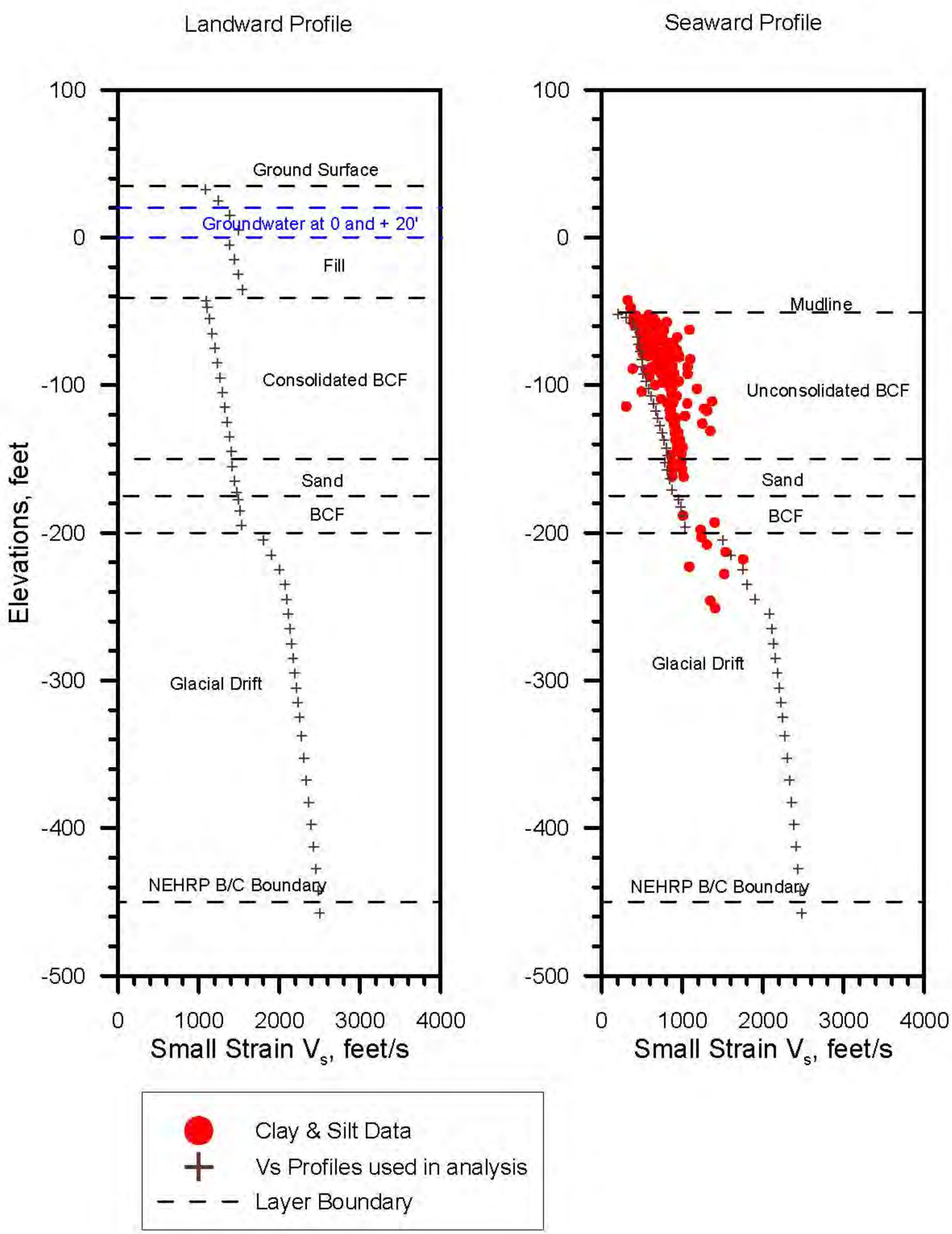


FIGURE 3.2-1. Idealized Soil Profiles for Site Response Analyses

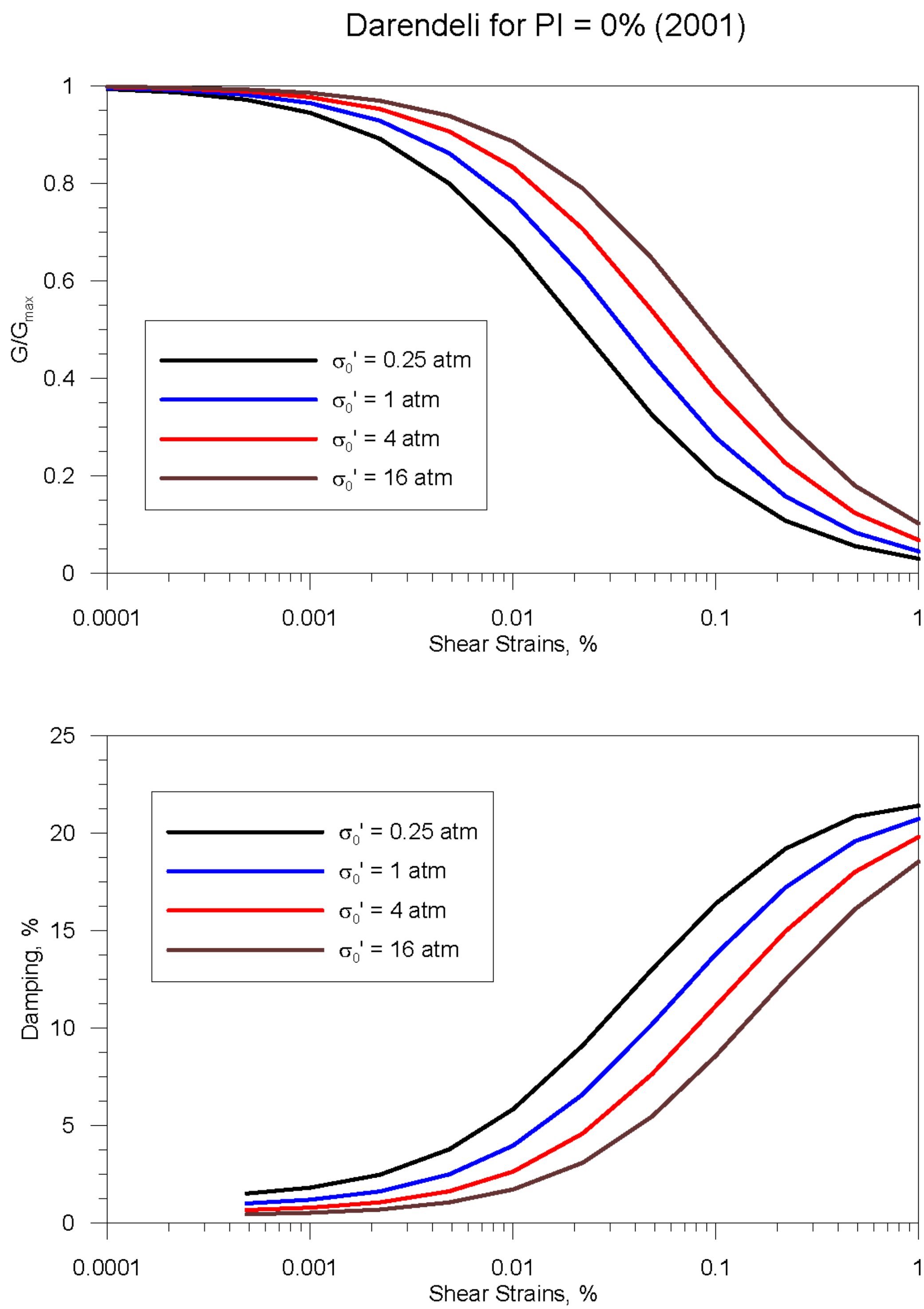


FIGURE 3.2-2. G/G_{max} and Damping vs. Shear Strain Curves for Sandy and Gravelly Soils

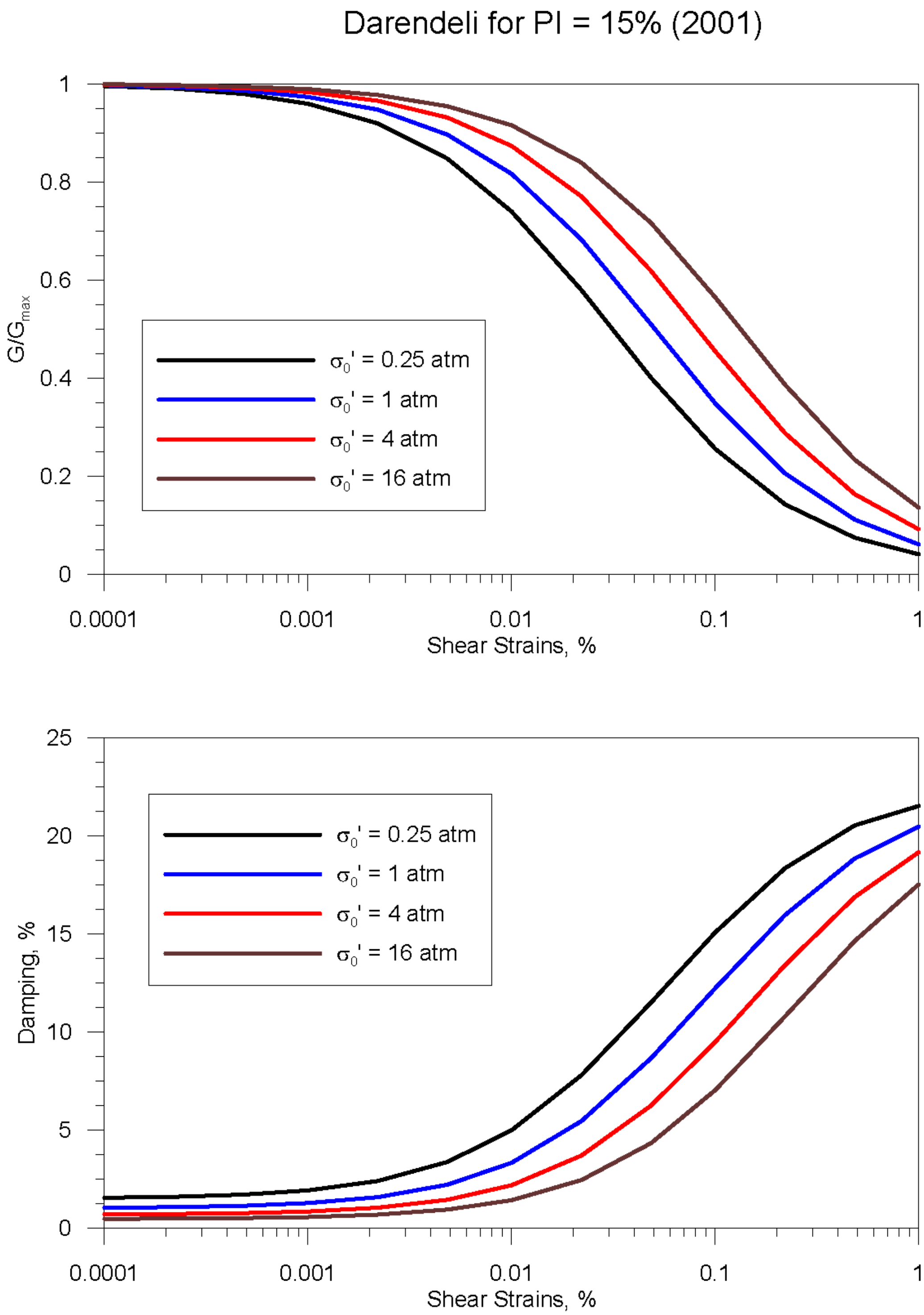


FIGURE 3.2-3. G/G_{max} and Damping vs. Shear Strain Curves for Clayey Soils

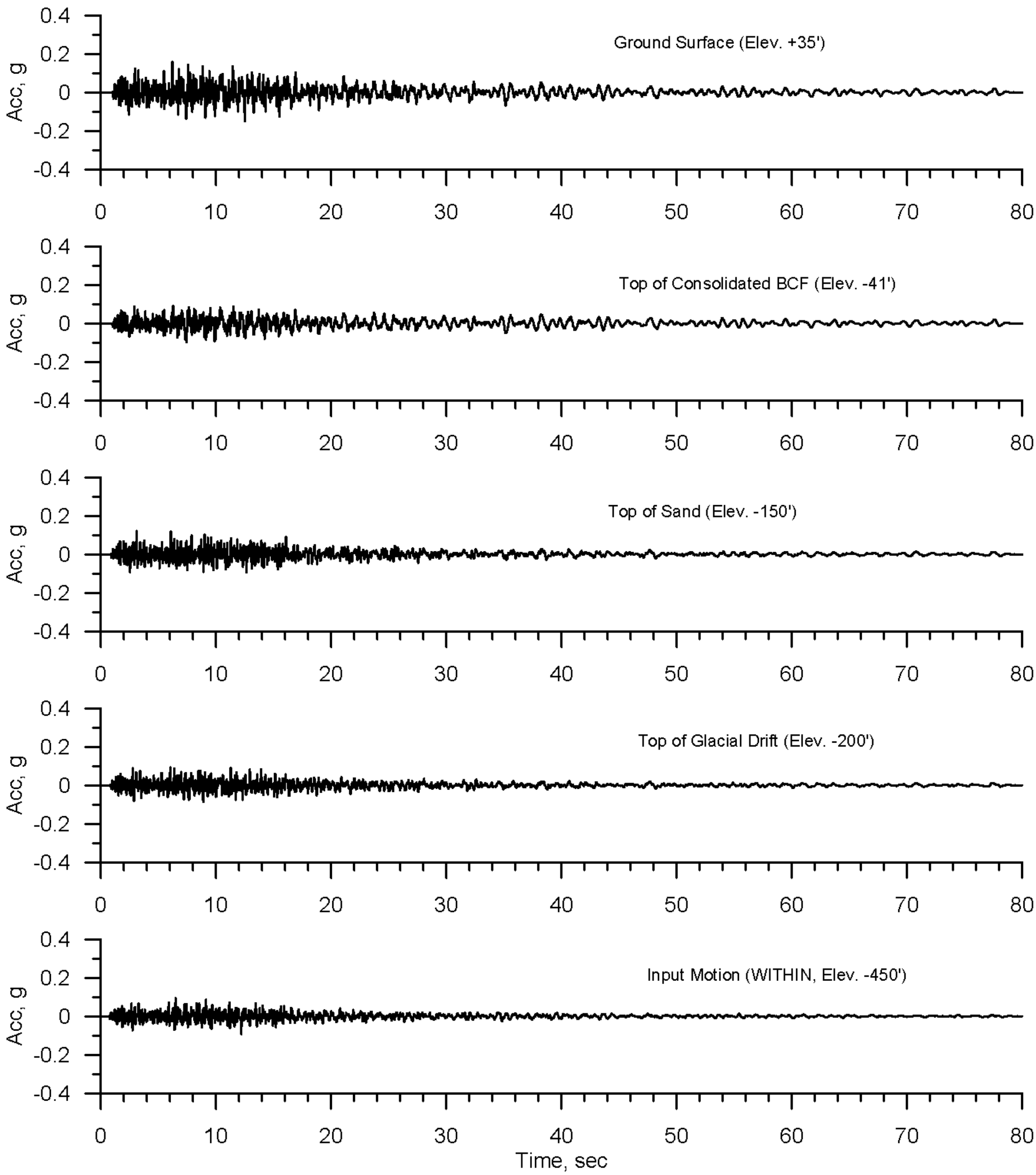


FIGURE 3.2-4. Calculated Time Histories – OLE, 1965 Puget Sound Earthquake at USGS Station 2101, 176 deg component, Landward Soil Profile

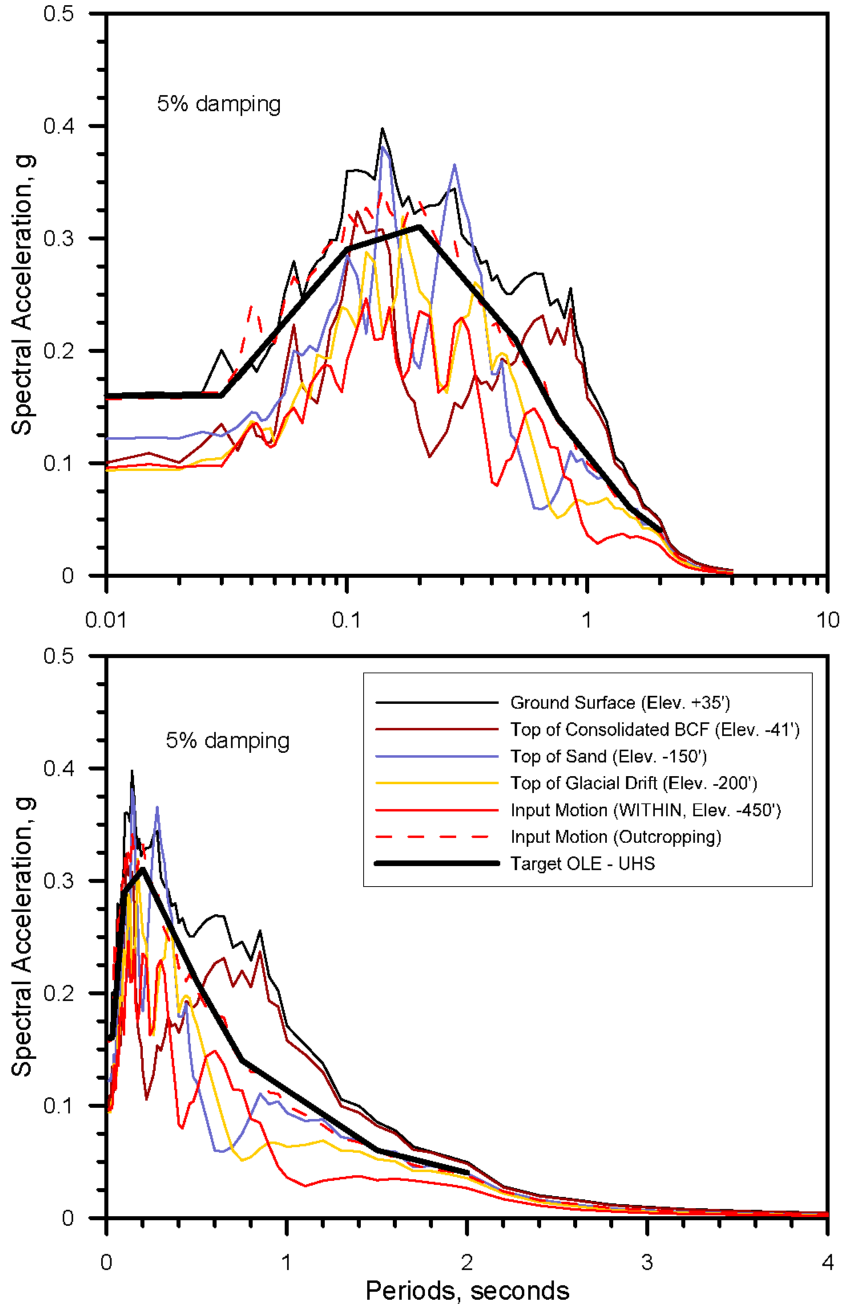


FIGURE 3.2-5. Calculated Response Spectra – OLE, 1965 Puget Sound Earthquake at USGS Station 2101, 176 deg component, Landward Soil Profile

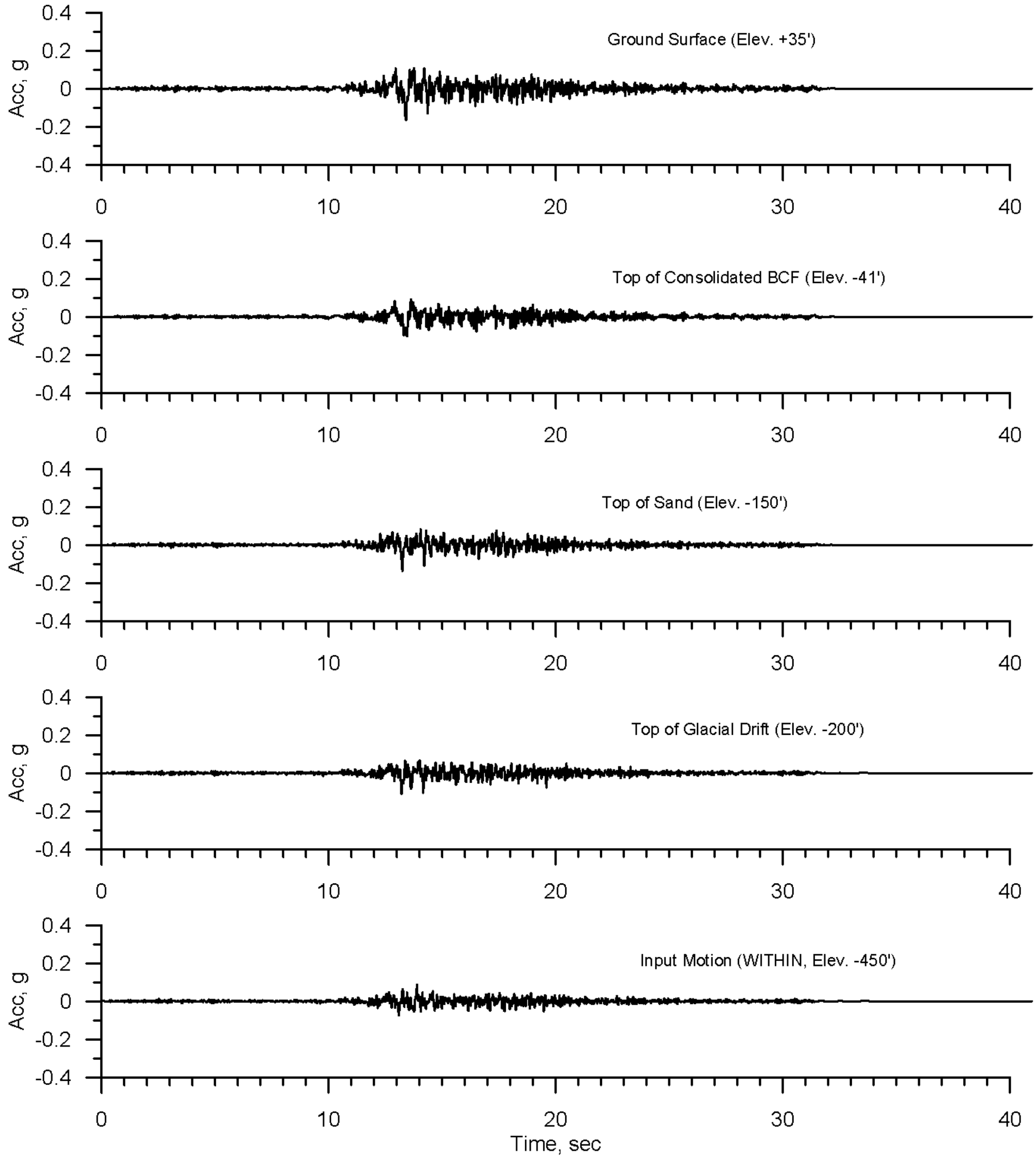


FIGURE 3.2-6. Calculated Time Histories – OLE, 1974 Peru Coast Earthquake at ZARATE Station, 0 deg component, Landward Soil Profile

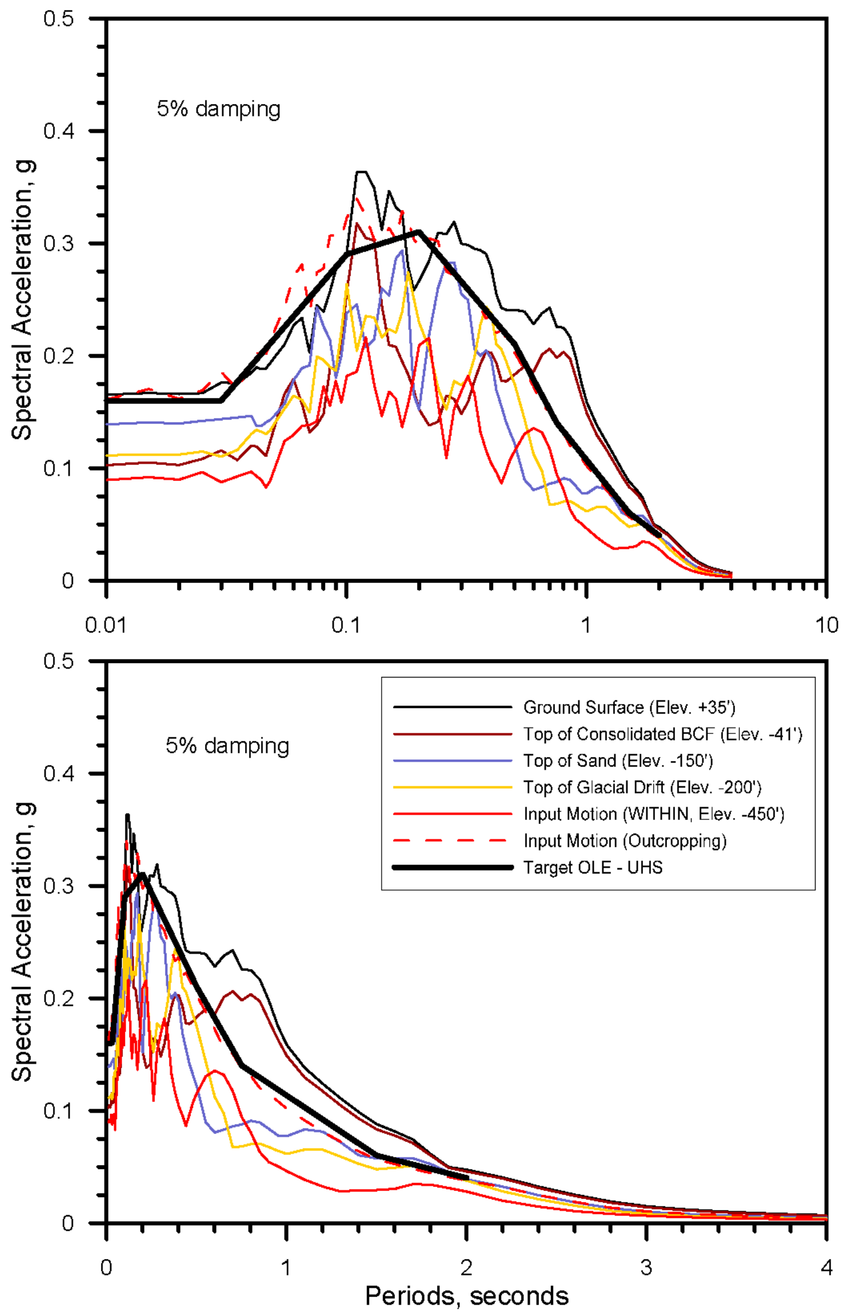


FIGURE 3.2-7. Calculated Response Spectra – OLE, 1974 Peru Coast Earthquake at ZARATE Station, 0 deg component, Landward Soil Profile

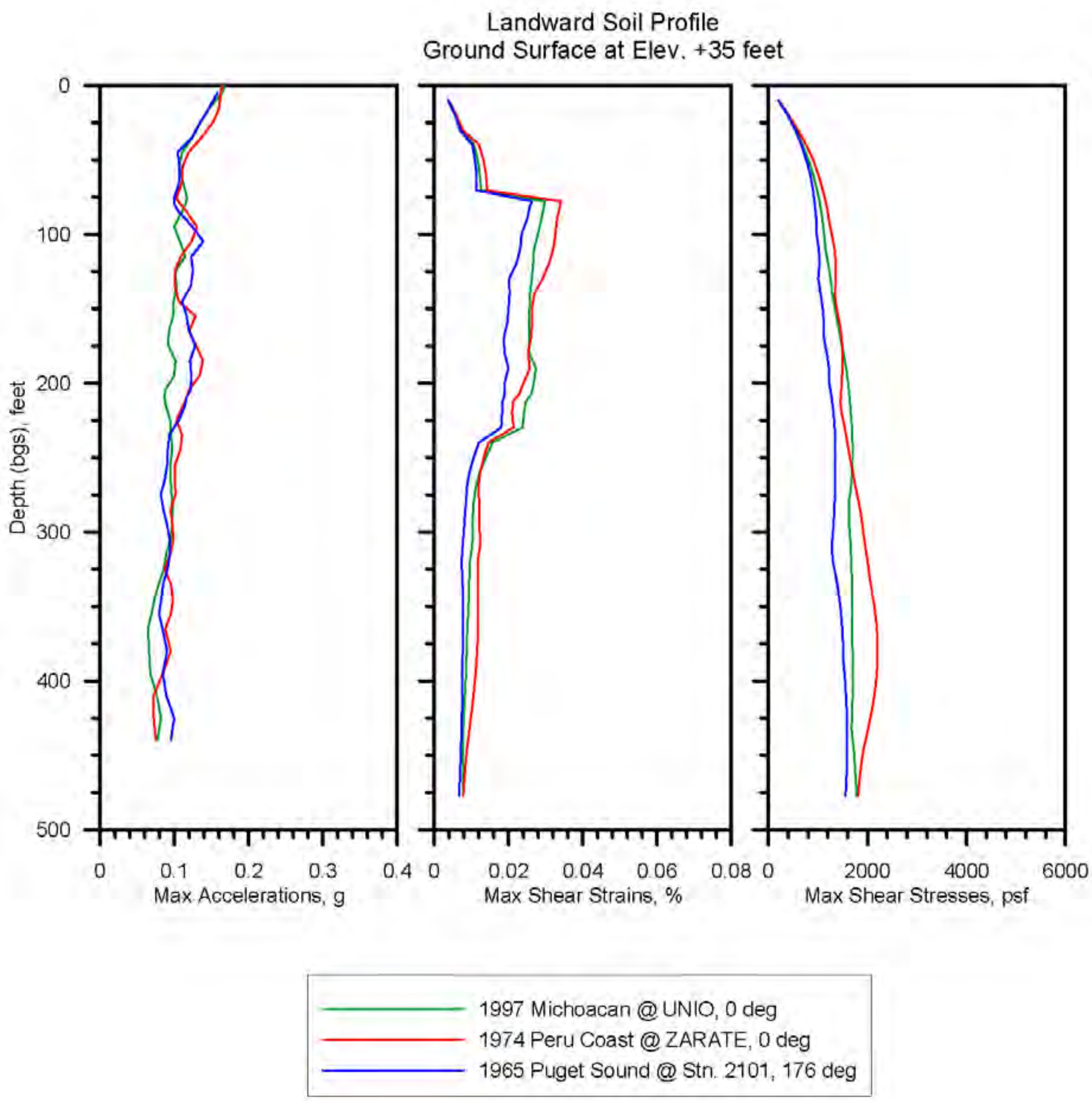


FIGURE 3.2-8. Calculated Maximum Accelerations, Shear Strains and Shear Stresses vs. Depth, Landward Soil Profile, OLE

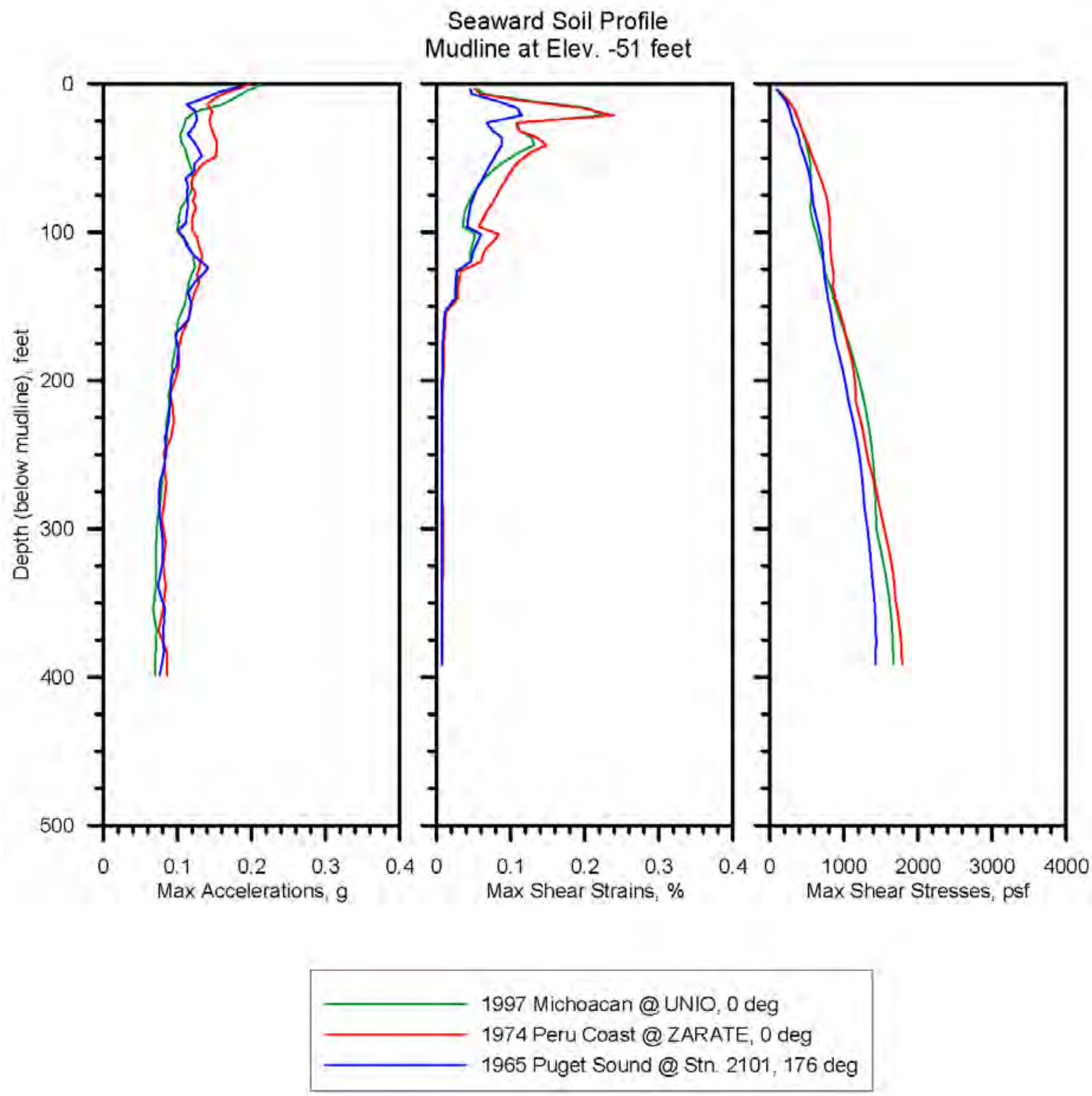


FIGURE 3.2-9. Calculated Maximum Accelerations, Shear Strains and Shear Stresses vs. Depth, Seaward Soil Profile, OLE

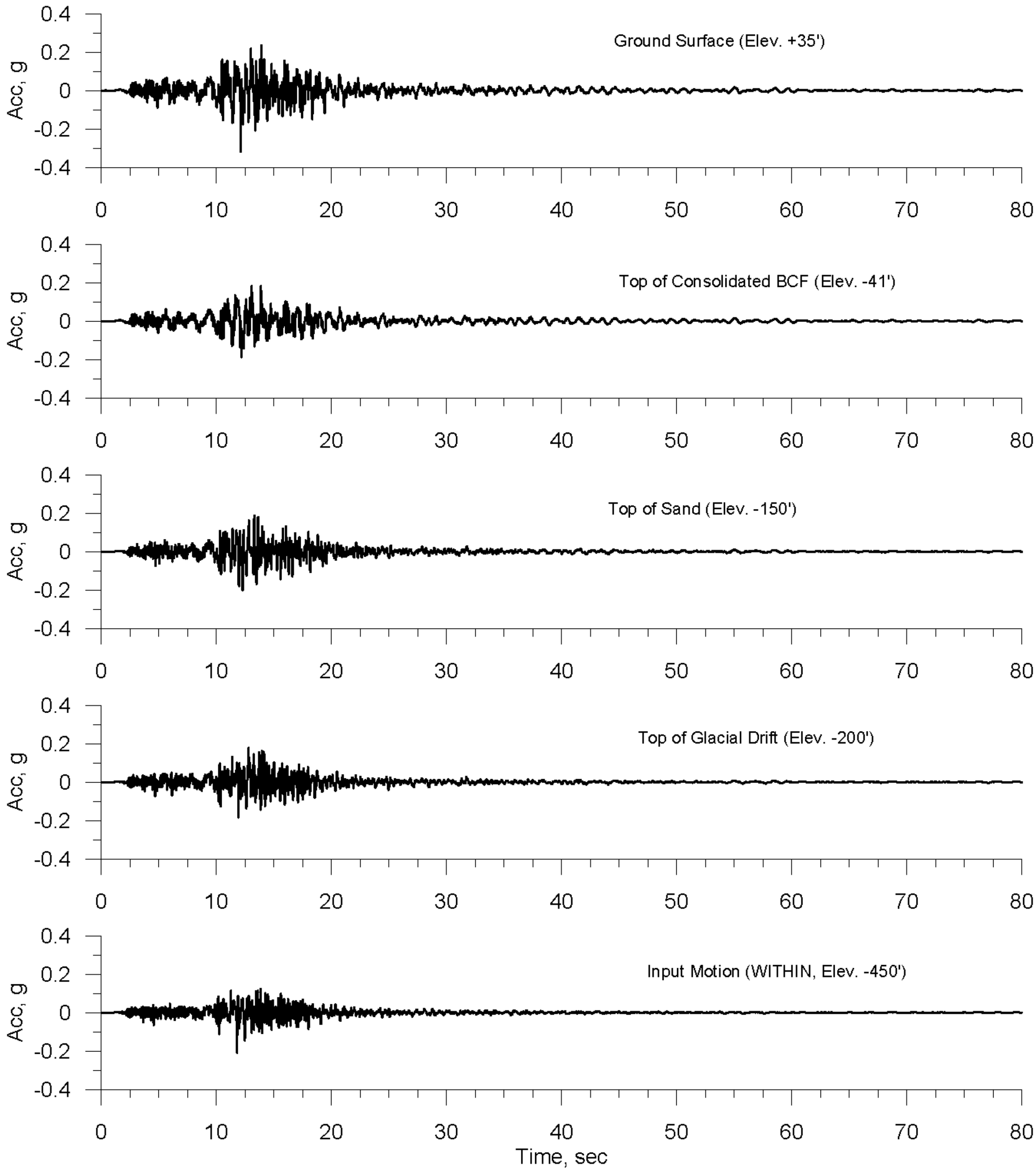


FIGURE 3.2-10. Calculated Time Histories – CLE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Landward Soil Profile

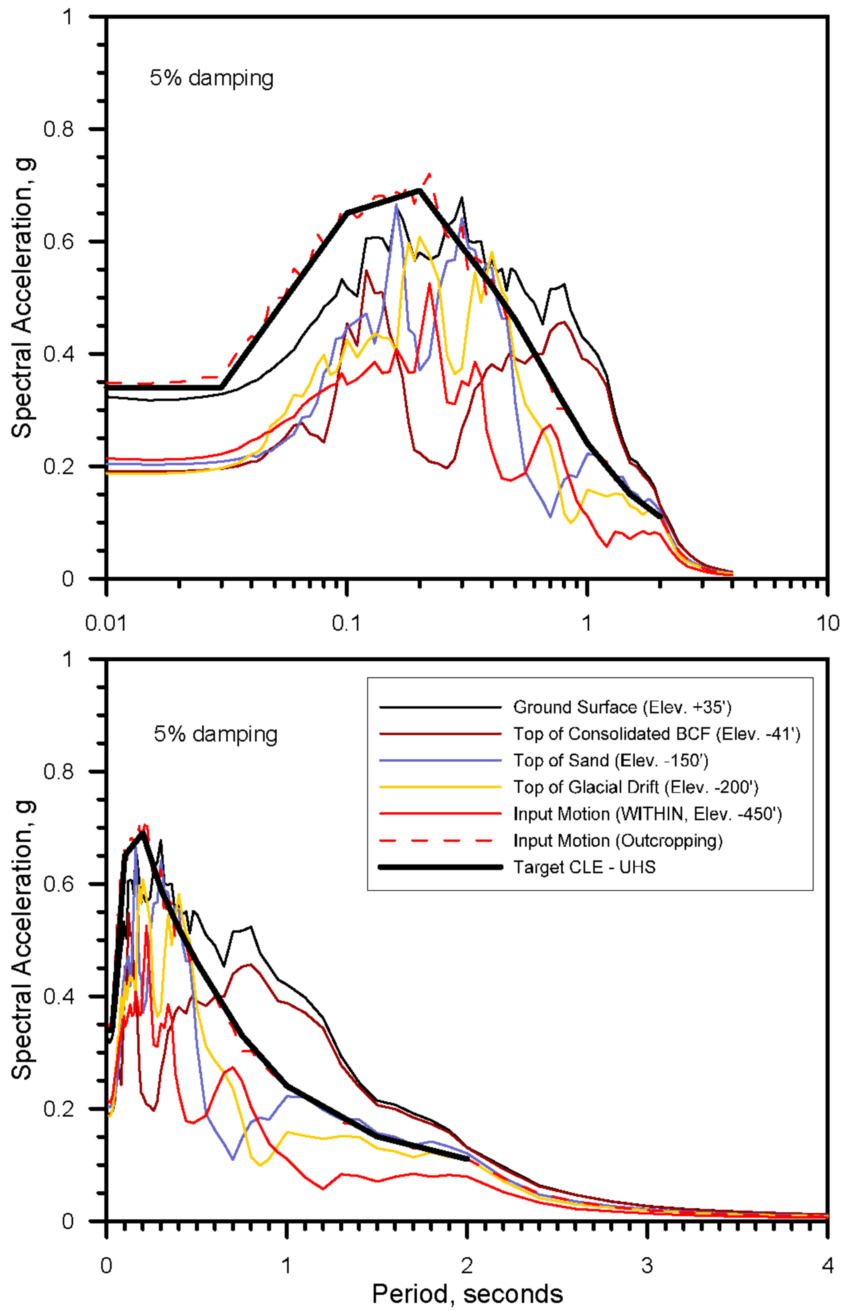


FIGURE 3.2-11. Calculated Response Spectra – CLE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Landward Soil Profile

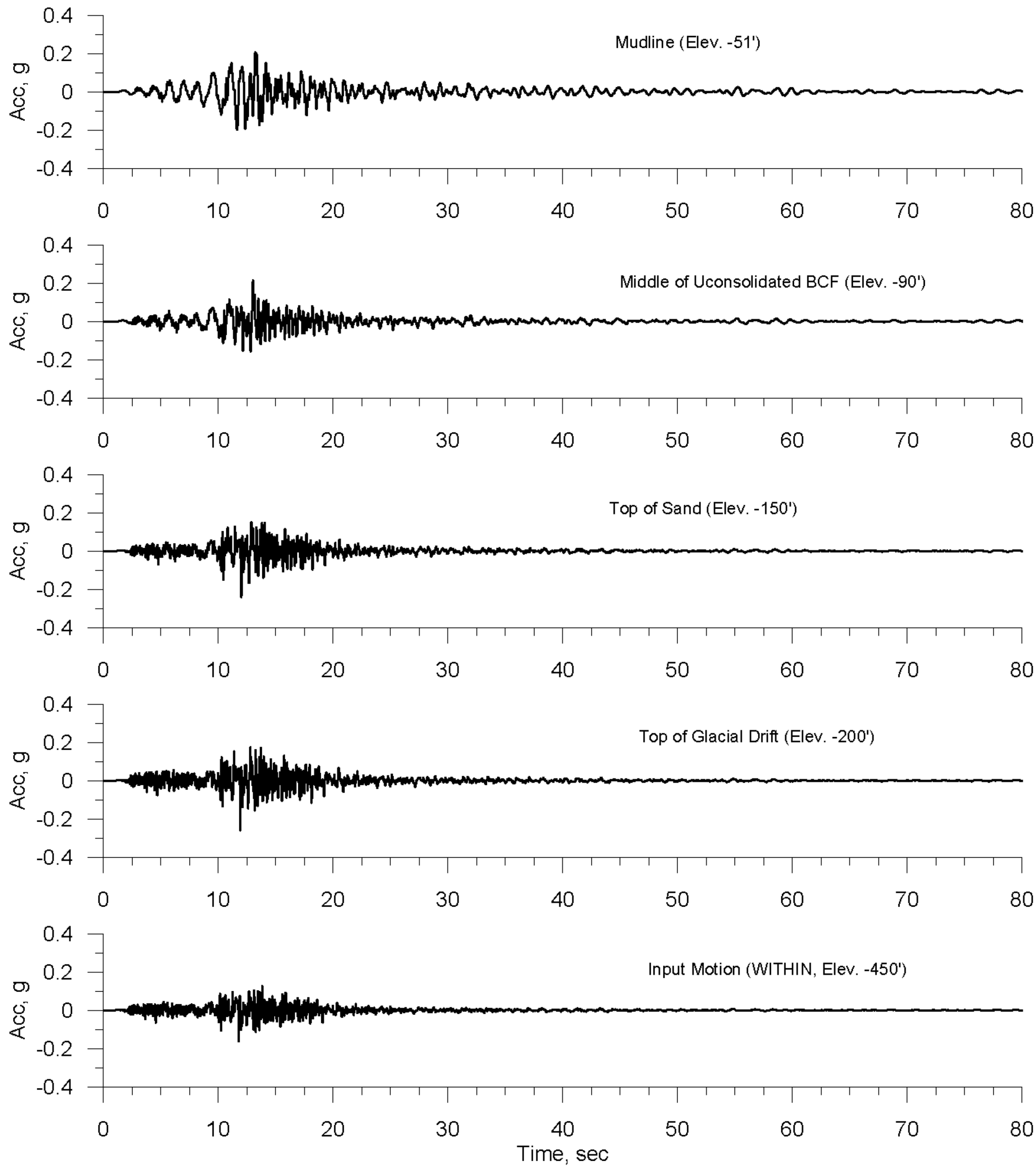


FIGURE 3.2-12. Calculated Time Histories – CLE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Seaward Soil Profile

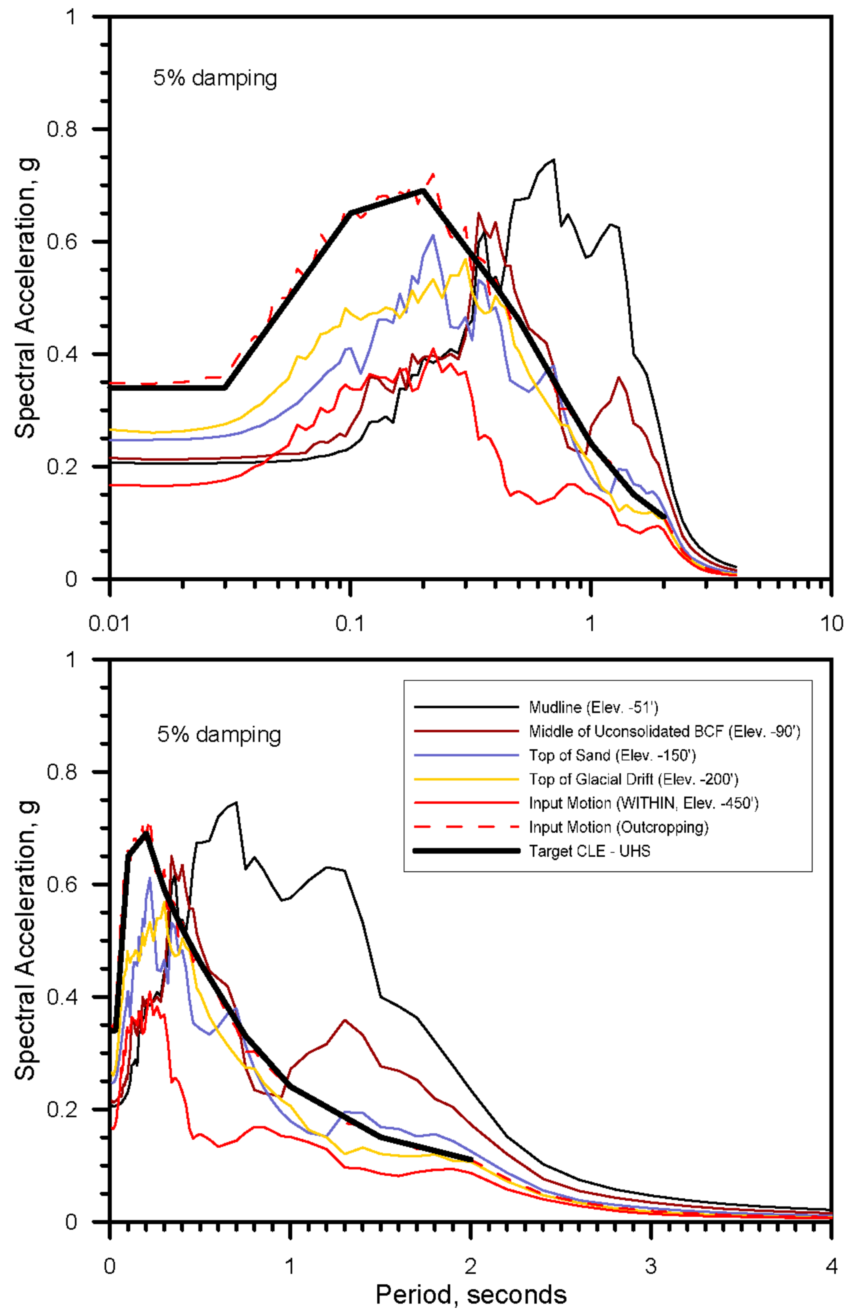


FIGURE 3.2-13. Calculated Response Spectra – CLE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Seaward Soil Profile

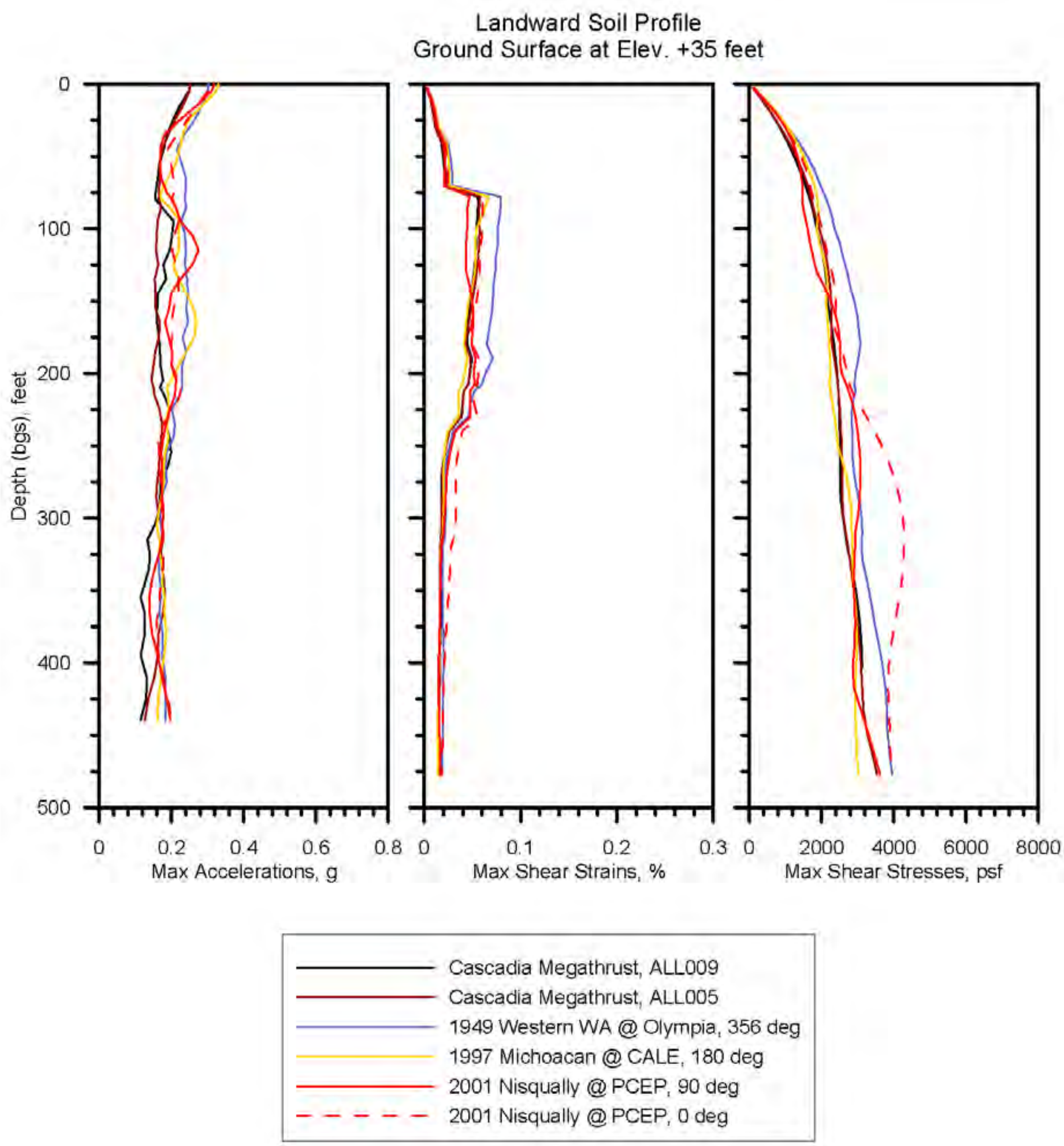


FIGURE 3.2-14. Calculated Maximum Accelerations, Shear Strains and Shear Stresses vs. Depth, Landward Soil Profile, CLE

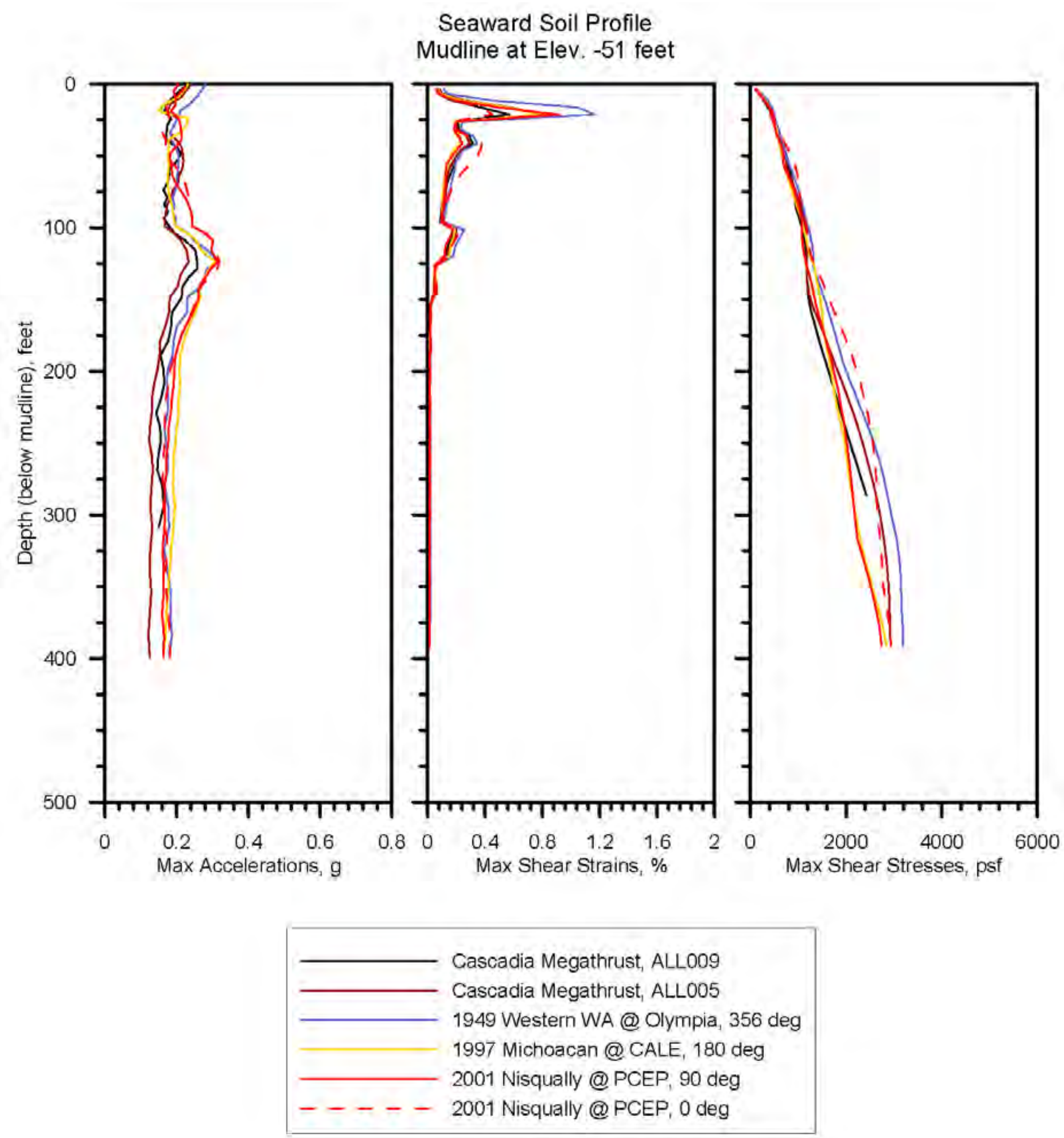


FIGURE 3.2-15. Calculated Maximum Accelerations, Shear Strains and Shear Stresses vs. Depth, Seaward Soil Profile, CLE

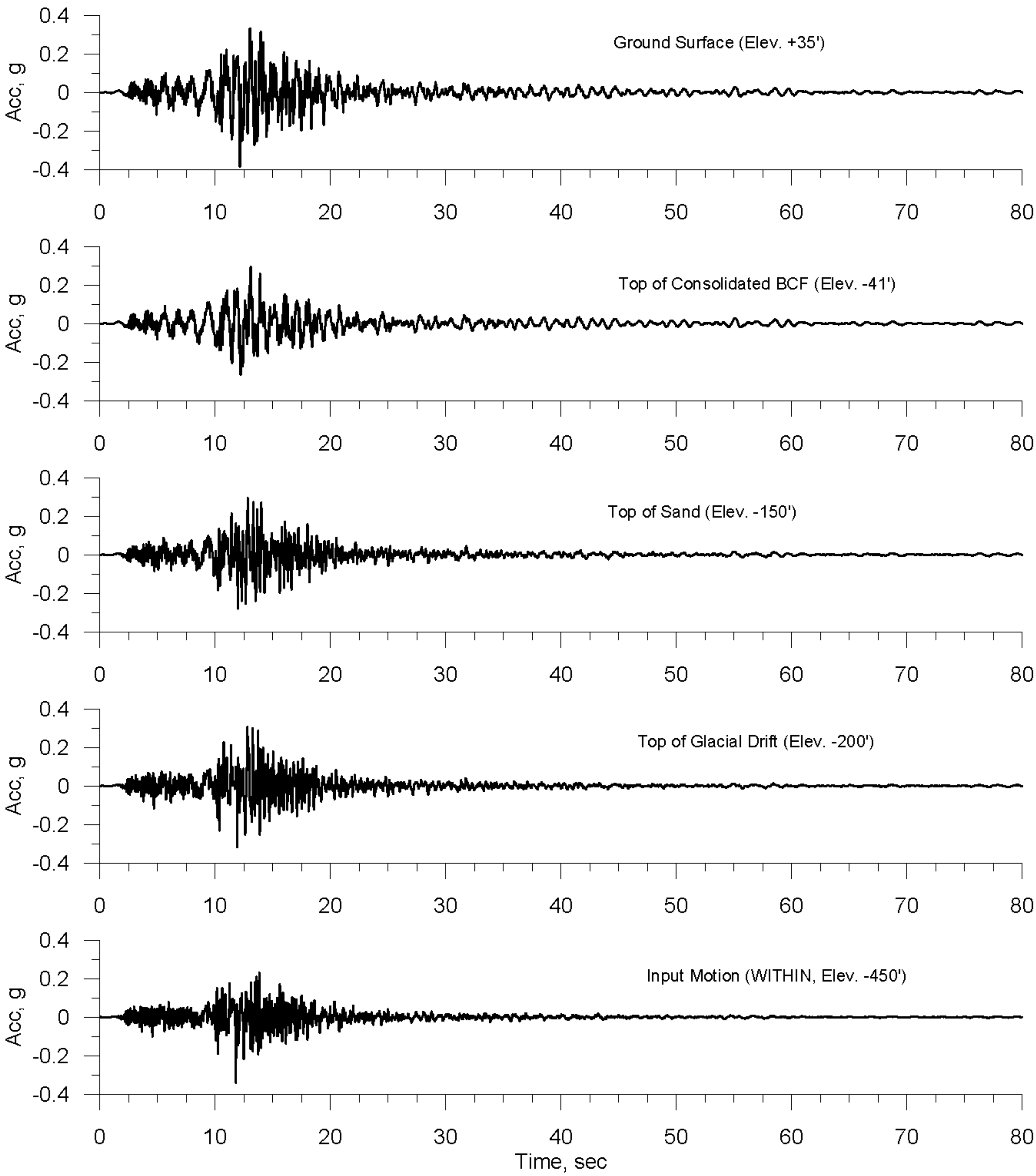


FIGURE 3.2-16. Calculated Time Histories – MCE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Landward Soil Profile

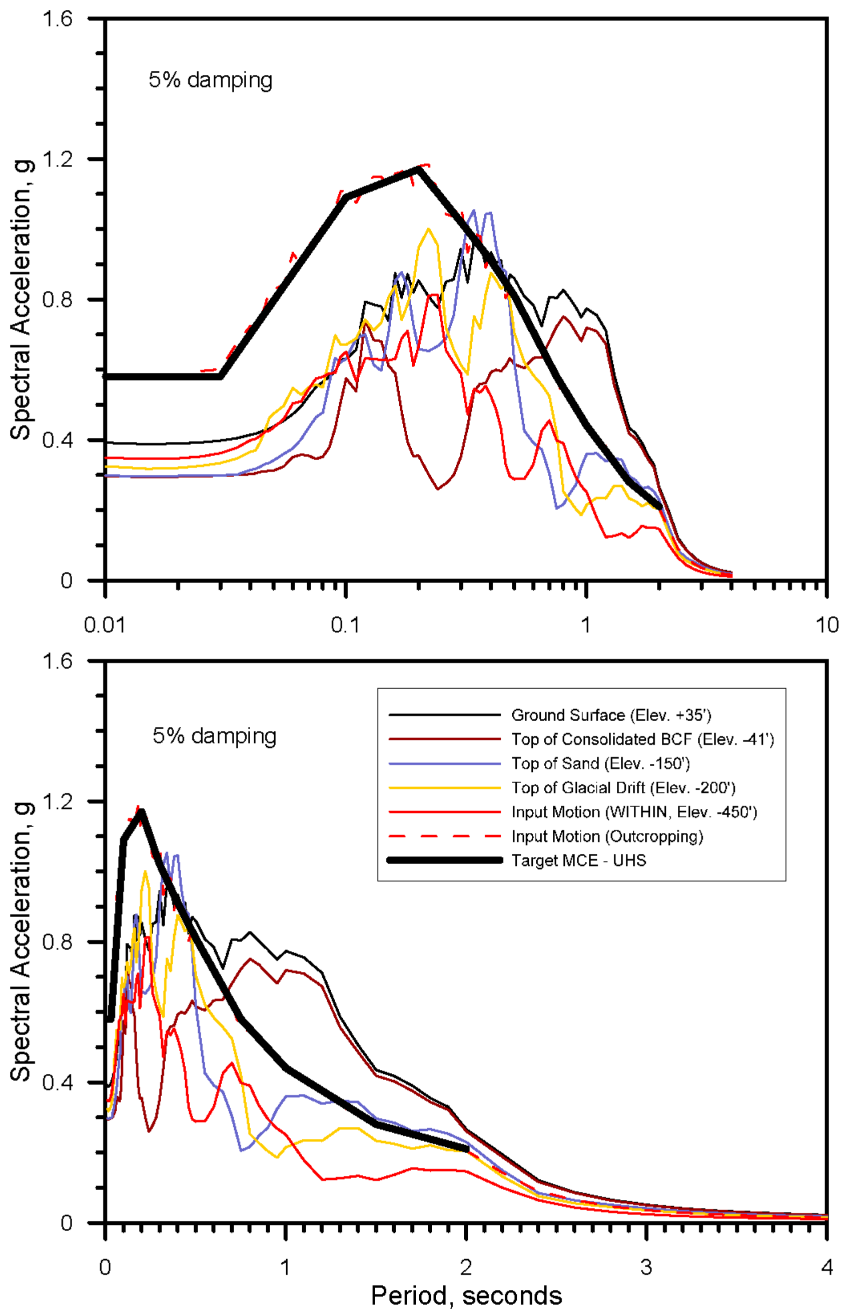


FIGURE 3.2-17. Calculated Response Spectra – MCE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Landward Soil Profile

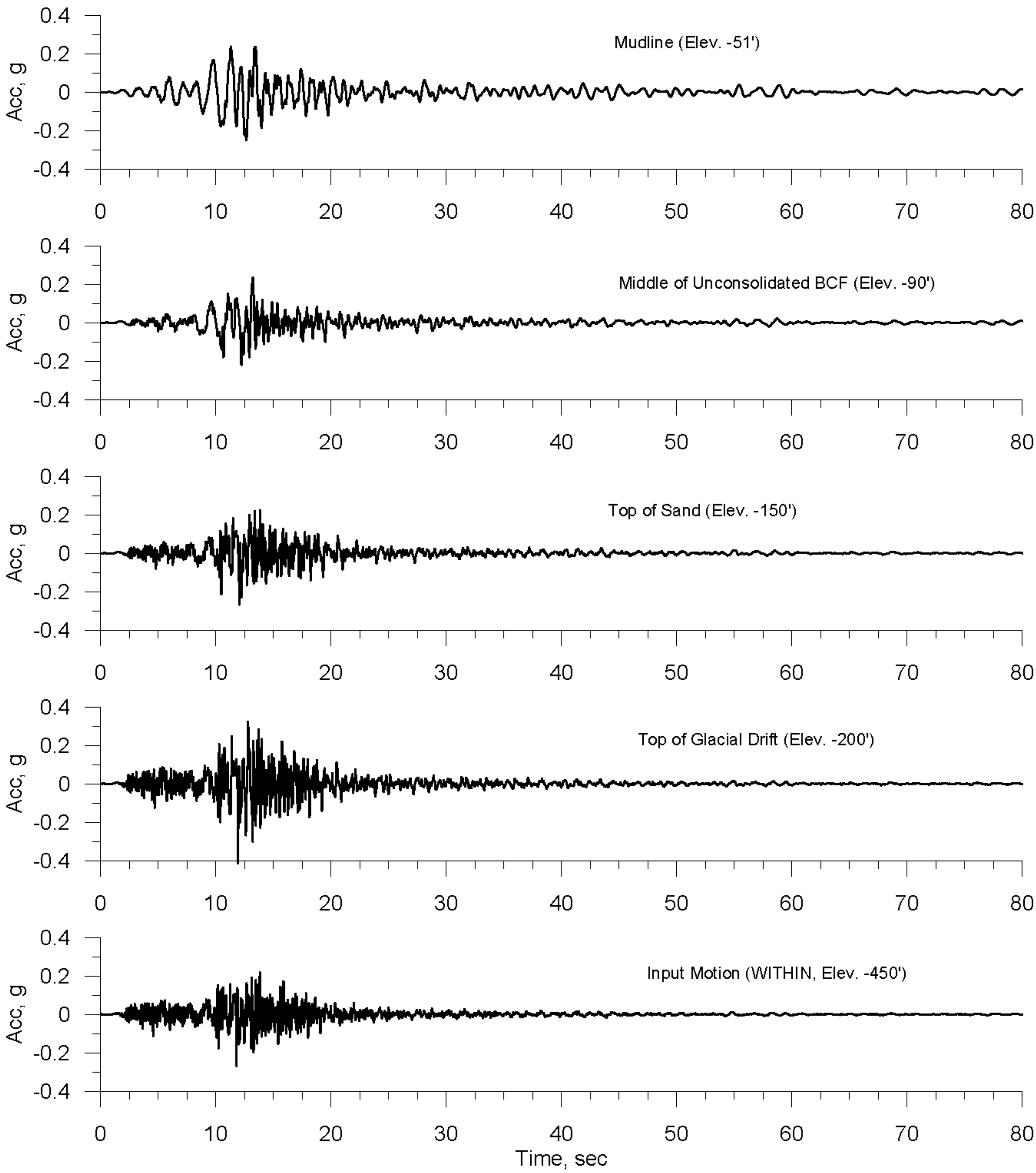


FIGURE 3.2-18. Calculated Time Histories – MCE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Seaward Soil Profile

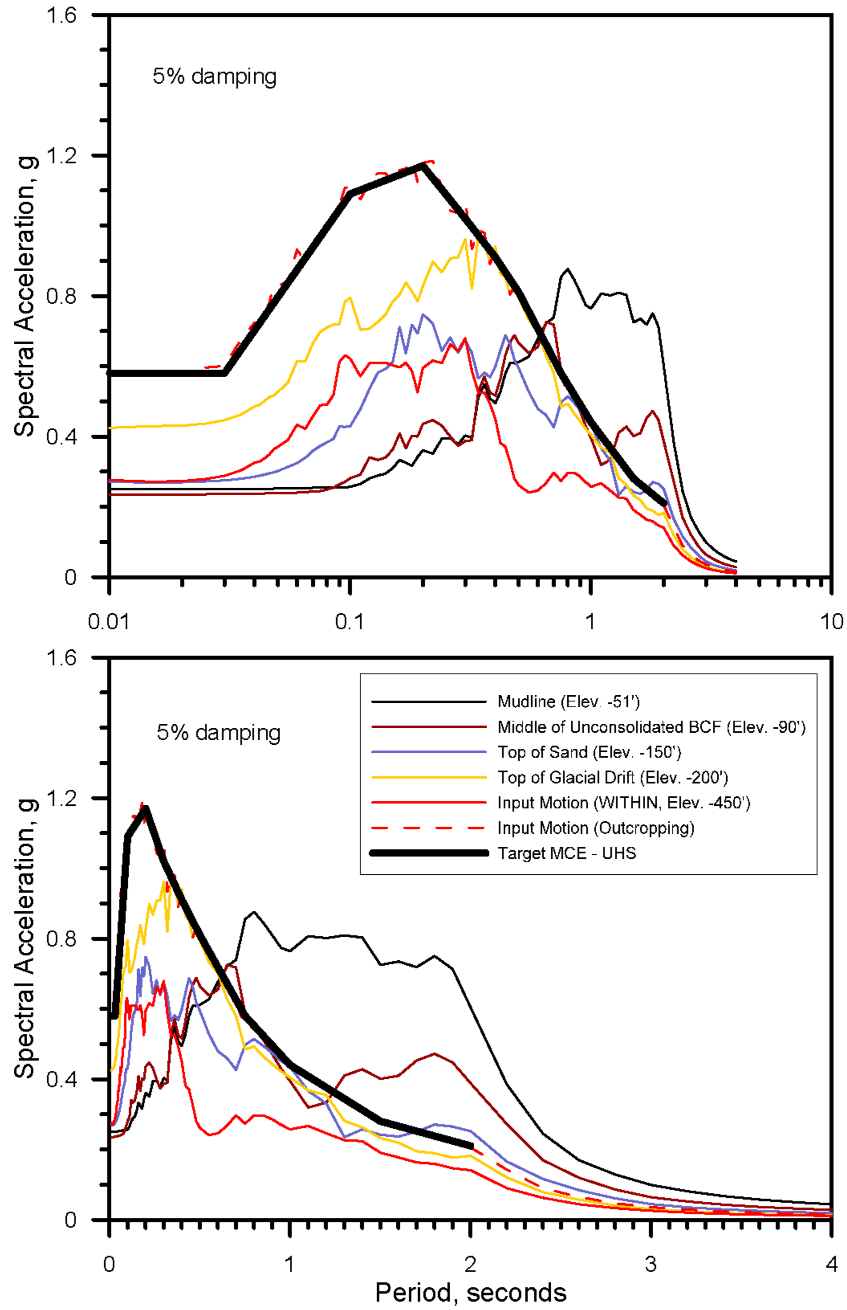


FIGURE 3.2-19. Calculated Response Spectra – MCE, 2001 Nisqually Earthquake at PCEP Station, 90 deg component, Seaward Soil Profile

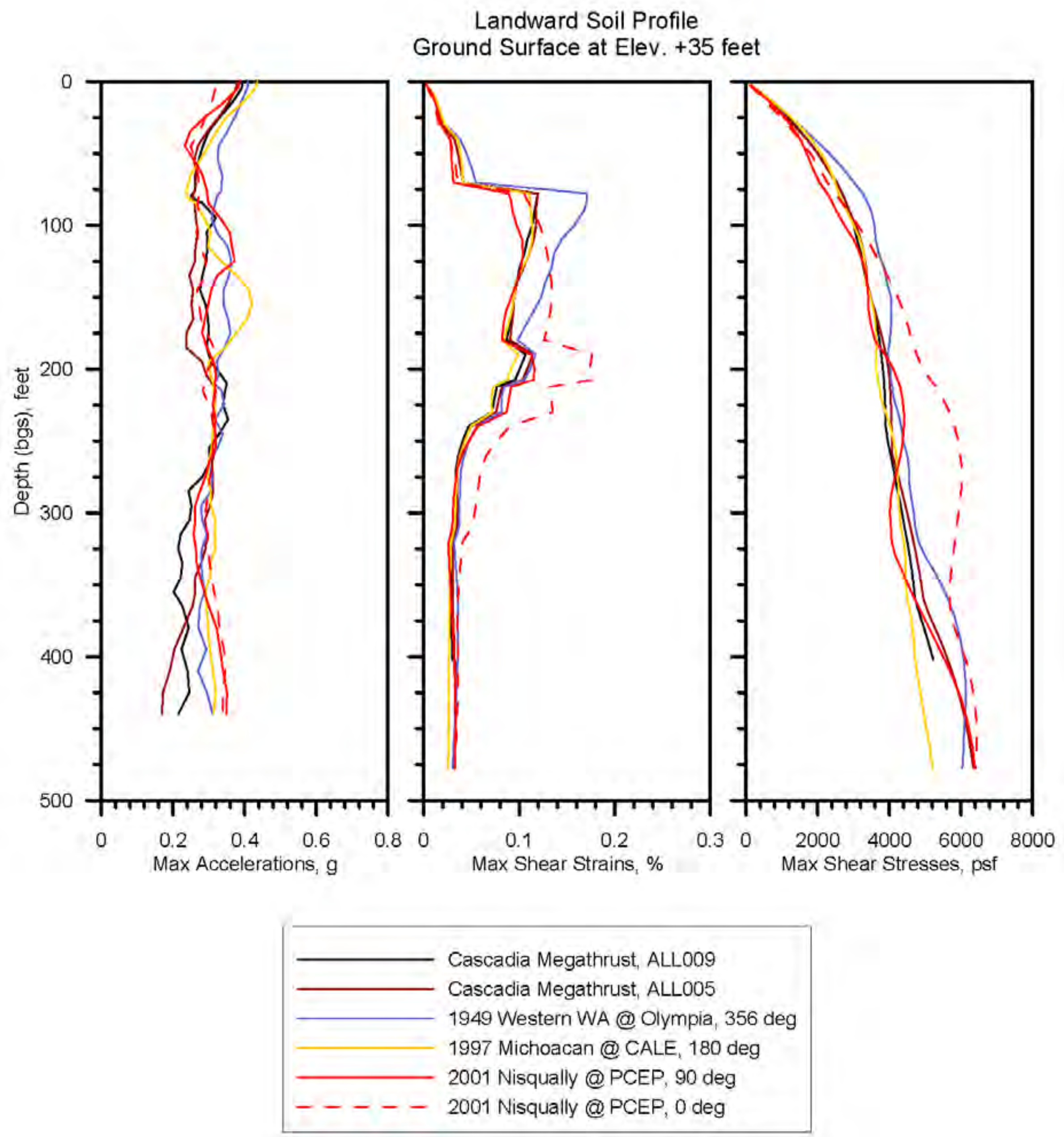


FIGURE 3.2-20. Calculated Maximum Accelerations, Shear Strains and Shear Stresses vs. Depth, Landward Soil Profile, MCE

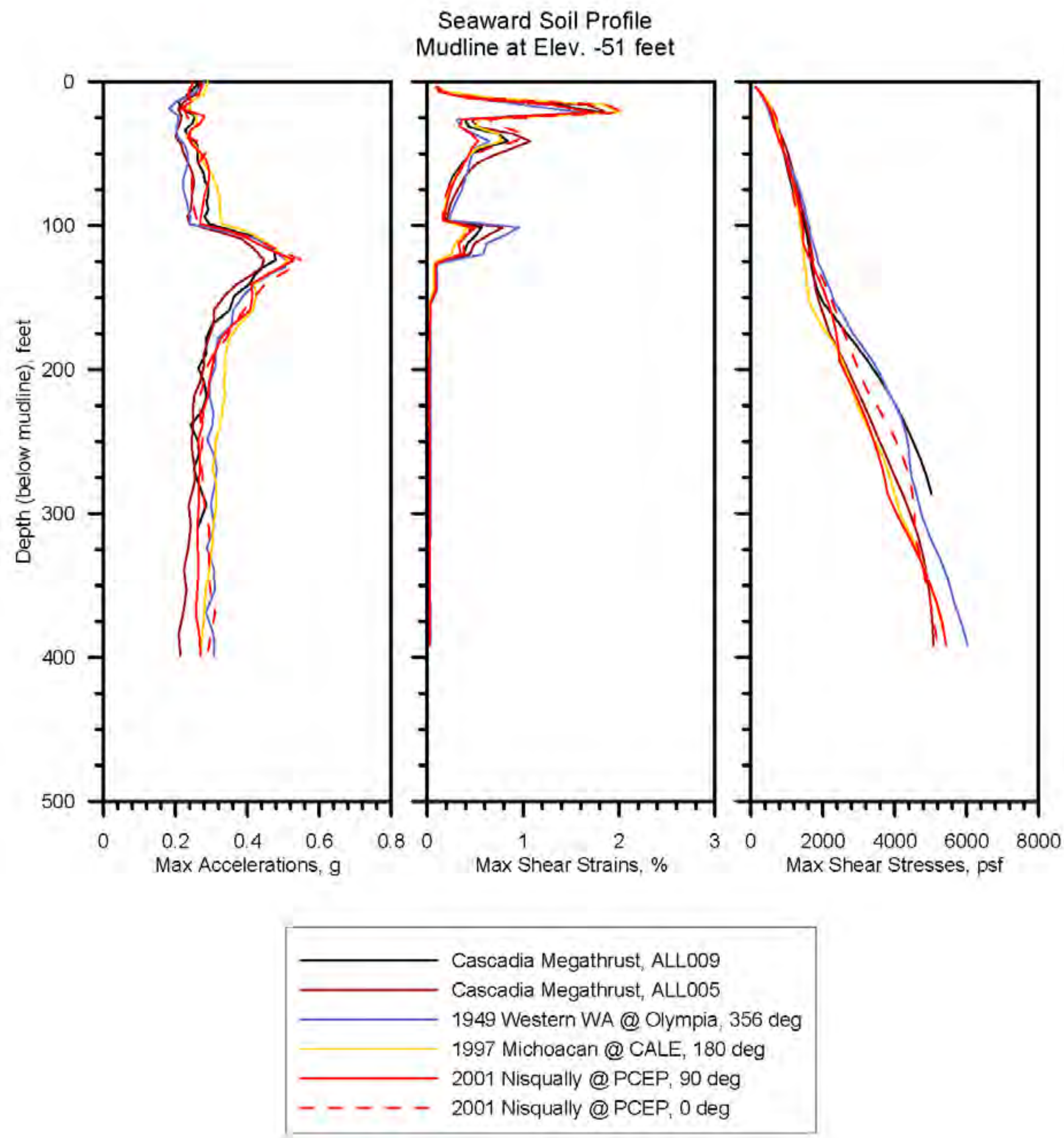


FIGURE 3.2-21. Calculated Maximum Accelerations, Shear Strains and Shear Stresses vs. Depth, Seaward Soil Profile, MCE

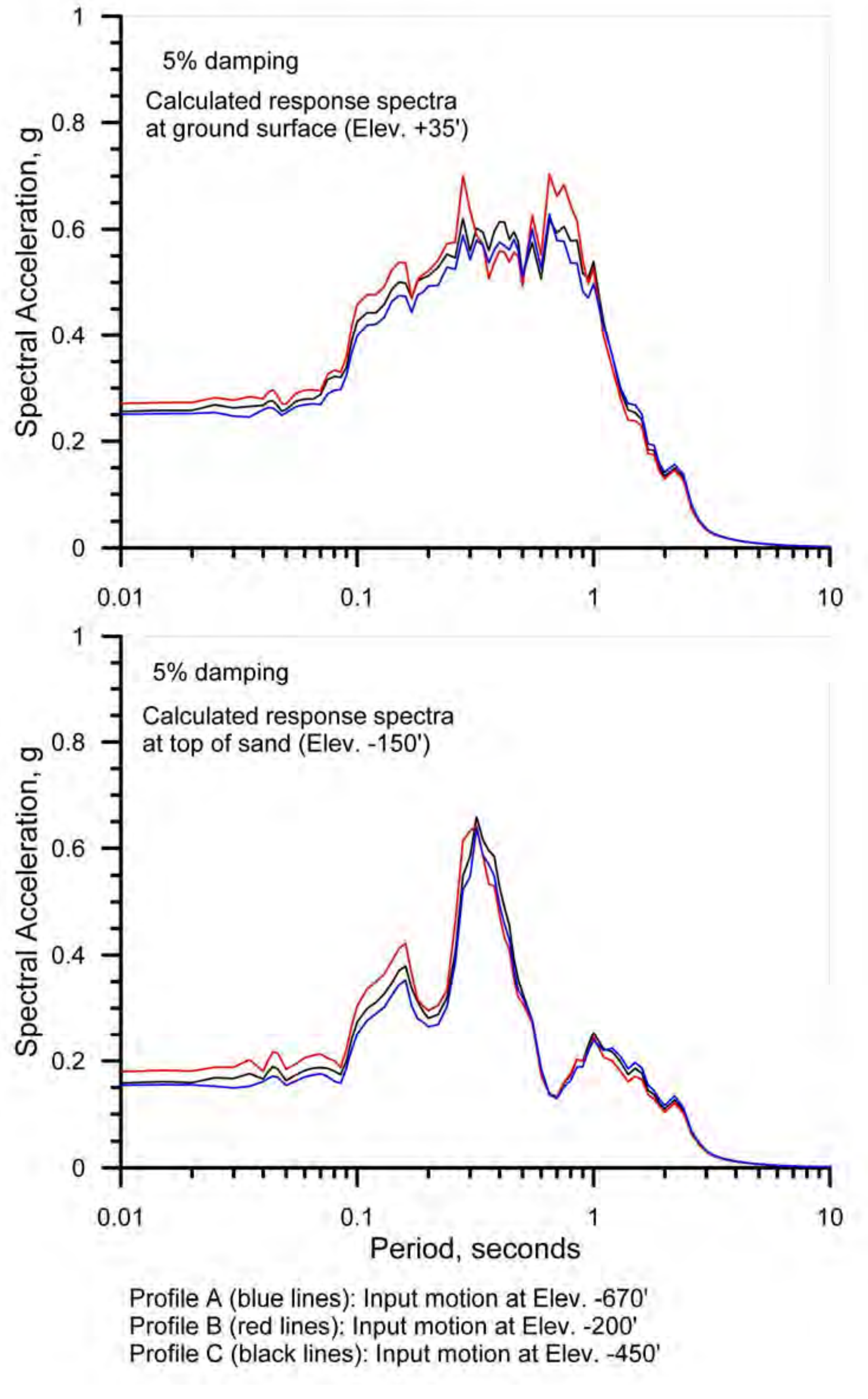


FIGURE 3.2-22. Sensitivity to Depth of Input Motions, Cascadia Megathrust Synthetic Earthquake – ALL005 (CLE), Landward Soil Profile

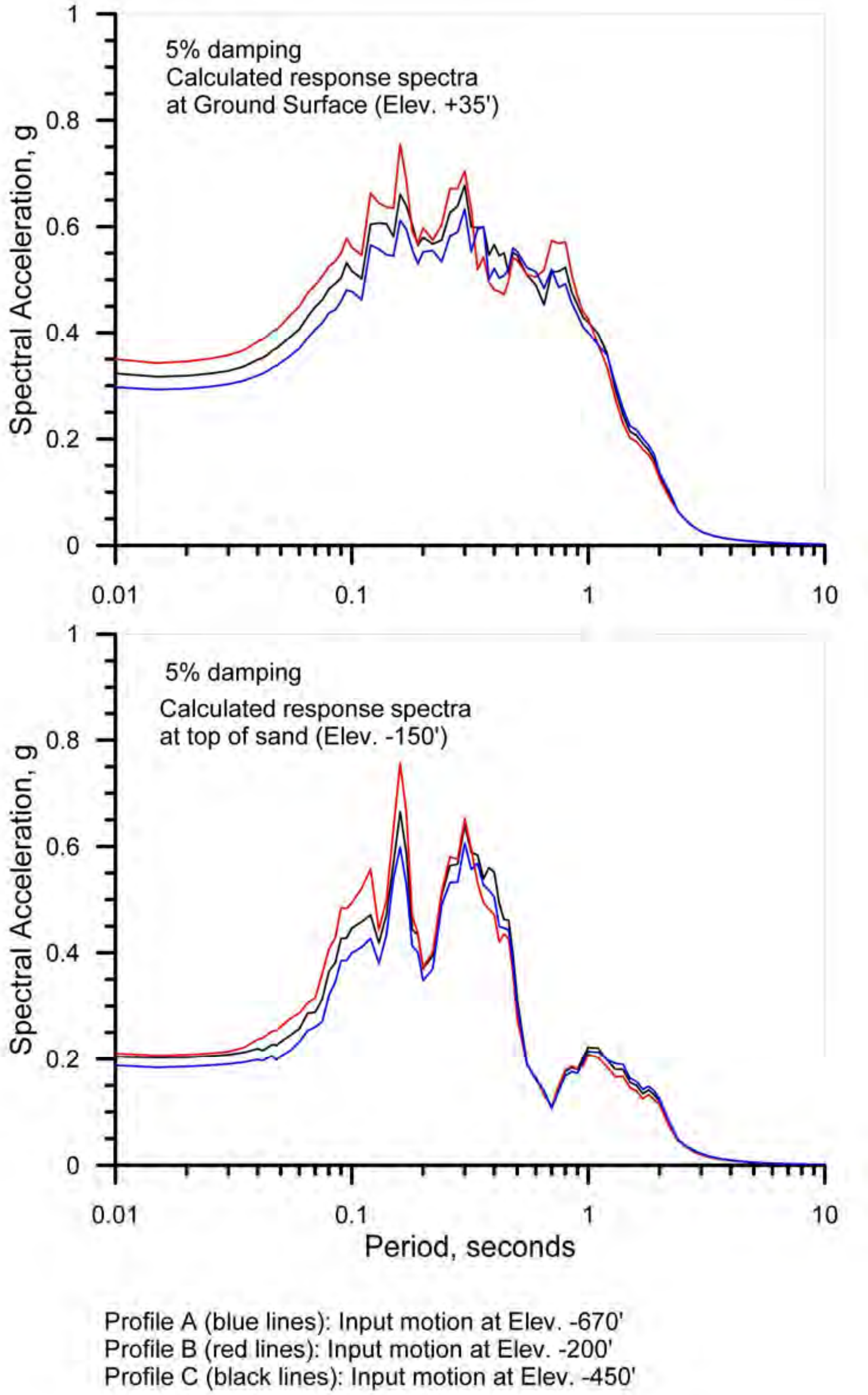


FIGURE 3.2-23. Sensitivity to Depth of Input Motions, 2001 Nisqually Earthquake at PCEP Station, 90 deg Component (CLE), Landward Soil Profile

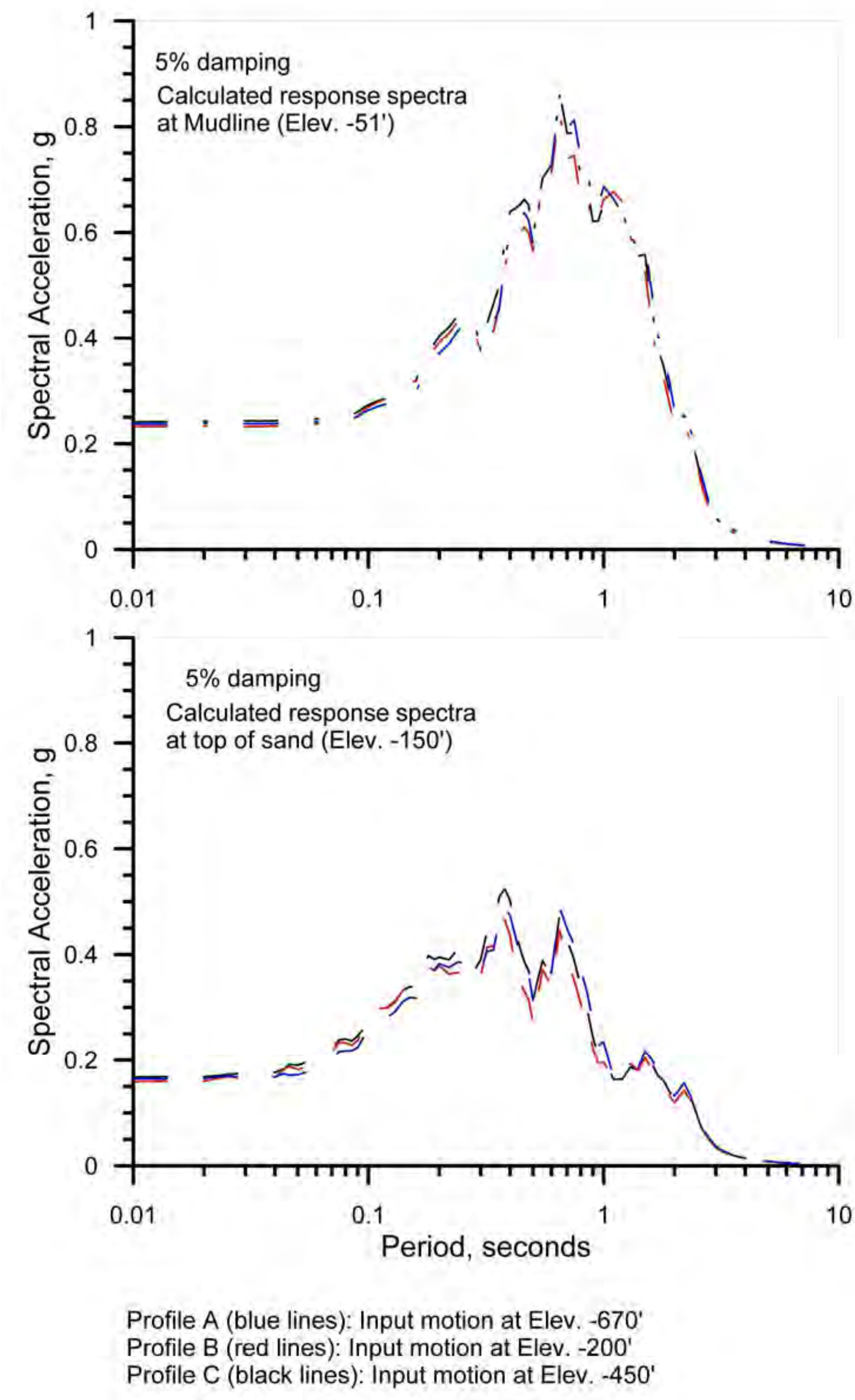


FIGURE 3.2-24. Sensitivity to Depth of Input Motions, Cascadia Megathrust Synthetic Earthquake – ALL005 (CLE), Seaward Soil Profile

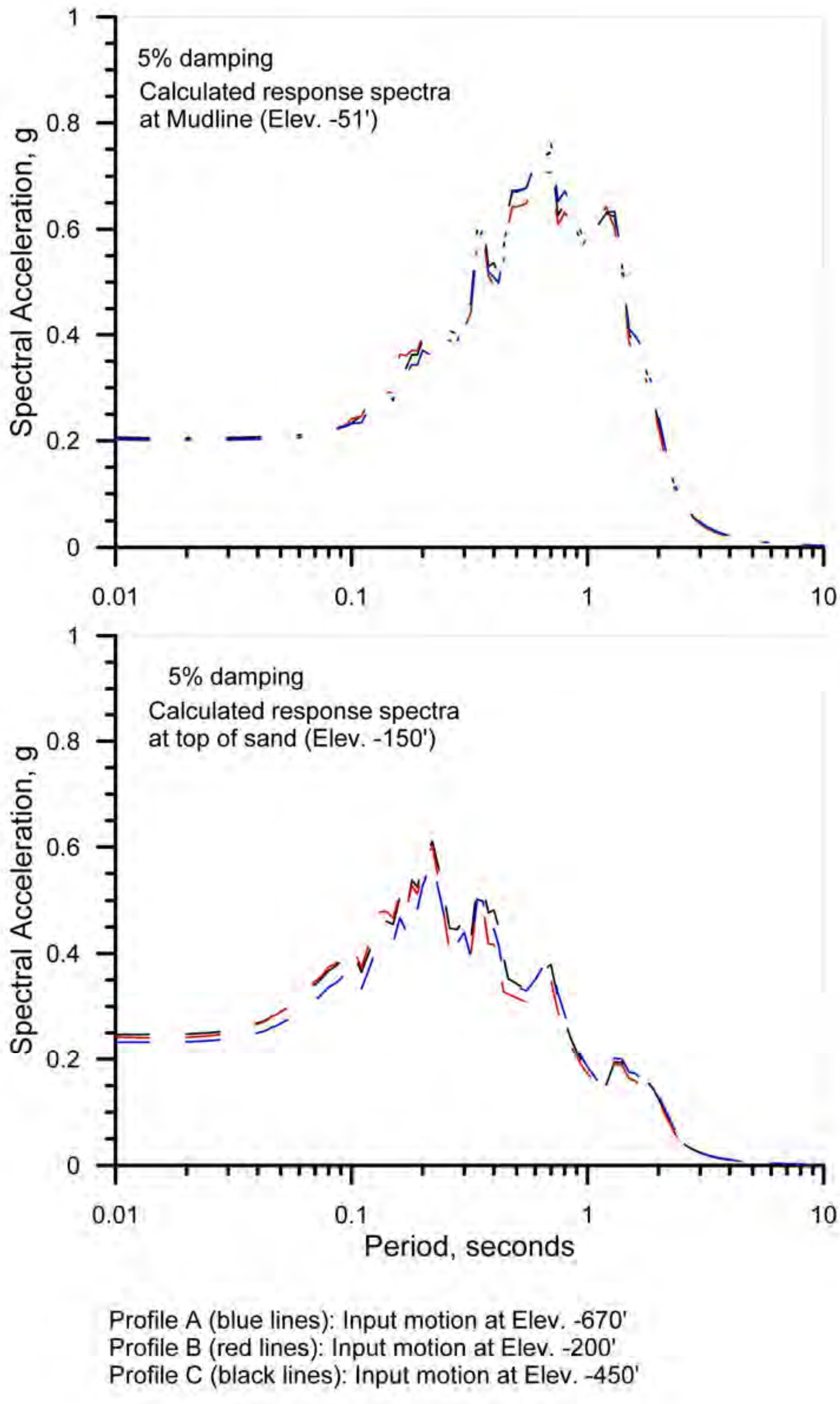


FIGURE 3.2-25. Sensitivity to Depth of Input Motions, 2001 Nisqually Earthquake at PCEP Station, 90 deg Component (CLE), Seaward Soil Profile

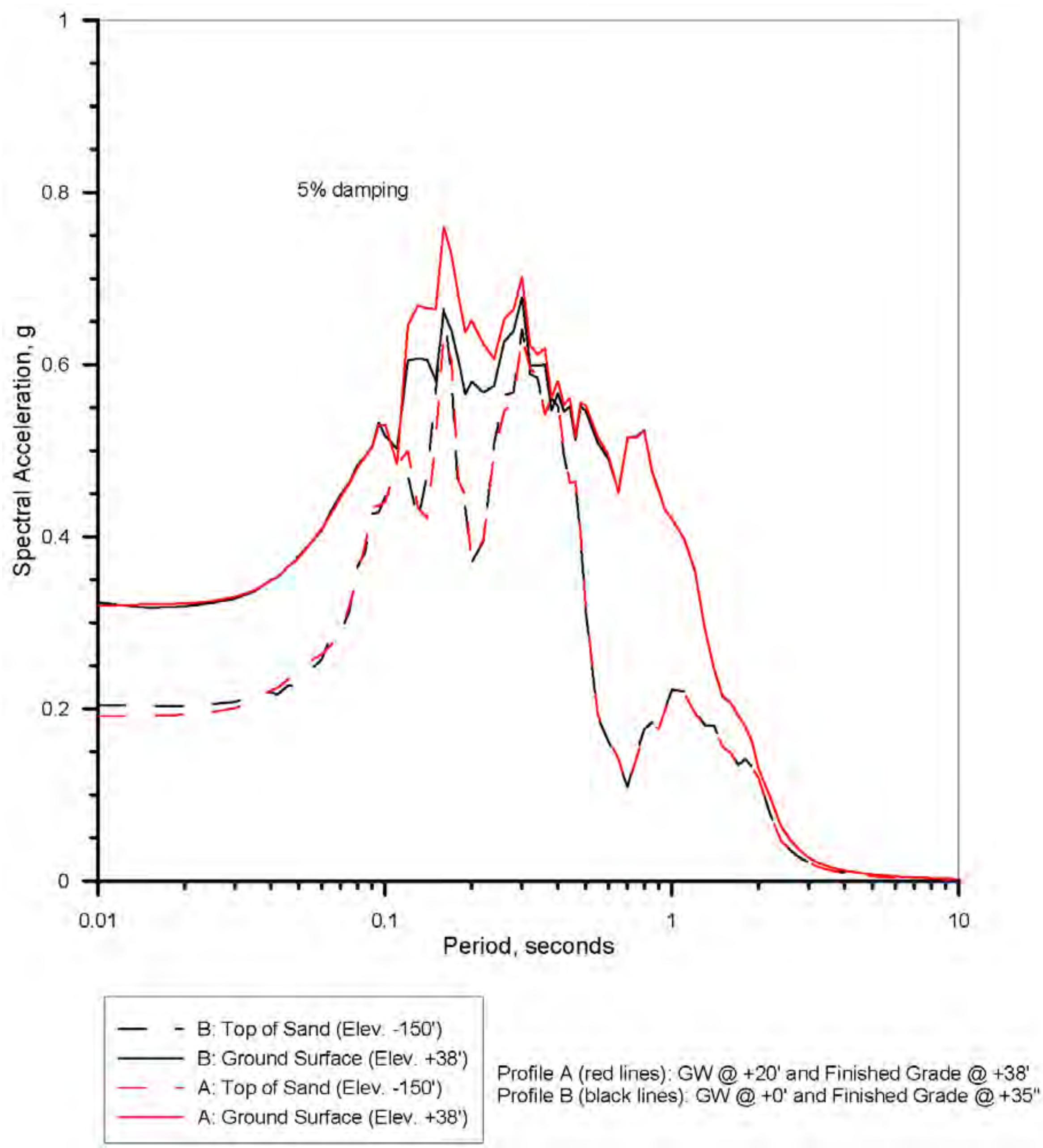


FIGURE 3.2-26. Sensitivity to Groundwater Depth and Finished Grade, 2001 Nisqually Earthquake at PCEP Station, 90 deg Component (CLE), Landward Soil Profile

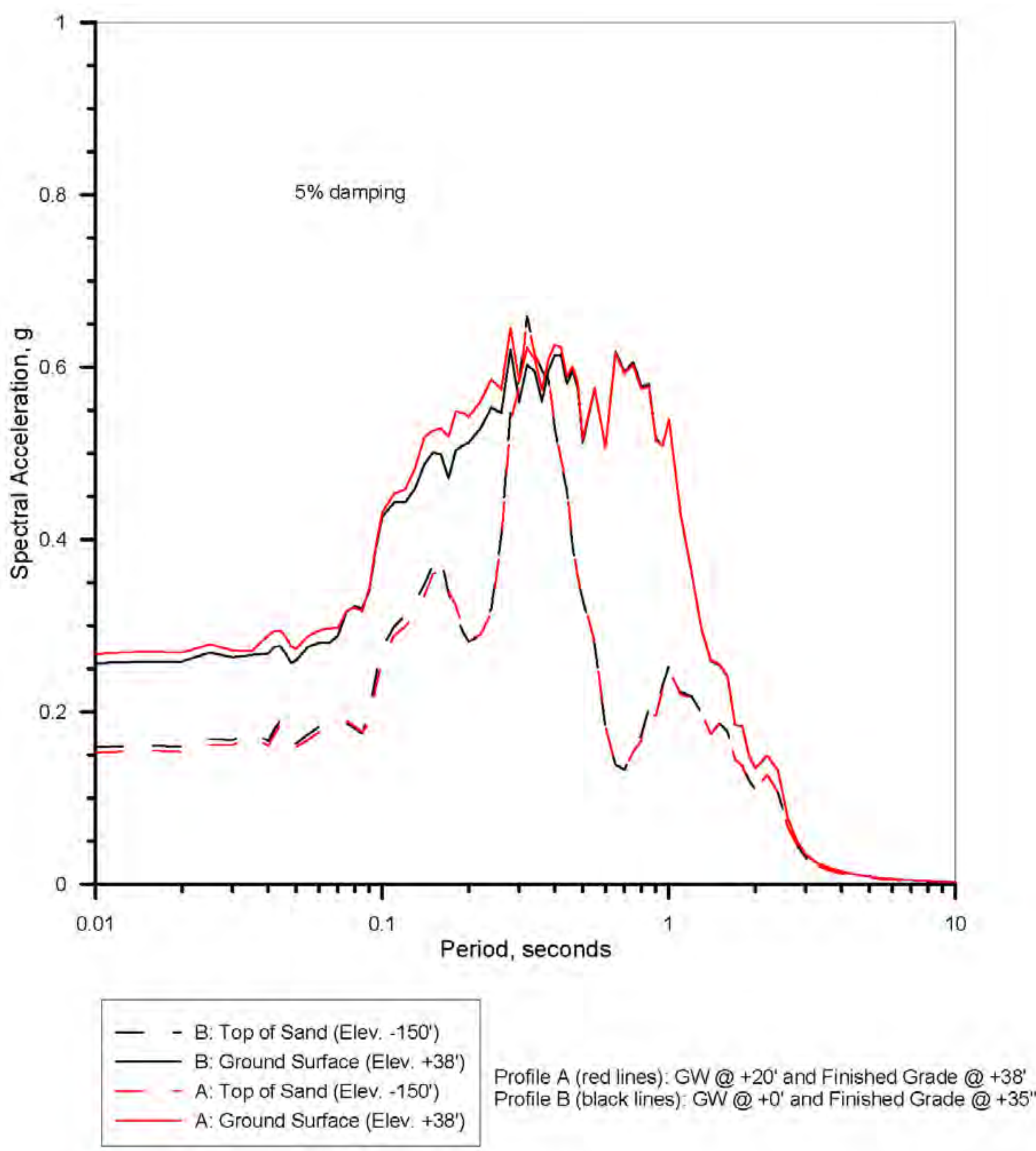


FIGURE 3.2-27. Sensitivity to Groundwater Depth and Finished Grade, Cascadia Megathrust Synthetic Earthquake, ALL005 (CLE), Landward Soil Profile

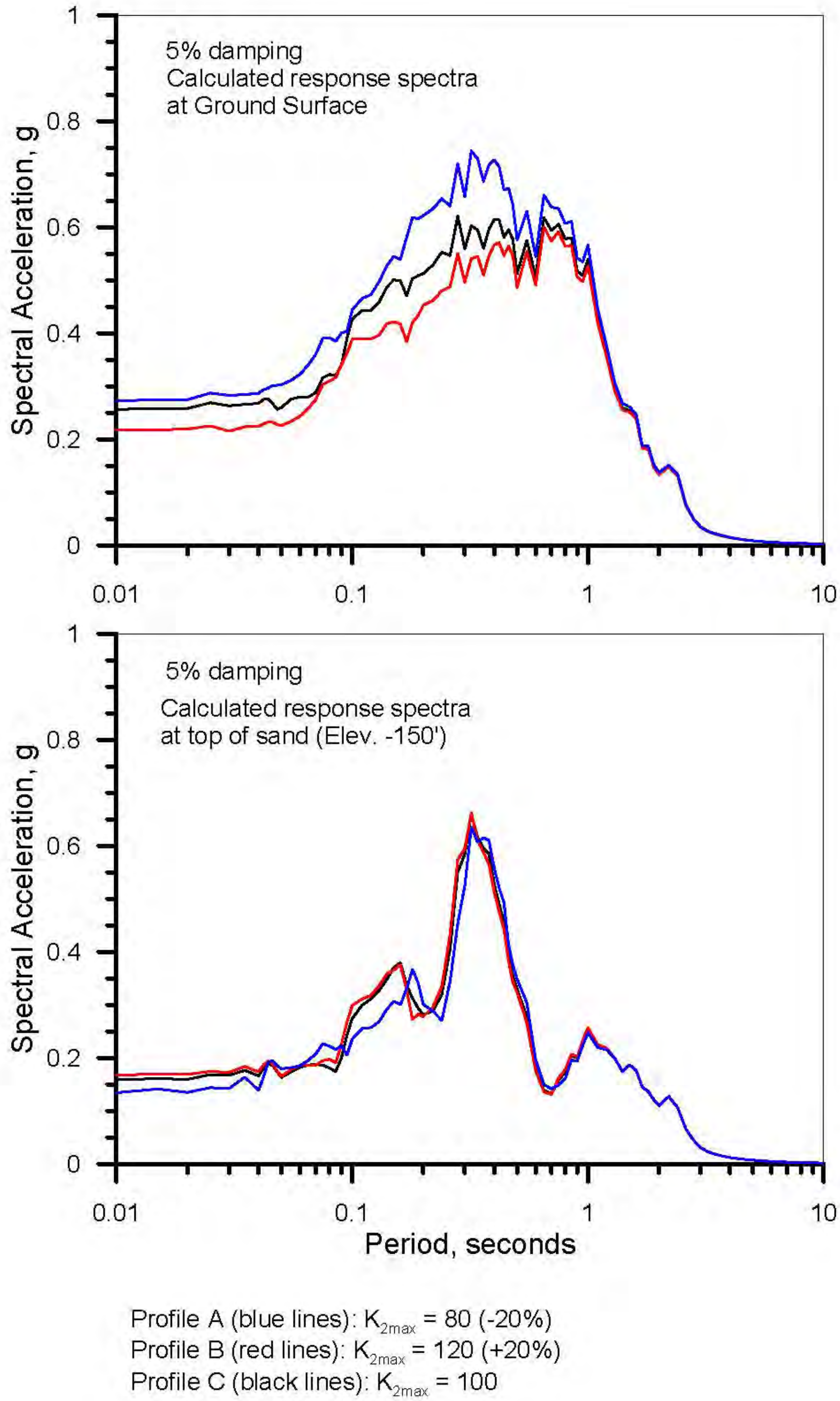


FIGURE 3.2-28. Sensitivity to Backfill Shear-wave Velocity, Cascadia Megathrust Synthetic Earthquake, ALL005 (CLE), Landward Soil Profile

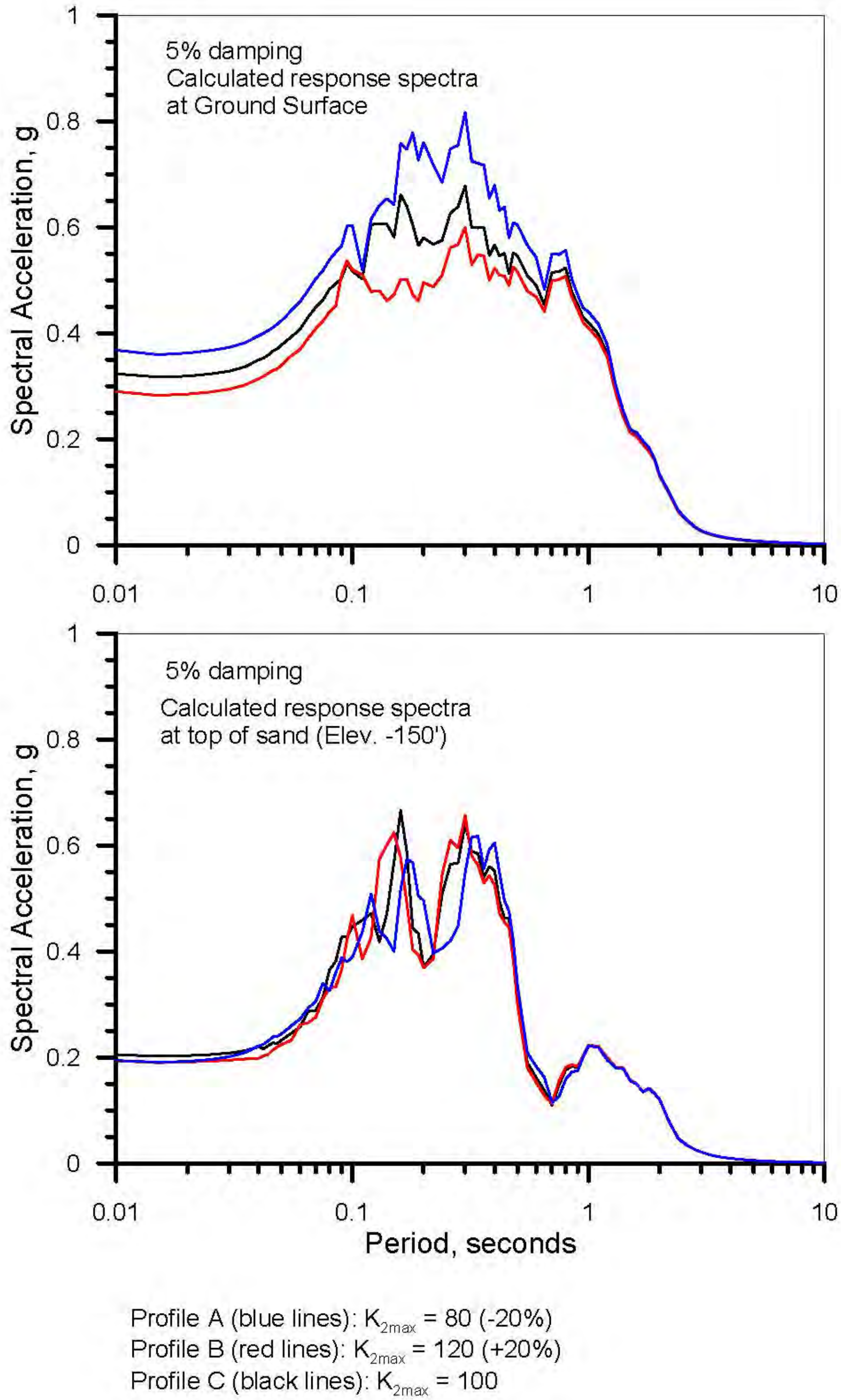


FIGURE 3.2-29. Sensitivity to Backfill Shear-wave Velocity, 2001 Nisqually Earthquake at PCEP Station, 90 deg Component (CLE), Landward Soil Profile

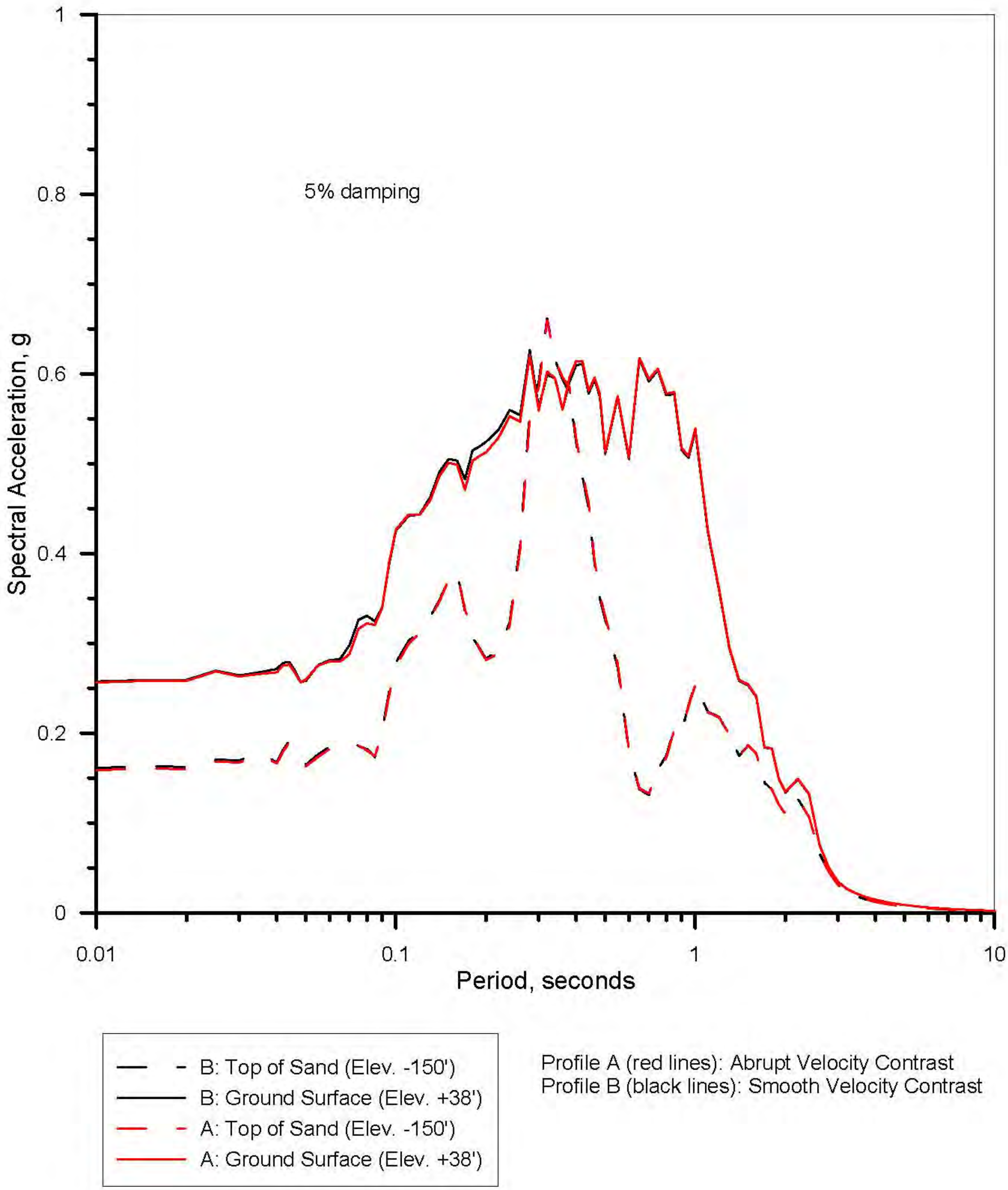


FIGURE 3.2-30. Sensitivity to Velocity Contrast at Interface between Backfill and BCF, Cascadia Megathrust Synthetic Earthquake, ALL005 (CLE), Landward Soil Profile

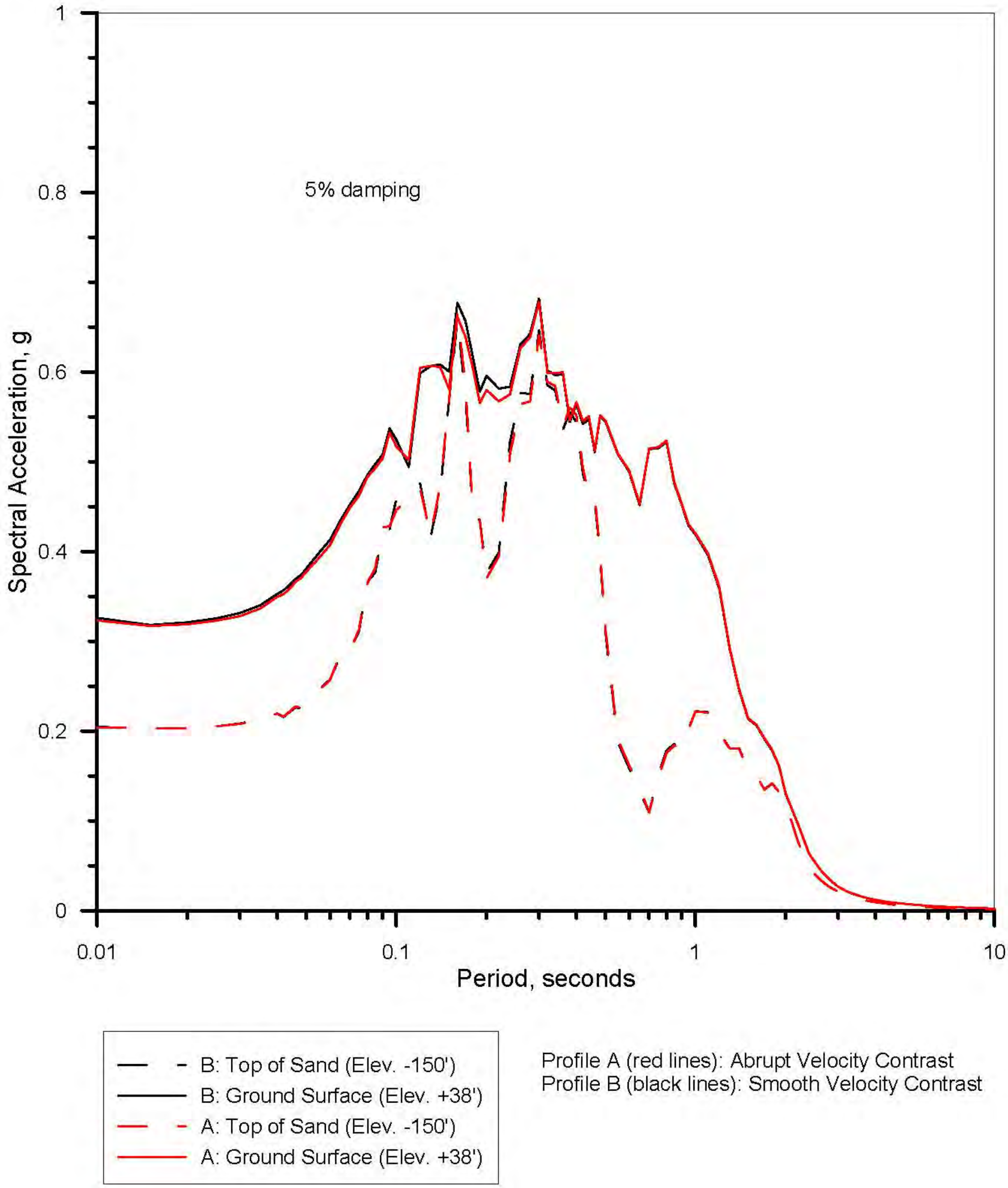


FIGURE 3.2-31. Sensitivity to Velocity Contrast at Interface between Backfill and BCF, 2001 Nisqually Earthquake at PCEP Station, 90 deg Component (CLE), Landward Soil Profile

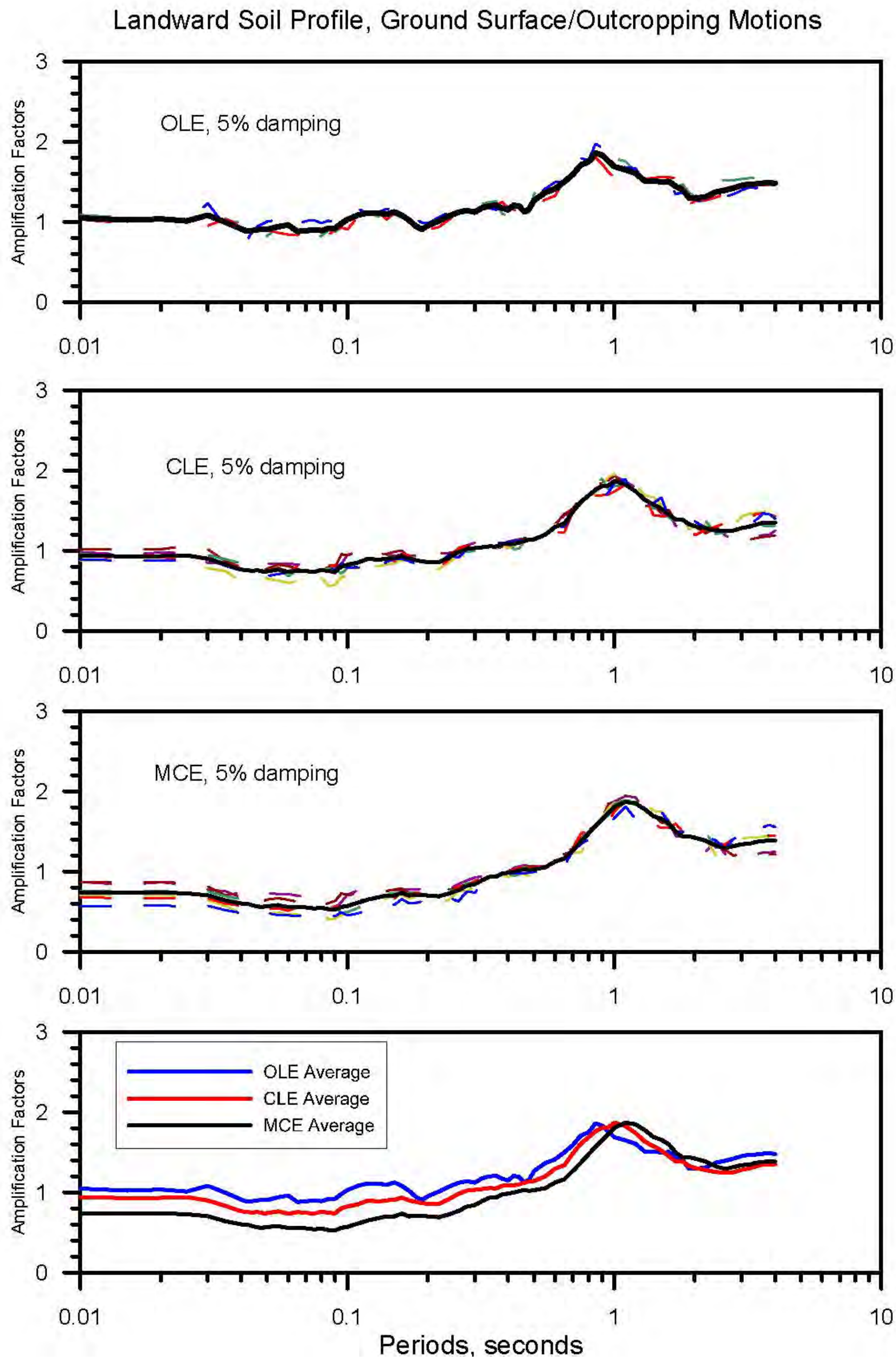


FIGURE 3.2-32. Spectral Amplification Factors. Ground Surface Over Outcropping Motions, Landward Soil Profile

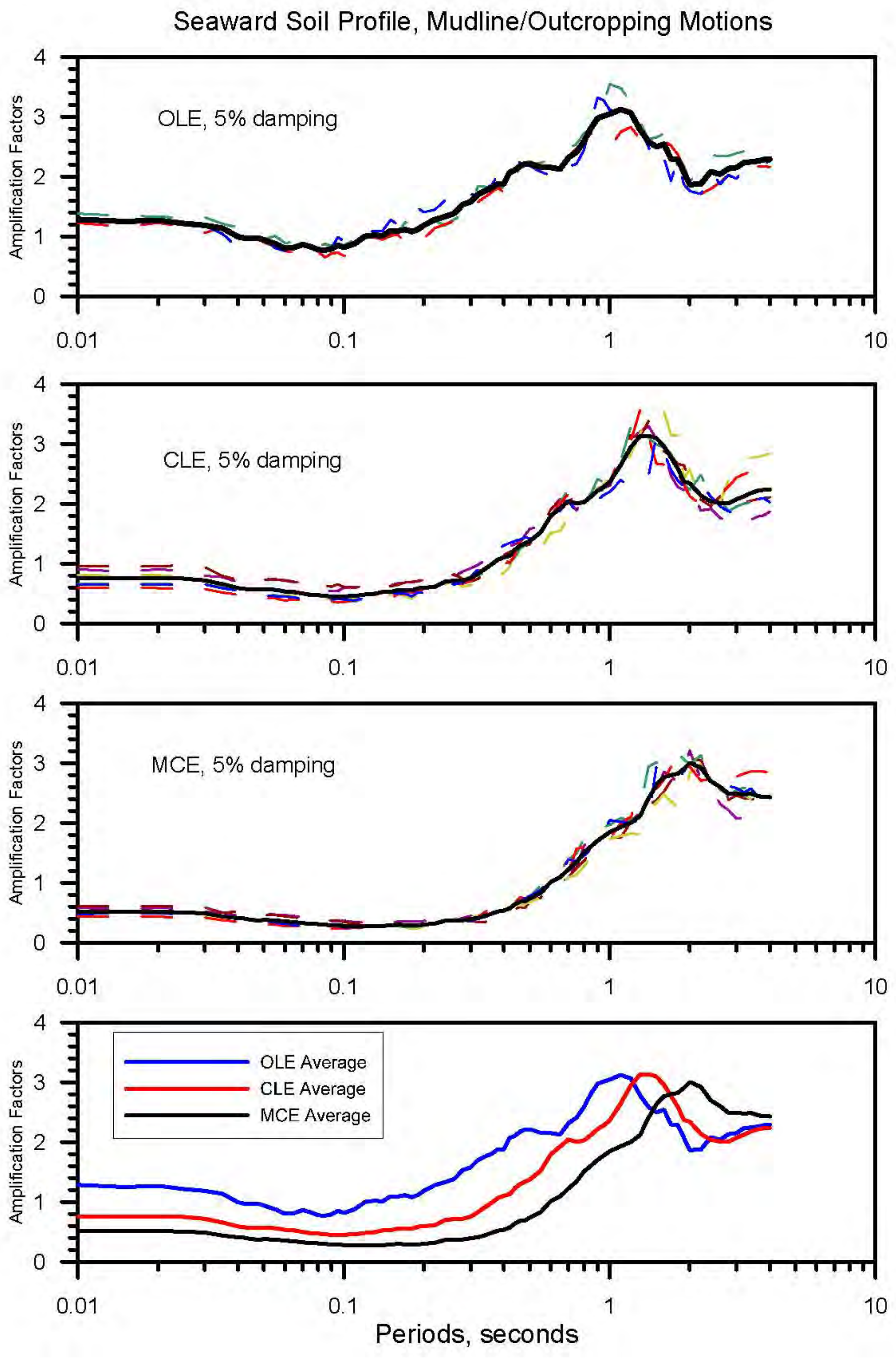


FIGURE 3.2-33. Spectral Amplification Factors, Mudline Over Outcropping Motions, Seaward Soil Profile

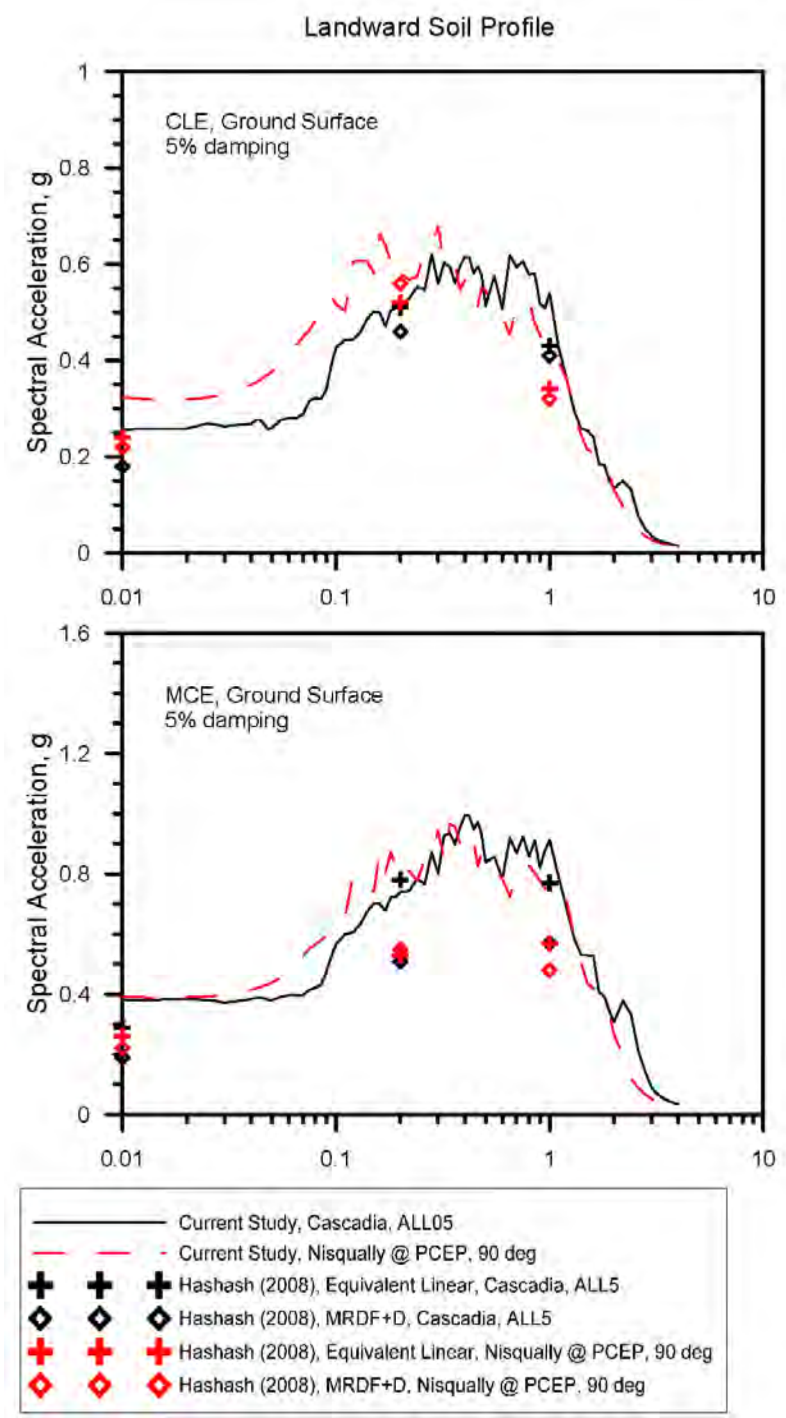


FIGURE 3.2-34. Comparison of Results with Previous Study, Landward Soil Profile

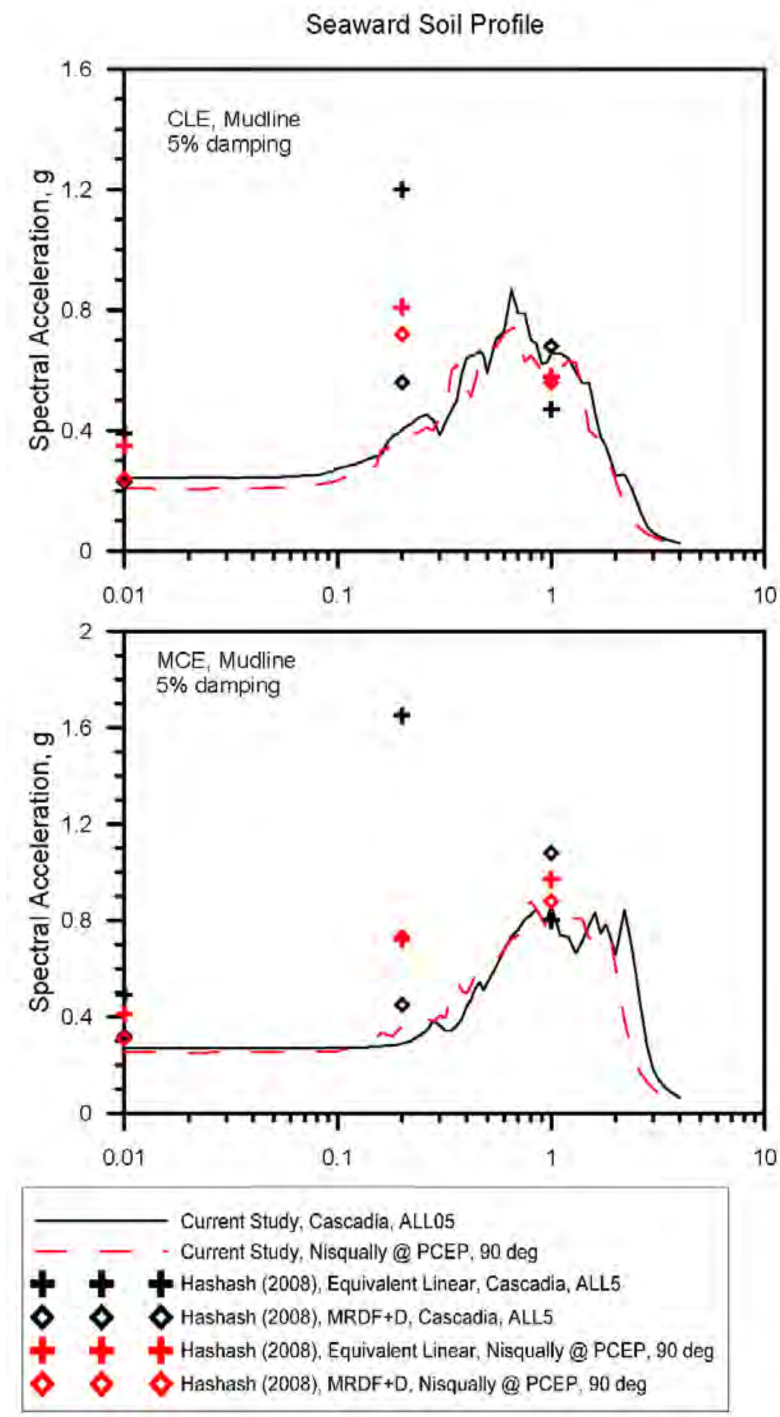


FIGURE 3.2-35. Comparison of Results with Previous Study, Seaward Soil Profile