SECTION 6

Structural Analysis

This section summarizes the structural evaluation of the as-built condition of the OCSP® structure constructed at the PIEP. The evaluation reviews the differences between the as-built condition and the original design, the critical elements of construction, and potential impacts of construction deviations. This section also presents an evaluation of life-cycle performance of the OCSP® system, including the impact of corrosion on the life-cycle performance of the bulkhead structure, operation and maintenance considerations, and life-cycle cost. The structural evaluation for the independent Open Cell bulkhead design performed by CH2M HILL is presented in Section 9.

6.1 As-Built Condition Analysis

The primary source documents used to review the original design as well as the as-built conditions of the OCSP® system include the following:

- 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*.
- ICRC. 2010. *Port of Anchorage Expansion, Wet Barge Berth and North Extension Preliminary Inspection Summary Report*.
- ICRC. 2011b. *Memorandum for Record – Cell 32 Damage*.
- PND. 2007a. *Port of Anchorage Expansion Project, Dry Barge Berth Conformed Drawings*.
- PND. 2008a. *Port of Anchorage Expansion Project, Barge Berth Phase 2 As-Built Drawings*.
- PND. 2008c. *Port of Anchorage Expansion Project, North Extension Conformed Drawings*.
- PND. 2008d. *Port of Anchorage Expansion Project, Barge Berth Phase 2 Conformed Drawings*.
- PND. 2010b. *Port of Anchorage Expansion Project, North Extension As-Built Drawings*.
- PND. 2010c. *Port of Anchorage Expansion Project, North Extension Bulkhead Drawings*.
- PND. 2010i. *Port of Anchorage Expansion Project, North Extension Bulkhead As-Built Drawings*.
- PND. 2011b. *Port of Anchorage Marine Terminal Redevelopment, Extended Wet Barge Berth Bulkhead 100% Plans*.
- PND. 2011c. *Port of Anchorage Expansion, Bulkhead Condition Analysis and Recommendations*.

In addition to the source documents listed above, the review also covered various construction documents, including requests for information (RFI), constructions observations, pile-driving records, and construction photos.

6.1.1 Overview of the As-Built Conditions

Figure 1.2-3 shows the overall limits of the PIEP North Expansion constructed to date. Four main areas of the bulkhead structure have been constructed: the Dry Barge Berth, the Wet Barge Berth, and North Extension 1 and 2. The typical bulkhead sheet pile lengths (from the top of the sheet pile to the tip of the sheet pile) at each of the three areas are approximately 46 feet, 70 feet, 80 feet, and 90 feet, respectively. With the exception of some end cells, the OCSP® bulkhead structures constructed to date consist of identical cells of 27.5-foot radius, as shown in Figure 6.1-1. The arched face of each typical cell is composed of 17 PS31 sheet piles (together denoted as "face sheets") connected through three-point "finger-and-thumb" interlock connections (Figure 6.1-2). These piles were driven vertically into the BCF clay layer. The face sheets of two adjacent cells meet at a 120 degree angle at the wye connection pile (Figure 6.1-3). The two adjacent cells share one common sheet pile wall (referred to as the tailwall) consisting of PS27.5 sheet piles. The tailwall is also connected to the wye pile, forming 120 degree angles with face sheets on both sides. The sheet piles in the tailwall are also connected with each other through the three-point interlock connection.

6.1.1.1 Dry Barge Berth

Figures 6.1-4 to 6.1-7 show the layout and typical sections of the Dry Barge Berth and the Wet Barge Berth, together covered in the Barge Berth Phase 2 As-Built Drawings plan set (PND, 2008a). The typical face sheets for the Dry Barge Berth extend from +36 feet MLLW at the top to -10 feet MLLW at the bottom for a total length of 46 feet. The wall height is 26 feet as measured from the top of sheet to the mudline at +10 feet MLLW. The tailwall length is 33 feet with a PS31 end anchor pile oriented 90 degrees attached to the end of the tailwall.

At the southern end of the Dry Barge Berth (Figure 6.1-6, Section 13A-13A), where it connects to the Wet Barge Berth, sheet piles extend further down to –40 feet MLLW for a total sheet length of 76 feet . The wall height is 61 feet as measured from the top of sheet to the future harbor dredge elevation at -25 feet MLLW as shown in Figure 6.1-6. The tailwall length for this section is 104 feet. These tailwalls have both end and intermediate anchors.

As shown in Figure 6.1-4, the typical maximum penetration of the facewall and tailwall sheet piles into the BCF clay layer is approximately 5 feet. As shown in Figure 6.1-6, the penetration into the BCF clay is significantly greater in Section 13A-13A at approximately 25 feet. The deeper penetration is to provide for future dredging.

Fender piles battered at an angle of 12:1 (vertical:horizontal) were installed on the seaward side of the bulkhead structure. A cap consisting of HP14×89 section and 14-inch diameter×1/2-inch wall concrete-filled steel pipe connects the top of the sheet piles in the Dry Barge Berth area.

6.1.1.2 Wet Barge Berth

Figure 6.1-7 shows that the typical face sheets extend from +30 feet MLLW down to -40 feet MLLW for a total sheet length of 70 feet. The wall height is measured from the top of the concrete utilidor at +38 feet MLLW to future dredge depth of -25 feet MLLW for a total height of 63 feet. Additionally, there is an allowance for up to 6 feet of dredging, as discussed in Section 2.1.5, in front of the OCSP® so the wall height used for analysis is 69 feet. The tailwall length in this section is approximately 104 feet. The tailwall consists of two segments of equal length, separated by an intermediate anchor. An end anchor is also present at the end of the tailwall. Both the intermediate anchor and end anchor consist of a single PS31 sheet pile oriented 90 degrees to the rest of the tailwall sheets.

Typical maximum penetration of facewall and tailwall sheets into the BCF clay layer is approximately 25 feet. The utilidor had not been constructed as of September 2012. Fender piles were also designed for the Wet Barge Berth area but have not been constructed.

6.1.1.3 North Extension 1 and 2

Figures 6.1-9 and 6.1-10 present the layout and typical sections of the North Extension 1 and 2 areas. The typical sheet pile lengths for the North Extension 1 (Section F-F) and North Extension 2 (Section G-G) areas are approximately 80 feet and 90 feet, respectively. Wall heights are measured as follows:

- **North Extension 1.** The wall height is measured from the top of the concrete utilidor at +38 feet MLLW to the future dredge depth of -35 feet MLLW for a total height of 73 feet. Additionally, there is an allowance for up to 6 feet of dredging, as discussed in Section 2.1.5, in front of the OCSP® so the wall height used for analysis is 79 feet. The tailwall length is approximately 144 feet. The typical maximum penetration of the facewall and tailwall sheets into the BCF is approximately 10 feet and 24 feet, respectively. Less penetration of the face sheets in the BCF clay occurs because of a sub-trench dredged along the facewall as shown in Figure 6.1-9.
- **North Extension 2.** The wall height is measured from the top of the concrete utilidor at +38 feet MLLW to the future dredge depth of -45 feet MLLW for a total height of 83 feet. Additionally, there is an allowance for up

to 6 feet of dredging, as discussed in Section 2.1.5, in front of the OCSP® so the wall height used for analysis is 89 feet. The tailwall length is approximately 183 feet. Typical maximum penetration of the facewall and tailwall sheet piles into the BCF clay layer is approximately 10 feet. Figure 6.1-10 shows the sub-trench beneath the face sheets and change in tailwall embedment landside of the facewall.

Tailwalls in the North Extension 1 and 2 areas also consist of two segments separated by an intermediate anchor and terminated with an end anchor. A utilidor and fender piles similar to those at the Wet Barge Berth were also planned but have not been constructed.

6.1.2 Deviations of the As-Built Condition from the Original Design

Based on the review of available design and construction documents for the North Expansion projects, some significant deviations of the as-built condition from the conditions assumed in the original design were identified. The identified deviations of the as-built condition from the original design fall within four main categories:

- Damaged sheets and disengaged interlocks
- Inadequate sheet pile embedment
- Backfill characteristics
- Groundwater and tidal water assumptions

The differences in as-built backfill characteristics and groundwater and tidal water assumptions are discussed in Section 5. The detailed discussions of the differences in the remaining categories are presented below.

6.1.2.1 Damaged Sheets and Disengaged Interlocks

In October 2009 and July 2010, dredge work was performed at the face of the partially completed North Extension bulkhead. Shortly after the dredge work, sinkholes started to develop on the land side of the bulkhead, indicating that loss of material had occurred and leading to suspicion that some of the face sheets might be damaged and holes had opened up on the facewall of the bulkhead. A systematic examination of the face sheets as well as tailwall sheets of the OCSP® bulkhead was carried out in order to determine the extent of the damage. The scope of this examination included excavating selected cells and subsequently pulling and inspecting sheet piles from these cells, both at the face of the cell and at the tailwall. The scope also included underwater inspection of the face sheets of the remaining cells. The examination work started in the North Extension - and south end of the Wet Barge Berth area and eventually extended further north to include all the cells south of the Cell 25 of the Wet Barge Berth. The findings of this examination were summarized in the Preliminary Inspection Summary Report (ICRC, 2010; Item H2 in Appendix H) and Dive Inspection Report (PND, 2010d; Item H2 in Appendix H).

As of October 2011, 24 cells within the North Extension and Wet Barge Berth area were examined, including Wet Barge Berth cells 25-33, 36-38, and North Extension cells 9-13, 31-32, 37-41 (see Figure 6.1-11 for cell locations). A total of 1,858 sheet piles were pulled and inspected. Among all the piles that were pulled, 627 piles, or approximately 34 percent, were found to be damaged. Damage was found in most cells that were excavated and was observed on both the face sheets and the tailwall sheets. The damage observed in the pulled sheet piles included damaged sheet pile tips, pinched sheet piles, bent sheet piles, and disengaged interlocks, as shown in Figures 6.1-12 through 6.1-19. Most of the damage was observed near the tip of the piling.

In additional to excavating selected cells, an underwater dive inspection was also performed in order to identify possible damaged piling within the cells that remain in place. The inspection consisted of a diver cleaning a section of the face sheet pile, wye pile, and interlocks manually or with a pressure washer. The diver then performed tactile and clear water bag visual inspections of all or a portion of the interlocks from mudline to as high as the diver could reach. For this inspection, the seabed in front of the bulkhead face was dredged to approximately 10 feet above the sheet pile tip elevation. In the Wet Barge Berth area, damage was found to be present at every cell from cells 27 to 38 at the face sheets or wye piles. In the North Extension area, damage was found at multiple cells throughout the area between cells 41 and 66. No damage was observed at North Extension cells 13 to 30. Most of the damage found was associated with disengaged interlock between adjacent piles. But pile tip damage was also observed in some areas.

The exact cause of the damage for each pile is uncertain, but is suspected to be associated with unbalanced soil loading during driving, hard driving during pile installation, and the presence of large rock in some areas of sheet pile installation. The exact sources of the large rocks are unclear, but it is suspected to be a result of riprap placed in previous phases of the PIEP. The extent of the damage was, however, severe enough that the structural integrity of the OCSP® system was a concern, and this concern eventually led to CH2M HILL's suitability study. A more detailed discussion of the construction issues is presented in Section 8.

In addition to the damages discussed above, a separate incident was documented where a number of tailwall sheets of Cell 32 in the North Extension area were bent inwards by water pressure and ice in front of the cell. The damage occurred when the cell was only partially backfilled. ICRC and PND concluded that the damage was the result of high wind coupled with heavy ice floes and removal of the driving templates prior to backfilling of the cell (ICRC, 2011). PND also concluded that the damage to the face sheets will not affect the structural integrity of the completed cell once the cell is properly backfilled. From available documents and pictures, the damage to the steel sheet piles appears to be relatively minor and there is also no evidence of broken interlock.

6.1.2.2 Inadequate Sheet Pile Embedment

Another notable deviation of the as-built condition from the original design is inadequate sheet pile embedment, or sheet piling not driven to their specified tip elevation. This deviation was documented in various construction submittals and RFIs and occurred at multiple cells. The difference between the actual tip elevation and the specified tip elevation is on the order of several inches to several feet. Based on available construction documents, most of the large variations (exceeding 1 foot) appeared to have occurred in the tailwalls.

6.1.3 Critical Elements of Design and Construction

The OCSP® bulkhead is a variation of the modified diaphragm cofferdam (U.S. Steel, 1975 and 1984). Structurally speaking, OCSP® bulkhead is a tension structure in the sense that it relies on the horizontal hoop tension developed in the arched cell facewall to retain soil at the front face; the tailwall and anchor pile together act as anchors to provide pullout resistance needed to prevent excessive horizontal deformation of the facewall. Interlock tension is also relied on to transfer the force from the facewall sheets to the wye pile and throughout the tailwall piles. Loss of any of these components can result in failure of one or more cells in the system. Section 6.1.5 provides additional discussion of the effects of damaged sheets or disengaged interlocks.

The combined system of facewall sheets, tailwalls, and retained soil contribute to the successful performance of the OCSP® system. Figure 1.2-4a illustrates the principles of the OCSP® system. The following elements determine the performance of the OCSP® system:

- **Interlock integrity.** Because the structural integrity of the OCSP® bulkhead relies on properly developing the interlock tension between the individual sheet piles, it is of paramount concern that the three-point "thumb and finger" interlock connectors between the adjacent sheet piles are properly aligned. Steel templates are usually used to maintain the relative position of the sheet piles during construction. During installation, a sheet pile is picked up by crane and threaded onto the thumb-and-finger connector of an adjacent sheet pile or wye connector (Figures 6.1-20 and 6.1-21). The full engagement of the interlock must be carefully preserved throughout the installation process.

- **Tailwall length and depth.** The length and depth of the tailwall is critical to the internal stability of the cell. The anchor force, or pullout resistance, of the tailwall must exceed the driving force from lateral earth pressure and water pressure for the cell to be stable. The pullout resistance is directly related to the effective area of the tailwall behind the active failure wedge; see Section 5.2.3 for a detailed discussion of methods used to estimate the anchor force. Any reduction in the effective tailwall area will reduce the factor of safety for pullout resistance.

- **Backfill properties.** The characteristics of the backfill also have an effect on the performance of a tailwall because the pullout resistance of the tailwall relies on the sheet pile-soil friction provided by the backfill. Improper backfill material with lower friction angles and inadequate compaction could result in lower pullout factors of safety than their designed values (refer to sensitivity analysis in Section 5.3). Inadequate compaction will also result in surface settlement, pavement failures, and additional load on some structural elements (such as the crane rail beams).
- **Groundwater and tidal elevations.** Groundwater and tidal water have a large impact on the performance of the OCSP® system, as discussed in Section 5. Although not a construction issue, this design parameter must be thoroughly characterized in order to establish a parameter value that can be used in design to ensure satisfactory performance.

In the absence of redundancy within the OCSP® system, it is critical to assure that interlocks and sheets have been installed correctly. Although observations can be made during installation and hard driving conditions or misalignments recorded, installed sheet pile is very difficult except for the exposed portion of the face sheets, inspection of the. There are no direct or indirect methods for checking the integrity of interlocks in places where the interlocks are not visible. For this reason, 100 percent confidence in the OCSP® system installation is not possible, and some risk will always be present. This issue is further discussed in Section 8.3.2.8. Pile Inspection Risk.

6.1.4 Baseline As-Built Performance

In order to establish a baseline for the deviation analysis, the structural performance of the as-built OCSP® bulkhead was evaluated. The controlling wall section at Section G-G near the south end of the North Extension (see Figures 6.1-8 and 6.1-10) was selected for the analysis. The bulkhead layout and geometry were based on as-built plans. The geotechnical parameters were based on CH2M HILL's interpretation of the site and soil conditions. Because the impact of backfill characteristics and groundwater and tidal water assumptions have already been evaluated in the sensitivity analysis presented in Section 5.3, the actual backfill characteristics and current groundwater and tidal elevation assumptions rather than the original design values were used in the this baseline analysis. This approach helps to isolate the effects of local defects such as damaged sheet piles and disengaged interlocks.

The detailed discussion of the methods used for the analysis is presented in Section 5.2.3. Table 6.1-1 shows the internal stability factors of safety (FS) for the baseline condition. The post-earthquake load case for interlock tension is required to be the same as the pre-earthquake undrained long-term static case to assure that no loss in interlock strength occurs as a result of earthquake loading. When compared with the required FS values, the baseline as-built condition has sufficient FS for all load cases except for the Interlock Tension in the Long-term Static Undrained and Post-Earthquake cases. For the Long-term Static Undrained and Post-Earthquake cases, the as-built condition showed marginal factors of safety for interlock tension (FS = 1.9, vs. 2.0 required).

TABLE 6.1-1
Baseline As-Built Condition – Internal Stability Factor of Safety

	Loading Conditions						
Failure Mode	**End of Construction**	**Long-term Static (Drained)**	**Long-term Static (Undrained)**	**OLE**	**CLE**	**MCE**	**Post-Earthquake**
Interlock Tension	1.9	2.7	1.9	1.9	1.6	1.4	1.9
Required FS	1.5	2.0	2.0	1.5	1.3	1.1	2.0
Pullout of Tailwall	3.1	4.2	3.0	3.2	2.6	2.1	3.1
Required FS	1.3	1.5[a]	1.5[a]	1.3	1.1	1.0	1.3

[a] See Section 6.5.1.3 for a discussion of load redistribution in the event of a tailwall failure. The load distribution allows use of FS = 1.5 rather than FS = 2 if no load redistribution were to occur.
CLE = contingency level earthquake
MCE = maximum considered earthquake
OLE = operating level earthquake

6.1.5 Deviation Analysis

Deviation analysis was performed in order to evaluate the performance of the OCSP® system subjected to the construction defects discussed in Section 6.1.2. The analysis involves numerical modeling with the computer program FLAC^{3D}as well as investigations using more conventional structural analysis methods. The focus of the analysis in this section was on the effects of damaged sheets/disengaged interlocks and insufficient sheet pile embedment. The effects of backfill characteristics and ground/tidal water assumptions are discussed in Section 5.

6.1.5.1 Approach to Defect Modeling

Local defects in the sheet piles comprising OCSP® bulkhead, such as damaged sheet piles and disengaged interlocks, effectively represent a weakened or broken link in an otherwise continuous tension structure because the transfer of horizontal hoop tension is disrupted around the area of the defect. The disruption will result in higher stresses and interlock tension in the area adjacent to the defect due to stress redistribution.

Of particular concern is the impact when a partially disengaged interlock propagates vertically, or “unzips,” through the whole length of the sheet pile due to this stress redistribution. If this unzipping behavior occurs on one of the face sheets, hoop tension across the whole arched face of the affected cell will be lost. Backfill material will spill out from between the sheet piling in the cell face, local ground subsidence will develop, and the cell will lose its ability to support operational load. Because adjacent cells share the wye connector, the loss of hoop tension in one cell will result in unbalanced loading condition at the wyes shared with the adjacent cells and may cause them to fail. As a result, progressive collapse of additional cells is possible. This failure mechanism is somewhat mitigated by the planned reinforced concrete utilidor which will rigidly connect to the tops of all the face sheets, wyes, and some of the tailwall sheets. The high strength and rigidity of the utilidor may limit the failure to a relatively small area of a few cells.

The propagation of interlock failure is a highly nonlinear phenomenon involving material yielding and large strain deformation at the three-point interlock. The problem is made more complex by the soil-structure interaction between the sheet pile and the backfill. As interlock failure propagates above the mudline, backfill will spill out from inside the cell through the opening created. The driving force created by soil pressure on the back of the cell face will decrease as a result. At the same time, the resistance will also decrease as the available intact interlock length is reduced. These two competing factors will determine whether the complete failure of the cell would occur. It is extremely difficult to fully account for all the nonlinear behaviors discussed above in a rigorous analysis.

As a result, a series of simplified numerical models incorporating nonlinear soil behavior and elastic structural behavior were developed using software package FLAC^{3D}. Although the software does not account for all the real-world, nonlinear mechanisms, it is expected these models would provide reasonable predictions of the potential impact of local defects and possibility of interlock loss propagation. These analyses were conducted for static earth and water loading only; seismic loads were not explicitly modeled. Differential water levels were considered in the analyses.

Detailed discussions of the development of the numerical models and analysis assumptions are presented in Section 7.5.2. For all the numerical models, the model geometry is similar to the critical section G-G discussed in Section 6.1.4. A brief discussion of the important results is given below.

6.1.5.2 Impact of Face Sheet Defects

As the results in Section 7.5.2 indicate, the highest horizontal hoop tensile stress in the face sheet tends to occur just above the mudline elevation and tensile stress below the mudline is small. As shown in the results for Face Sheet Defect Model 1 on the left hand side of Figure 6.1-22, a 10-foot-long defect was introduced at the tip of one of the face sheets by removing a portion of the structural elements in the model representing the sheet pile. The defect is completely buried under the final mudline.

Different colors are used to indicate different construction stages, with backfill gradually raised to elevation +30 feet MLLW, landside water level (LWL) gradually raised to +20 feet MLLW, and seaside mudline (ML) gradually dredged to elevation -48.4 feet MLLW. [Note: elevation -48.4 MLLW resulted from the element size used in the FLAC^{3D} modeling and differs from the planned dredge elevation. For this evaluation the slight difference in elevation will not affect conclusions.] The different line styles are used to distinguish the different locations where the horizontal stress is measured: at the location where the defect was introduced (centerline of the cell), at the centerline of the adjacent cell, and at the centerline of the cell one cell away. As shown in the figure, the horizontal hoop tensile stress at all three locations showed very little change from the baseline “2h” model, which does not have a defect. The results suggest that a small, localized damage at the pile tip is not a source of concern if it is buried under the mudline/future dredge line.

The right-hand side of Figure 6.1-22 shows the situation where a larger defect is introduced in the model (Face Sheet Defect Model 2). The defect extends from the tip of the sheet pile to approximately elevation -30 feet MLLW with a total length of 31 feet, or about one third of the total sheet pile length. As shown in the figure, the horizontal stress above elevation -30 feet MLLW stayed relatively unchanged from the baseline model throughout the analysis stages. Below elevation -30 feet MLLW, the horizontal stress at the defect drops to almost zero, as expected. As discussed in Section 7.5.2 and Appendix G, the LWL and the seaside water level (SWL) are only equal in the first stages of the model. The legend of Figure 6.1-22 gives the LWL at the model stages depicted in the figure. The SWL remains constant at elevation 0 feet MLLW throughout the model’s construction stages. The delta illustrated in the figure for the model step corresponding to raising the backfill from elevation +26.9 to +30 and raising the LWL from elevation 0 to 20 feet MLLW suggests that the differential water load has a significant effect on wall movements as one would expect. This model result confirms that lowering the SWL and increasing the differential water load on the wall facing would increase the hoop stress in the facewall. However, the model may tend to overpredict this stress increase as the differential water load in the model is an imposed load (pore pressure) condition and not the result of a flow calculation with consideration of the defect.

No significant stress concentration was observed near the defect, as was confirmed by the stress contour plot in Figure 6.1-23. The horizontal stress at the centerline of the adjacent cells remained relatively unchanged from the baseline model until the mudline was lowered to elevation -30 feet MLLW. After the mudline was lowered below -30 feet MLLW, the horizontal stress of the adjacent cells start to fall below the baseline values. This phenomenon can be attributed to the fact that, once the mudline is lowered below -30 feet MLLW, the backfill material would be prone to loss through the opening on the face sheets. The loss of backfill material in the cell will relieve the earth pressure on the face sheets and consequentially the horizontal tensile stress.

Due to the apparent lack of stress concentration, upward propagation of the defect seems unlikely, and the overall integrity of the cellular structure is likely to be preserved. However, ground subsidence behind the cell with the defect will likely develop. The documented sinkholes that developed as a result of dredging in front of the wall during inspections seems to confirm this conclusion. Pictures of these sinkholes are included in Appendix H, Item H2.

6.1.5.3 Impact of Tailwall Defects

Numerical modeling results in Section 7.5.2 show that the magnitude of horizontal tensile stress in the tailwall closely follows the hoop tensile stress of face sheets. At the wye connection, the horizontal tensile stress in the tailwall equals the hoop tensile stress of the face sheet at the same elevation. The horizontal tensile stress in the tailwall gradually decreases at the piles farther away from the wye connection as load is gradually transferred to surrounding soil.

Several different numerical models were created to evaluate the impact of damaged sheet pile or broken interlock in the tailwall on the structural integrity of the OCSP® bulkhead. The first model, presented in Figure 6.1-24, introduced a defect extending from the bottom of the tailwall (at approximate elevation -51 feet MLLW) to approximately elevation -40 feet MLLW, or 11 feet in length. The defect is located relatively close the wye connection. The distance from the defect to the wye is approximately 13 feet. As shown in the figure, the horizontal stress at elevation -40 feet MLLW in the tailwall shows significant increase around the defect. The horizontal stress at the end of the analysis stage increased from approximately 10 ksi in the baseline model (no defect) to approximately 25 ksi (12.5 kips/inch). This peak stress level is lower than the specified interlock strength of 20 kips/inch, indicating interlock failure propagation is unlikely. The horizontal stress at elevation -29.5 feet MLLW, which is about 10 feet above the defect, shows a smaller but still noticeable increase around the defect location. The horizontal hoop stress in the face sheets also shows a pronounced peak at elevation -40 feet MLLW, further confirming the large stress concentration around the defect. These results are explained in more detail in Section 7.5.2.

In the second model, the defect was extended to approximately elevation -30 feet MLLW. This translates to a defect size of approximately 21 feet. As shown in Figure 6.1-25, the tailwall horizontal tensile stress just above the defect again shows significant increase in the region around the defect, increasing to approximately 54 ksi (27 kips/inch) at the peak. This level of tensile stress exceeds the manufacturer's guaranteed interlock strength for the sheet piles, as well as the minimum yield strength of the steel material. The interlock connection would likely yield under such stress, suggesting the broken interlock would propagate upward. As the broken interlock propagates upward, the remaining intact portion of the tailwall interlock picks up all the horizontal loads that used to be transferred through the whole height of the tailwall. The stress concentration, clearly shown in the stress contour in Figure 6.1-26, would likely become even more severe, thus causing the "unzipping" of the whole tailwall. Eventually the interlock along the whole height of the tailwall would be completely broken as a result.

In the third model, the defect was moved farther away from the wye. The defect is now located approximately 42 feet MLLW from the wye and still extends from the bottom of the tailwall to approximately elevation -30 feet MLLW. As shown in Figure 6.1-27, the stress concentration just above the defect is still present. Due to the overall lower stress level at locations farther away from the wye, the magnitude of peak stress concentration drops to approximately 49 ksi (24.5 kips/inch). This stress value is still significantly higher than the specified interlock strength of 20 kips/inch, suggesting "unzipping" is likely to occur.

When the complete loss of interlock occurs at a tailwall, especially on a tailwall pile close to the wye connector, most, if not all of the tailwall resistance would be lost and the face sheets connected to the failed tailwall will experience unacceptable deformation. Because the tailwall is shared by two adjacent cells, complete or partial failure of the two cells next to the failed tailwall is likely. Whether or not progressive failure of the rest of the cells will occur is hard to predict. Again, the potential for progressive failure is somewhat mitigated by the planned utilidor which, because of its stiffness, would serve as a lateral "load redistributor" and distribute the driving force from the failed cells to neighboring cells.

As a conservative estimate, assuming the utilidor will redistribute the driving force to four nearby tailwalls, and also assuming the pullout resistance of the two tailwalls immediately adjacent to the failed tailwall would be reduced by 50 percent due to loss of backfill from the failed cells, the pullout factor of safety for the Long-term Static (Drained) case will be reduced from 4.2 to approximately 2.5. Factors of safety for other load cases for such a scenario are shown in Table 6.1-2. Compared with the allowable values in Table 2.2-2, the overall system appears to retain sufficient factor of safety for pullout resistance and the failure is expected to limited to the localized area around the failed tailwall.

TABLE 6.1-2
Tailwall Pullout Factors of Safety Due to Complete Loss of a Single Tailwall

	Loading Conditions						
Failure Mode	**End of Construction**	**Long-term Static (Drained)**	**Long-term Static (Undrained)**	**OLE**	**CLE**	**MCE**	**Post-Earthquake**
Pullout of Tailwall	1.9	2.5	1.8	1.9	1.6	1.3	1.9
Allowable	1.3	1.5	1.5	1.3	1.1	1.0	1.3

CLE = contingency level earthquake
MCE = maximum considered earthquake
OLE = operating level earthquake

Because of the difficulty associated with inspecting the tailwalls, it is unclear what kind of damage is present in the remaining OCSP® bulkhead constructed at the North Expansion project to date. While complete tailwall failure has not been observed on the PIEP, it should be noted that dredging has not been completed along the whole length of the wall According to available construction documents, dredging to -45 feet MLLW has been completed at the critical section, with pockets of dredging down to elevation -50 feet MLLW completed to facilitate inspection. Future harbor dredging would allow an overdredging depth of -51 feet MLLW, which will significantly increase the driving force on the OCSP® cells. Additional dead load from 3 feet of surfacing, operational live load and earthquakes will also create further loads on the system.

6.1.5.4 Impact of Insufficient Pile Embedment

Tailwall sheet piles not driven to the specified tip elevation will result in reduced tailwall pullout resistance from their design values. As explained in Section 6.1.3, the tailwall pullout resistance is a function of the effective tailwall area and shortened piles will reduce this effective area. Based on available construction documents, the amount of reduction in effective area in any given tailwall is less than 10 percent of the total effective area.

Accounting for the higher wall-soil resistance at the bottom of tailwall where the reduction of area occurs, the tailwall pullout resistance decreases by a maximum of approximately 15 percent. Given the relatively high factors of safety for tailwall pullout, shown in Table 6.1-1, the impact of reduction in tailwall resistance is small for this condition and not likely to affect the structural integrity of the bulkhead.

6.2 As-Built Life-Cycle Performance

Port structures are exposed to a variety of corrosive conditions that can adversely affect long-term performance. Corrosion reduces the wall thickness of metal components and, subsequently, reduces the structural capacity of these components. Corrosion rates of steel in marine structures vary based on geographical location and the exposure location (“zone”). The long-term performance of a marine structure is closely tied to the integrity of its major structural components. The following subsections provide a discussion of the life-cycle performance of the OCSP® system for the Port of Anchorage.

6.2.1 OCSP® Corrosion Protection System

Structural components at the PIEP will be exposed to the atmosphere, marine splash, cyclical immersion in seawater (tides), full immersion in seawater, mud, and soil.

As part of the North Expansion, a corrosion protection system was designed to extend the service life of the structures. This system consists of galvanized (zinc-coated) structural steel components and an impressed current cathodic protection system. The galvanized steel components were included in the contract documents and have been installed. The intent of the design was to install the impressed current cathodic protection system within one year of structure installation, but the system has not yet been installed.

The effectiveness of the corrosion protection system, the rate of corrosion, and the metal thickness are the primary factors used to evaluate life-cycle performance.

6.2.2 Summary of Exposure Conditions

The PIEP is located on Cook Inlet. Marine structures installed in Cook Inlet are exposed to a combination of mechanically aggressive and corrosive conditions, which result in the potential for rapid metal deterioration unless protected from corrosion. Corrosion protection systems for marine structures must be effective across a series of zones on the structure. These types of structures are typically divided into five distinct exposure zones—atmospheric, splash, tidal, submerged, and soil (or mud):

- **Atmospheric zone (seaside).** This zone is at the top of the structure (above approximately elevation +35 feet MLLW at POA) and is an area that is exposed only to atmospheric conditions. This portion of the structure is usually exposed to mist and relatively high humidity, but is not frequently wetted by splash from waves.

- **Splash zone (seaside).** The splash zone is the area above mean high tide that is subject to frequent wetting by wave and wind action (between elevation +29 feet MLLW and elevation +35 feet MLLW). The continual wetting action and high oxygen availability creates the potential for very corrosive conditions, especially near the bottom of the splash zone that is most frequently wetted. In some geographical locations, the highest corrosion rates on a marine structure have been observed in this zone. However, this does not appear to be the case in Cook Inlet (and other Alaska marine communities). In Alaska, the corrosion rates in this zone have been found to be similar to those in the atmospheric zone. The lower corrosion rates reported for this zone in Alaska marine environments appear to be due to primarily to the large tidal fluctuation range, which significantly reduces the time each day when the splash zone exists.
- **Tidal zone (seaside).** The portion of the structure between the mean high tide (elevation +29 feet MLLW) and the mean low tide (elevation -5 feet MLLW) is exposed to the tidal zone. In this zone, the structure is cyclically submerged in seawater and exposed to the atmosphere. Structural components in this zone are also subject to potential abrasion from ice in the winter months. A rapid form of corrosion termed accelerated low water corrosion (ALWC) is present on existing steel pipe piling at the POA in the tidal zone from approximately 1.5 feet below mean low tide to lowest astronomical tide. ALWC appears to be microbially influenced and can cause rapid metal loss.
- **Submerged zone (seaside).** The submerged zone is the area below extreme low tide (elevation -6.4 feet MLLW) to the mud line, which varies.
- **Mud or soil zone (seaside and landside).** The portion of the structure driven into the soil is exposed to the "mud" zone. The structure backfilled with soil is considered to have similar corrosion protection requirements as the mud zone.

6.2.3 Overview of Corrosion Protection System

A corrosion protection system was developed, designed, and included in the construction documents for the PIEP. This corrosion protection system is described below.

6.2.3.1 Galvanizing

All steel structural components of the bulkhead structure were specified to be galvanized in accordance with ASTM A123 or ASTM 153 with a minimum zinc thickness of 6-12 mils. The background for the coating thickness range is noted in Section 8.3.2.11. Galvanizing was selected for the protective coating due to its inherent toughness, relative ease of application, and cost. The contract documents include provisions for repairing small localized damaged galvanizing with heat-applied "galvanizing sticks." All sheet piles longer than 70 feet require splicing by field welding. Thermal spray coatings were specified for welded splices. Thermal spray coatings were specified to be 20-mil metal alloy (85 percent zinc, 15 percent aluminum) with a penetrating sealer. Surface preparation and quality control procedures for thermal spray coatings were included in the contract documents.

Galvanizing is the sole corrosion protection element for the steel structure exposed to the atmospheric and splash zones. Galvanizing provides corrosion protection for the tidal, submerged, and mud zones as well, although it will deteriorate at faster rates in these zones unless it is placed under cathodic protection. Galvanizing also protects small defects in the coating where bare metal is exposed (a form of cathodic protection).

Galvanizing is a zinc coating. The zinc has some inherent corrosion-resistant properties, but it is a metal and it will deteriorate by corrosion; that is, it has a finite service life. The service life of galvanizing is a function of its thickness and the corrosivity of the environment. As the zinc deteriorates and the steel surface is exposed, the rate of galvanizing deterioration will increase. Once the galvanizing is consumed, the steel will begin to corrode unless a replacement coating is applied (for the atmospheric and splash zones) or cathodic protection is installed and maintained (for the tidal, submerged, and mud/soil exposure zones).

6.2.3.2 Cathodic Protection

An impressed current cathodic protection system was designed to protect structural components that are submerged or in contact with soil. If a portion of the structural component is not in contact with a bulk electrolyte (like soil or water), cathodic protection will not be functional; cathodic protection provides no benefit to metals exposed in the splash and atmospheric zones.

The impressed current cathodic protection system was designed to provide protection to metal surfaces exposed on the seaside and the landside. To achieve this goal, cathodic protection anodes are installed on the seaward side of piling in Cook Inlet for protection of seaside surfaces, and additional anodes are installed in drilled holes landside to protect surfaces exposed to soil and mud. This arrangement of the cathodic protection system components provides more uniform distribution of cathodic protection current and a level of redundancy. The seaside cathodic protection system includes pile-mounted anodes, air-cooled rectifiers, anode terminal boxes, and wiring. The landside cathodic protection system consists of deep anodes, air-cooled rectifiers, anode terminal boxes, and connecting wiring. Provisions were included for electrical bonding of piles to ensure electrical continuity (all metal components must be electrically bonded into the cathodic protection system). Monitoring test wells were included to allow testing and adjustment of the landside cathodic protection system.

This impressed current cathodic protection system will be maintenance-intensive. The seaside cathodic protection components will be exposed to mechanical damage from debris and ice. The landside cathodic protection system components will be less susceptible to mechanical damage, but there may be other issues associated with the long-term performance of this installation. These issues are discussed below in Section 6.2.5.

The intent of the corrosion protection system design was to install a cathodic protection soon after the structure was completed (within one year). Timely installation of the cathodic protection system would have two benefits. One, the system would help extend the life of the galvanizing in the tidal zone. Second, the cathodic protection current required to protect zinc is expected to be lower than the current required to protect bare steel. This could translate into lower power costs over the life of the structure.

6.2.4 Life-Cycle Considerations

The life-cycle performance of this structure will depend on several items. These include:

- The original thickness of the galvanizing.
- Corrosion rates of galvanizing above and below the water line.
- Corrosion rates of steel with cathodic protection (submerged or soil contact) and without cathodic protection (atmospheric or splash zones).
- Corrosion of the galvanizing and steel before the cathodic protection system is installed and after the 40-year design life of the cathodic protection system (the service life used in the life-cycle cost analysis). Note the design life of the structure is 50 years.
- The effectiveness of the cathodic protection system over time.
- Continual operation and maintenance of the cathodic protection system.

6.2.5 Metal Thickness Reduction

The capacity of the structural components associated with marine terminal construction will decrease with time as the metal thickness is reduced due to corrosion. Corrosion rates for zinc galvanizing and carbon steel for the various zones of corrosion were developed to allow calculation of metal loss.

Deterioration rates of galvanizing reported in the literature vary significantly with geographical area and exposure conditions. Atmospheric corrosion rates are generally low, but corrosion rates of galvanizing exposed to seawater are reported to range from 0.2 to 3.5 mils per year. Corrosion rates of galvanizing in contact with soil are reported to range from less than 0.1 mil per year to 0.6 mil per year. Observations of galvanized structural components at marine terminals in Alaska indicated little or no loss of galvanizing occurred above mean tide (Nottingham et al., 1983). Observations and tests also indicated that 25-mil galvanizing below mean tide would provide approximately 10 to 20 years of service. Based on these observations and tests, the corrosion rate of submerged galvanizing in the Alaska marine environment would be in the order of 1.25 to 2.5 mils per year.

Coffman (2007a) estimated life-cycle costs for the facility based on a galvanizing service life greater than 10 years for atmospheric, less than 10 years for buried, and approximately 5 years for submerged exposure conditions. Based on a minimum zinc thickness of 6 mils required for this project, the corrosion rates based on these service life estimates would be less than 0.6, greater than 0.6, and approximately 1.2 mils per year for atmospheric, buried (mud), and submerged, respectively. For the purposes of this analysis, comparable corrosion rates for galvanizing were assumed. A summary of the corrosion rate data for galvanizing, including the values that were selected for evaluating zinc performance, is shown in Table 6.2-1.

The corrosion rate of carbon steel in marine environments was also established. General corrosion and localized corrosion rates were developed based on a literature review. Two documents specific to marine structures in the Alaska environment were also reviewed (Nottingham et al., 1983; PND, 2006e). The results of this review are summarized in Table 6.2-2.

The corrosion rates summarized in Tables 6.2-1 and 6.2-2 were used to estimate metal loss reduction over time.

TABLE 6.2-1
Corrosion Rates of Galvanizing in Marine Zones

	Corrosion Rate of Galvanizing (mils per year [mpy])		
Source	**Atmospheric**	**Submerged**	**Soil**
Literature Review	Nil	0.2 to 3.5	<0.1 to 0.6
Nottingham et al. (1983)	Nil	1.25 to 2.5	Not Reported
Coffman Engineers (2007b)	<0.4	0.8	>0.4
CH2M HILL Recommended	0.25	1.0	0.4

TABLE 6.2-2
Corrosion Rates of Steel in Marine Exposure

	Corrosion Rate of Carbon Steel (mpy)				
Source	**Atmospheric**	**Splash**	**Tidal**	**Submerged**	**Mud/Soil**
Without Cathodic Protection					
Literature Review	1-3	3 to 5	5 to 10 (40[a])	4 to 6 (40[a])	1 to 2
Nottingham et al. (1983)[b]	2	2 to 2.5	3 to 5	2 to 6	1
Coffman Engineers (2007b)	4	4	4 to 6 (60[c])	4 to 6 (60[c])	2 to 4
With Cathodic Protection, Combined Sources					
Carbon Steel	4	4	3	2	0.5
Galvanized Steel	0.25	0.25	1.0	0.5	0.5

[a] High rates of pitting associated with ALWC.

[b] Steel corrosion rates as determined from metal loss measured on piling at Cordova, Alaska.

[c] Highest rate of pitting anticipated for marine structures located in Cook Inlet.

These metal loss estimates were based on the following assumptions:

- Cathodic protection will be applied within 5 years of installation (the existing facility is already 3 years old).
- Corrosion of the steel components will begin when the galvanizing has been consumed. In the case of submerged surfaces, the galvanizing may be consumed before the cathodic protection system is installed.
- The galvanizing is not replaced in the atmospheric or tidal zones once it has deteriorated.
- A steel corrosion rate of 3 and 2 mils per year, with cathodic protection, for the tidal and submerged zones, respectively. This corrosion rate is intended to account for: a) the possibility of lower levels of corrosion protection provided by the cathodic protection system in the tidal zone, which is not always immersed; b) the possible effects of scour/abrasion in the submerged zone; and c) intermittent, short-duration loss of seaside cathodic protection system components.
- A steel corrosion rate of 0.5 mil per year, with cathodic protection, for surfaces of the piling in contact with soil. This corrosion rate is intended to account for intermittent, short-duration loss of landside cathodic protection system components.
- Linear corrosion rates.

Two sheet piling sections for the OCSP® system are being used for the PIEP, PS27.5 and PS31. The PS27.5 piling has a nominal web thickness of 0.4 inch and is being used primarily for the tailwalls. The PS31 piling is 0.5 inch thick and is being used primarily on the facewall sheets. Remaining wall thickness at the end of 50 years was estimated for both types of piling. The remaining wall thickness was estimated assuming corrosion occurs on both sides of the pile. Corrosion rates for each exposure zone (atmospheric, splash, tidal, submerged, and soil) were used depending on the side (soil or water) and vertical location on the sheet piling.

The reduction in wall thickness was based on the assumptions itemized above for a 40- and 45-year cathodic protection service life. The original life-cycle cost analysis assumed a 40-year cathodic protection system design life for the marine terminal. The remaining wall thicknesses were calculated for these two service periods to allow evaluation of the potential cost impacts associated with an additional 5 years of cathodic protection operation and maintenance (discussed in Section 6.2.5). The results of the calculations are summarized in Table 6.2-3 and Table 6.2-4.

Note that the reduction in the structural capacity of the OCSP® system is assumed to be a function of the metal loss in the pile web. Literature sources and experience indicate that metal sections at the interlocks are relatively thick and may not appear to be as susceptible to corrosion-related failures as the webs. Although it is possible that corrosion could occur in the interlocks, for the purposes of this evaluation, corrosion of the web is assumed when considering impacts of corrosion on structural capacity.

6.2.6 Effects of Corrosion on Sheet Pile Tensile Strength

The remaining wall thickness values presented in Tables 6.2-3 and 6.2-4 were used to calculate the effect on the horizontal tensile strength for both types of piling. The horizontal tensile strength of the OCSP® bulkhead depends on both the web yielding strength (f_y = 50 ksi) and the interlock strength. The lower of the two will govern the design.

For the pristine sheet piling sections not subjected to corrosion, the web yielding strength usually exceeds or at least equals the interlock tensile strength. Hence the standard design assumption is the latter governs the design. Because the metal sections at the interlocks are relatively thick, corrosion is considered to have a negligible impact on the interlock strength. When the web thickness of the piling is significantly reduced by corrosion, the web yielding strength of the sheet piling section is reduced to a level lower than the interlock strength and becomes the controlling factor for horizontal tensile strength.

TABLE 6.2-3

Reduction in Piling Web Thickness, with Cathodic Protection for 40 Years

	Metal Loss of a 50-Year Period				
	Atmospheric	**Splash**	**Tidal**	**Submerged**	**Mud/Soil**
PS27.5 (0.4-in) Piling					
Landside	N/A	N/A	N/A	N/A	0.038 inch
Landside	N/A	N/A	N/A	N/A	0.038 inch
Total	N/A	N/A	N/A	N/A	0.075 inch
Percent Penetration	N/A	N/A	N/A	N/A	19%
Remaining Thickness	N/A	N/A	N/A	N/A	0.325 inch
PS31 (0.5-in) Piling					
Seaside	0.104 inch	0.104 inch	0.162 inch	0.116 inch	0.038 inch
Landside	0.038 inch	0.038 inch	0.038 inch	0.038 inch	0.038 inch
Total	0.158 inch	0.158 inch	0.200 inch	0.154 inch	0.075 inch
Percent Penetration	28%	28%	40%	31%	15%
Remaining Thickness	0.359 inch	0.359 inch	0.301 inch	0.347 inch	0.425 inch

N/A = not applicable

TABLE 6.2-4

Reduction in Piling Web Thickness, with Cathodic Protection for 45 Years

	Metal Loss of a 50-Year Period				
	Atmospheric	**Splash**	**Tidal**	**Submerged**	**Mud/Soil**
PS27.5 (0.4-in) Piling					
Landside	N/A[1]	N/A	N/A	N/A	0.018 inch
Landside	N/A	N/A	N/A	N/A	0.018 inch
Total	N/A	N/A	N/A	N/A	0.036 inch
Percent Penetration	N/A	N/A	N/A	N/A	9%
Remaining Thickness	N/A	N/A	N/A	N/A	0.364 inch
PS31 (0.5-in) Piling					
Seaside	0.104 inch	0.104 inch	0.147 inch	0.096 inch	0.018 inch
Landside	0.018 inch	0.018 inch	0.018 inch	0.018 inch	0.018 inch
Total	0.122 inch	0.122 inch	0.165 inch	0.114 inch	0.036 inch
Percent Penetration	24%	24%	33%	23%	7%
Remaining Thickness	0.379 inch	0.379 inch	0.336 inch	0.387 inch	0.464 inch

N/A = not applicable

The effect of reduced wall thickness can be determined by applying a reduction factor β to the horizontal tension factors of safety. β is calculated by the following equation:

$$\beta = 1 - \frac{\Delta t}{t_w}$$

where Δt is the reduction in wall thickness below the critical web thickness and t_w is the critical web thickness. Critical web thickness is the web thickness when the web yield strength equals to the interlock tension strength. For both the PS27.5 and PS31 sheet piling, the critical web thickness equals 0.40 inch. For the face sheets, the critical location for horizontal tension usually occurs slightly above the mudline, so the remaining wall thickness at the submerged zone was used to calculate factors of safety. For the tailwall, the remaining thickness for the mud/soil zone was used. The results for the controlling bulkhead section are presented in Table 6.2-5 and Table 6.2-6.

TABLE 6.2-5
Horizontal Tension Factor of Safety – Cathodic Protection System with 40-Year Service Life

Structural Component		Loading Conditions					
		Long-term Static (Drained)	**Long-term Static (Undrained)**	**OLE**	**CLE**	**MCE**	**Post-Earthquake**
Face Sheet (PS31)	Original	2.7	1.9	1.9	1.6	1.4	1.9
	After 50 yrs	2.3	1.6	1.6	1.4	1.2	1.6
Tailwall[a] (PS27.5)	Original	2.7	1.9	1.9	1.6	1.4	1.9
	After 50 yrs	2.2	1.5	1.5	1.3	1.1	1.5
Required FS		2.0	2.0	1.5	1.3	1.1	2.0

[a] Critical section at the wye connection.

TABLE 6.2-6
Horizontal Tension Factor of Safety – Cathodic Protection System with 45-Year Service Life

Structural Component		Loading Conditions					
		Long-term Static (Drained)	**Long-term Static (Undrained)**	**OLE**	**CLE**	**MCE**	**Post-Earthquake**
Face Sheet (PS31)	Original	2.7	1.9	1.9	1.6	1.4	1.9
	After 50 yrs	2.6	1.8	1.8	1.5	1.3	1.8
Tailwall[a] (PS27.5)	Original	2.7	1.9	1.9	1.6	1.4	1.9
	After 50 yrs	2.4	1.7	1.7	1.5	1.3	1.7
Required FS		2.0	2.0	1.5	1.3	1.1	2.0

[a] Critical section at the wye connection.

When compared with the required factors of safety for interlock tension in Table 2.2-2, results in Table 6.2-5 indicate sufficient factors of safety for horizontal tension strength could not be achieved for the Long-term Static (Undrained) and Post-Earthquake cases at the end of the 50-year design life, assuming a cathodic protection system with a service life of 40 years. The factors of safety are controlled by the tailwall critical section at the wye connection. Table 6.2-6 indicates that, if the service life of the cathodic protection system could be extended to

45 years, factors of safety will improve for all cases. But sufficient factors of safety still could not be achieved for the Long-term Static (Undrained) and the Post-Earthquake case.

6.2.7 Thermal Spray Coatings/Weld Splices

The construction contract documents require weld splices and larger exposed surfaces of bare metal to be repaired by metallizing (thermal spray). The documents are specific, requiring near-white metal abrasive blast and application in accordance with AWAS C2.23, "Specification for the Application of Thermal Spray Coatings (Metallizing) of Aluminum, Zinc, and Composites for the Corrosion Protection of Steel." Surface preparation, application, and quality-control testing are being observed by a NACE International Coating Inspector.

The specifications call for a metallizing alloy consisting of 85 percent zinc and 15 percent aluminum, applied to a minimum thickness of 20 mils. A low-viscosity penetrating sealer is applied to seal pores in the thermal spray coating. For this project, the sealer used was an aluminum pigmented moisture-cured polyurethane, thinned 15 percent. Sealing the pores improves the performance of the thermal spray coating by reducing pathways that allow corrosives to reach the steel substrate.

Properly applied, thermal spray coatings should be an acceptable coating repair procedure for galvanized steel components. Long-term performance testing of flame-sprayed coatings by the American Welding Society indicates that unsealed zinc-sprayed coatings can provide 19 years of service when submerged in seawater (12 mils minimum thickness). In severe marine atmospheres, 9 mils of unsealed zinc or 3 to 6 mils of zinc with a sealer should provide 19 years of service life. It appears that, at 20 mils minimum thickness, thermal-spray coatings should provide similar, or better, performance than hot-dipped galvanizing. Since the deterioration rate of hot-dipped galvanizing is anticipated to be higher than the thermal-spray zinc, the corrosion rates of hot-dipped galvanizing were used in the life-cycle analysis.

Corrosion has been reportedly been observed on some of the welded connections that have been thermal-spray coated. The appearance of corrosion on surfaces coated with thermal-sprayed zinc would not be expected in 3 years. Corrosion was not observed by CH2M HILL personnel in these zones during a June 2012 inspection of the facility. However, early stages of corrosion were observed at locations where the galvanizing was damaged by impact or welding from the land side of the structure.

The zone with the highest potential for corrosion of welded connections will be between elevation -10 feet MLLW and elevation +10 feet MLLW. Where possible, weld splices should be made above the tidal zone (which will allow visual observation and repair, if required in the future) or below the mud line where the potential for corrosion is much lower.

6.2.8 Corrosion Protection System – Operation and Maintenance Considerations

The following two subsections provide brief summaries of operations and maintenance considerations for the corrosion protection system. These summaries cover localized galvanizing repair and maintenance of cathodic protection systems.

6.2.8.1 Galvanizing

The galvanizing on structural steel components will deteriorate at various rates, depending on the exposure condition. Once the structure is installed, it will not be possible to repair or maintain galvanized steel coating in the submerged, tidal, or mud zones. Nor will it be possible to replace the galvanizing on the piling exposed to soil.

In the atmospheric zone, it would be possible to apply surface-tolerant marine epoxy coatings where steel is exposed at isolated defects in the galvanizing. These repairs would be recommended only for small isolated areas where corrosion becomes apparent within a short period of time, and would be intended to provide corrosion protection only as long as the remaining galvanizing remains intact.

As noted above, CH2M HILL personnel observed corrosion at isolated locations where the galvanizing was damaged by landside activities (mechanical impact, welding). It will not be practical to repair these areas with a thermal spray product. To reduce the impact on the service life of the zinc coating in the vicinity of the damaged area, the surface should be mechanically tool cleaned (SSPC SP-11) and coated with a marine grade, underwater-curing epoxy. The epoxy repairs will need to be monitored and replaced as part of the annual maintenance program.

6.2.8.2 Cathodic Protection System

To achieve the maximum life cycle for the marine terminal structures, uninterrupted operation and diligent maintenance of the cathodic protection system will be required. This cathodic protection system will be complex, have a very high capacity, and will include a large number of rectifiers, anodes, terminal boxes, and connecting wires. The complexity of the system is necessary, however, to effectively distribute a large amount of cathodic protection current to the buried and submerged surfaces of the structure. The complexity of the system also provides some degree of redundancy.

The operational status of each rectifier will need to be checked and recorded on a monthly basis. The entire system will need to be tested and adjusted on an annual basis. A standard operating procedure should be developed to ensure timely implementation of all repairs that are identified during annual and monthly testing. Annual maintenance costs for an impressed current cathodic protection system were identified in the original life-cycle cost analysis, but these costs were shown only at 5-year increments in the spreadsheets included with the report. This issue is discussed in more detail in Section 6.2.10.

The seaside cathodic protection system components will be continuously subject to mechanical damage. The corrosion control report (Coffman, 2007b) acknowledges the possibility of anode damage, and the seaside anodes were designed to allow replacement without any underwater work. The seaside anodes are installed on individual, dedicated fender piles, so anode replacement will require removal and replacement of fender piles.

The seaside anodes were designed for a 20-year service life. The seaside anodes would therefore need to be replaced once to achieve a 40-year life cycle. However, CH2M HILL's experience is that 20 years may be too long for these components exposed to this marine environment. A 15-year life cycle may be more appropriate. Using a 15-year service life cycle would require replacement of the seaside anodes two times over a 40-year structure life cycle. If the seaside anodes are to be used over the full 50-year structure life cycle, they would need to be replaced three times. The change in anode replacement schedule affects structure life-cycle costs, and is discussed in more detail below.

The landside cathodic protection anodes will be installed in an 8-inch-diameter hole, drilled to a depth of 210 or 270 feet, depending on the number of anodes installed (15 or 24 anodes, respectively). Two deep anode groundbeds are to be installed inside each cell. If operated at 75 percent of the maximum current output, the maximum current discharged through a single deep groundbed with 24 anodes would be 33 amperes (1.4 amperes per anode). The anodes specified are tubular cast iron, Anotec 2684 (or equivalent). At 1.4 amperes per anode, the anodes should have a theoretical life of 40 years.

The design life in the original life-cycle cost estimate for the landside groundbeds is 40 years. Although the groundbed components have a theoretical capability to provide 40 years of service, CH2M HILL's experience has been that deep anode groundbeds can develop high resistance over time. As the circuit resistance of the groundbed increases, the current output decreases, which can eventually compromise the level of cathodic protection. High current discharge through the anode groundbeds can accentuate this process. Regular monitoring of the rectifier output and review of the annual testing will identify changes in circuit resistance that can be used to schedule deep anode replacements, if required.

Replacement of the deep anode groundbeds is technically feasible, but will add cost to the project. To meet the 50-year design life, one scheduled replacement of the deep anode groundbeds should be included in the life-cycle cost analysis. The impact of this replacement activity is discussed in more detail below.

6.2.9 Structure Inspections

Periodic inspections of the structure are recommended to verify effective operation of the cathodic protection system and identify areas that may be subject to continued corrosion (such as the tidal zone). Welded joints in the tidal zone should be monitored. Costs for periodic inspections were included in the original life-cycle cost analysis.

6.2.10 Life-Cycle Costs

A life-cycle cost analysis was developed for this facility in 2007. The life-cycle cost analysis compared the relative costs of 10 corrosion-control alternatives. Galvanized structural elements with an impressed current cathodic protection system were found to have the lowest life-cycle cost (net present value).

Several items were identified in this review that will impact the original 40-year life-cycle cost. The original life-cycle cost analysis was modified to illustrate the impact of these items on total net present value cost and to show that the selected alternative was still the least costly. The following items were included in the update:

- Power costs. It is assumed that the galvanizing will be consumed on the seaside surfaces before the cathodic protection system is installed. Since it will take more current to protect bare steel than galvanized steel, the power costs were increased.
- Annual maintenance costs. Annual maintenance costs were shown as $100,000 in the analysis, but were added only at 5-year intervals.
- Replacement of seaside anodes at 15 and 30 years. In the original analysis, replacement of the seaside anodes was placed in year 30. Due to the very aggressive conditions and potential for seaside anode damage, two seaside anode replacements were scheduled and included at year 15 and year 30.
- Replacement of the landside groundbeds in 25 years. The original life-cycle analysis did not include replacement of the deep anode groundbeds, since the intended design life was 40 years. For the purposes of this evaluation, the landside groundbeds were scheduled for replacement in year 25. Anode replacement costs are estimated at $100,000 per groundbed (2008 dollars).

The original cost analysis was prepared using 2008 dollars and was based on a 40-year design life. The updated analysis is also based on 40 years and 2008 dollars, to allow direct comparison with the original values. A revised life-cycle cost for a 50-year service life was also prepared. Comparison between original and updated 40-year life-cycle costs and the updated life-cycle costs for a 50-year life cycle are summarized in Table 6.2-7 (for Options A and J). Option A is the option that was selected for this project: galvanized steel structural components and impressed current cathodic protection. Option J, bare steel structural components with cathodic protection, was the second lowest cost option.

TABLE 6.2-7
Life-Cycle Cost Comparisons, 40-Year Life Cycle

Option	Original Life-Cycle Cost Analysis (40 years)	Revised Life-Cycle Cost Analysis (40 years)	Revised Life-Cycle Cost Analysis (50 years)
A (Galvanized Piles, Cathodic Protection)	$218,000,000	$231,903,000	$236,963,000
J (Bare Piles, Cathodic Protection)	$276,000,000	$288,988,000	$294,913,000

Note: All costs are in 2008 dollars.

6.3 Summary of Findings

OCSP® bulkhead constructed at the PIEP is a tension structure relying on the horizontal tension developed between the sheet piles to retain soil at the front face, as well as transfer the loads through the tailwalls to the backfill. The loss of this horizontal tension will result in complete failure of one or more cells of the structure. The most critical structural element of the OCSP® bulkhead is the interlock connection between the sheet piles. Appropriate construction methods and quality control procedures are crucial in order for the interlocks to achieve their designed strength.

Significant deviations from the original design were found in the review of the as-built conditions in the PIEP. The most significant deviations were found to be the widespread defects of sheet pile damage and disengaged interlocks. The causes of the damage are believed to be associated with several different sources, including unbalanced soil loading during driving, hard driving during pile installation, and the presence of large rock such as riprap in the area of sheet pile installation.

The presence of the defects was evaluated by conducting numerical analyses using the computer program FLAC^{3D}. These analyses were conducted for static earth and water loading only; seismic loads were not explicitly modeled. Results of the analyses show that the presence of defects has different impacts on the facewall sheets and the tailwalls:

- **Facewall defect.** A moderately sized defect at the bottom of the facewall sheets will not likely propagate upward based on this defect evaluation. Local ground subsidence could develop at the cell with the facewall sheet defect. The ground subsidence may cause pavement and utility damage, be a life safety issue, and hinder the normal operations of the wharf. However, complete failure of the entire bulkhead system or progressive collapse seemed unlikely. However, this conclusion is based on a static analysis; repeated load effects could result in a progressive failure. Modeling this in the dynamic realm would be extremely difficult.
- **Tailwall defect.** A moderately sized defect at the bottom of the tailwall, especially at locations close to the wye connection, is highly likely to propagate upward, causing the “unzipping” of an entire tailwall. Once the entire tailwall resistance is lost, failure of adjacent cells is likely to occur. If the planned utilidor is constructed, the failure will likely be limited to a localized area of several cells. As with the facewall, these evaluations were based on static loading. Repeated cycles of seismic load would be expected to accelerate any unzipping mechanism.

The original design of the PIEP included a corrosion protection system consisting of galvanizing and an impressed current cathodic protection system. Life-cycle costs indicate this is the most cost-effective over the service life of the structure. The protection system will reduce, but not eliminate, metal loss due to corrosion. In addition, higher corrosion rates will be experienced on buried and submerged surfaces before the impressed current cathodic protection system is installed. Although the original design assumed that the cathodic protection would be in place within 2 years of construction, this has not occurred. For this analysis, it was assumed that the cathodic protection system will not be installed for the first 5 years of the structure’s life. Therefore, for a 50-year structure service life, the maximum time cathodic protection will be in place is 45 years.

Using corrosion rates for structures exposed to the Cook Inlet environment, the reduction in metal thickness over time was calculated. These calculations show that sufficient factors of safety for the structure could not be achieved for the Long-term Static (Undrained) and Post-Earthquake cases at the end of a 50-year design life, assuming a cathodic protection system with a service life of 40 years (per the original life-cycle cost evaluation). If the service life of the cathodic protection system could be extended to 45 years (the maximum possible for a 50-year structure service life), factors of safety will improve for all cases. But factors of safety for the Long-term Static (Undrained) and the Post-Earthquake case remain marginal (approximately 15 percent under the required values for Section G-G in North Extension 2). However, OCSP® structures with lower stresses such as the Dry Barge Berth with a lower wall height are above the required FS values.

It should also be noted that ALWC could potentially further lower the FS between elevation +5 and -5 MLLW. ALWC occurs primarily by localized corrosion (pitting); it is more difficult to estimate the effects of localized corrosion on the FS since it is not a uniform reduction in metal thickness. However, under a worst-case scenario (a uniform corrosion loss of 40 mils per year due to ALWC), it is possible that the metal thickness could be reduced to a remaining thickness of 0.250 to 0.200 inch after 50 years in the lower tidal/upper submerged exposure zones. This corresponds to a 50 to 60 percent reduction in wall thickness of a PS31 pile, and would be additional erosion into the FS.

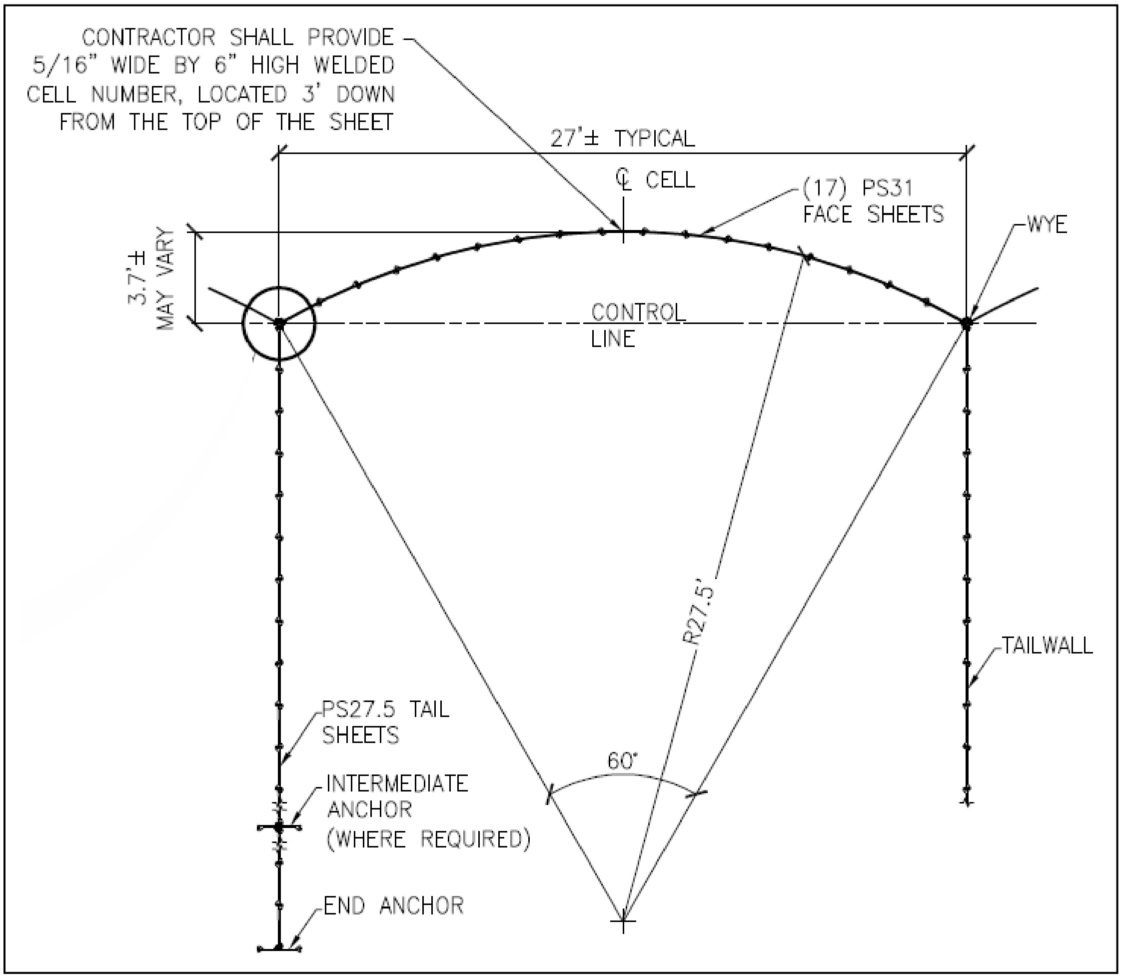


FIGURE 6.1-1. Typical Sheet Pile Cell Layout (PND, 2010b)

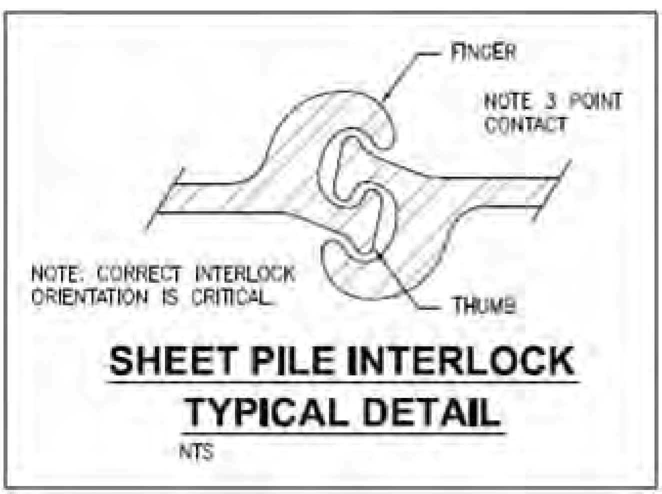


FIGURE 6.1-2. Three-Point “Finger-and-Thumb” Interlock (PND, 2010b)

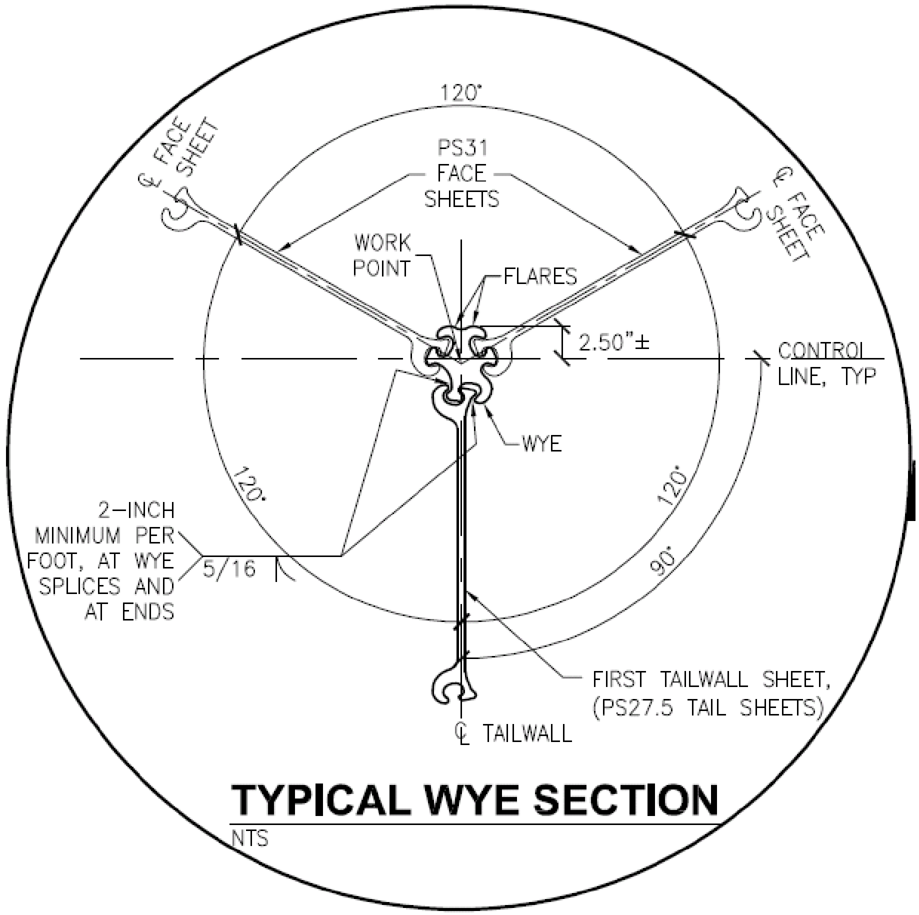


FIGURE 6.1-3. Typical Wye Pile Section (PND, 2010b)

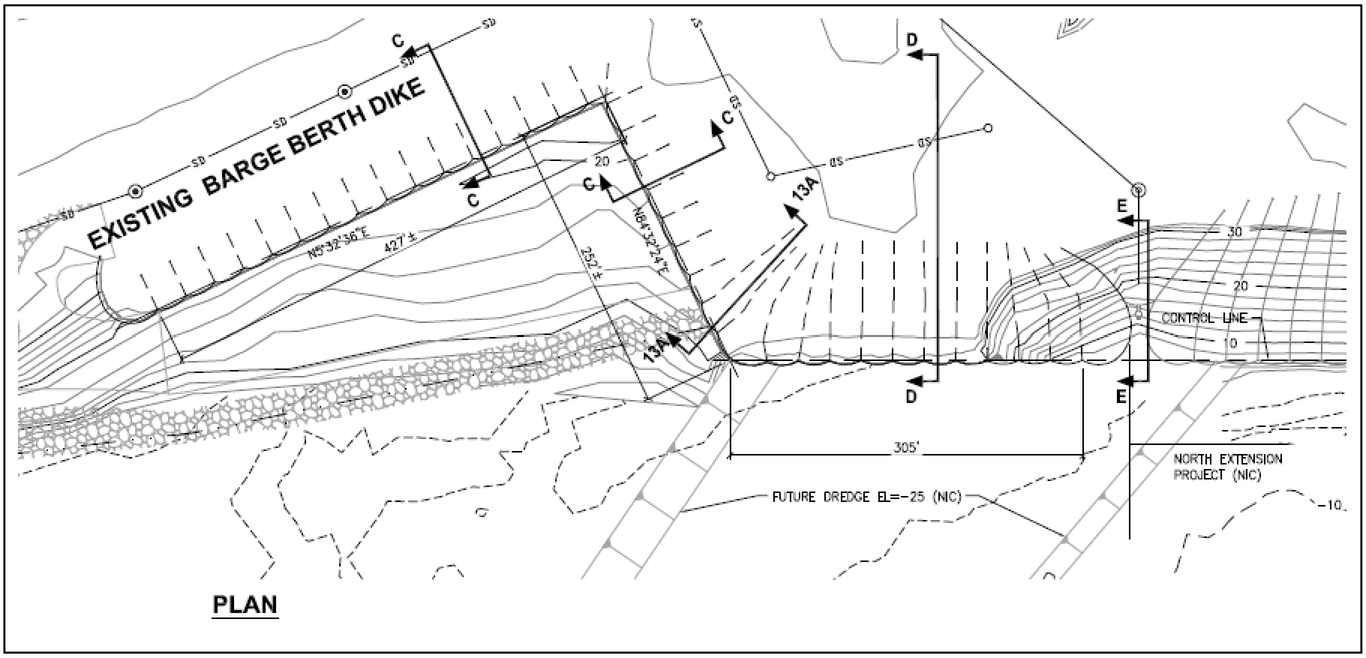


FIGURE 6.1-4. Dry Barge Berth and Wet Barge Berth As-Built Layout (PND, 2008c)

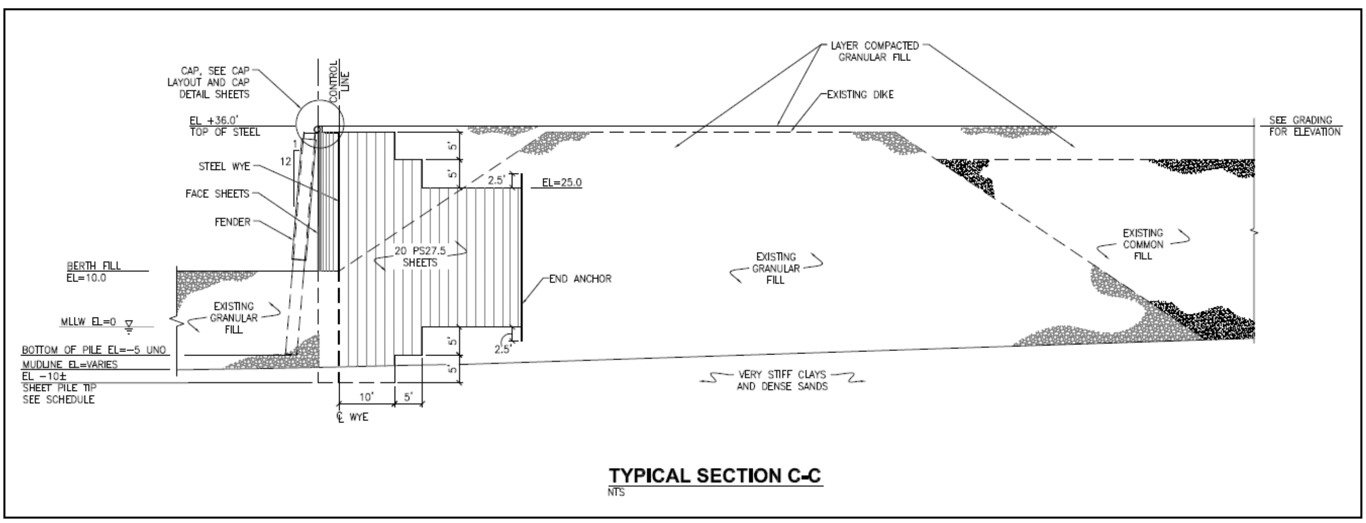


FIGURE 6.1-5. Dry Barge Berth As-Built Typical Section C-C (PND, 2008c)

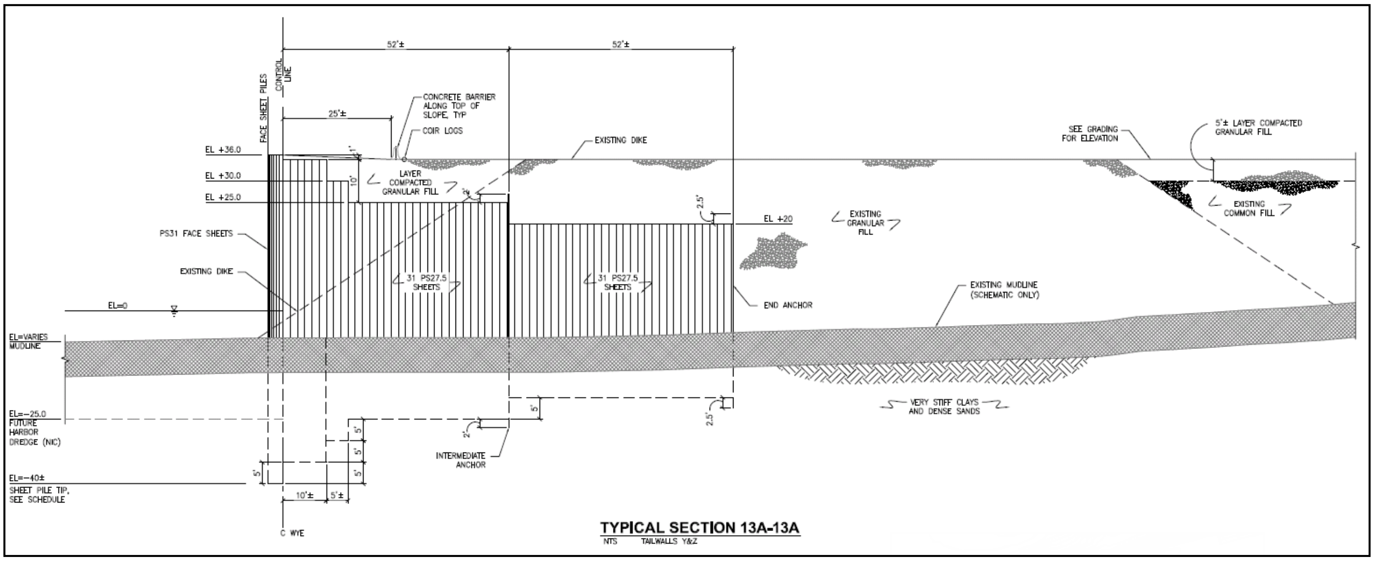


FIGURE 6.1-6. Dry Barge Berth As-Built Typical Section 13A-13A (PND, 2008c)

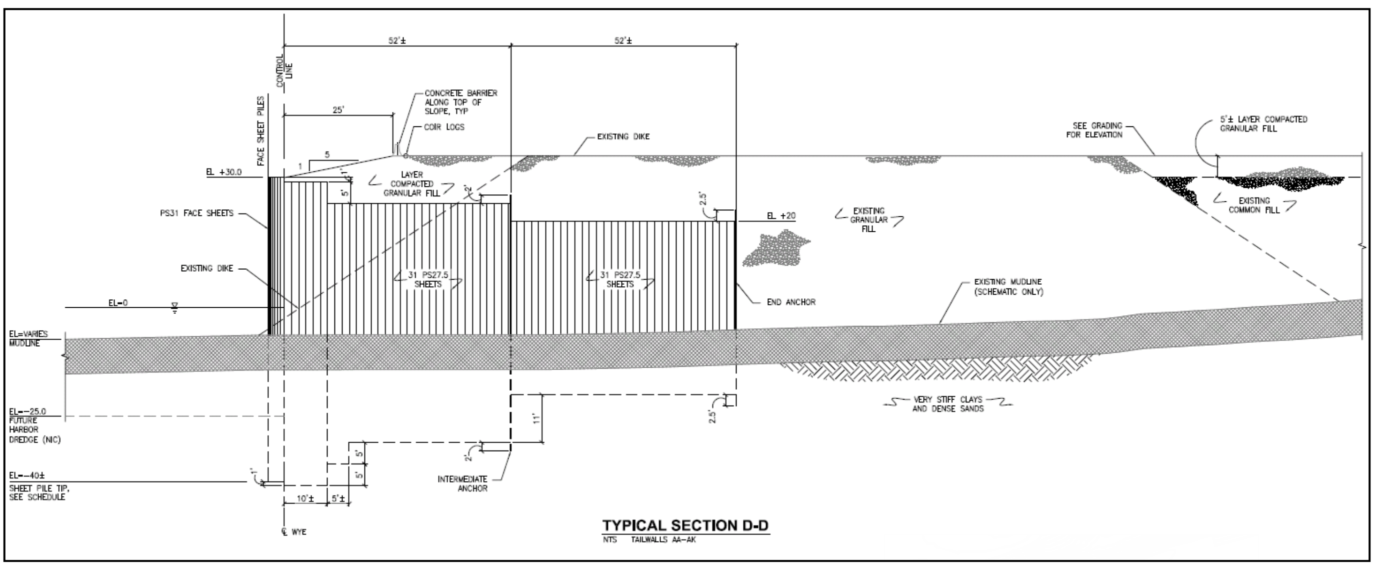


FIGURE 6.1-7. Wet Barge Berth As-Built Typical Section D-D (PND, 2008c)

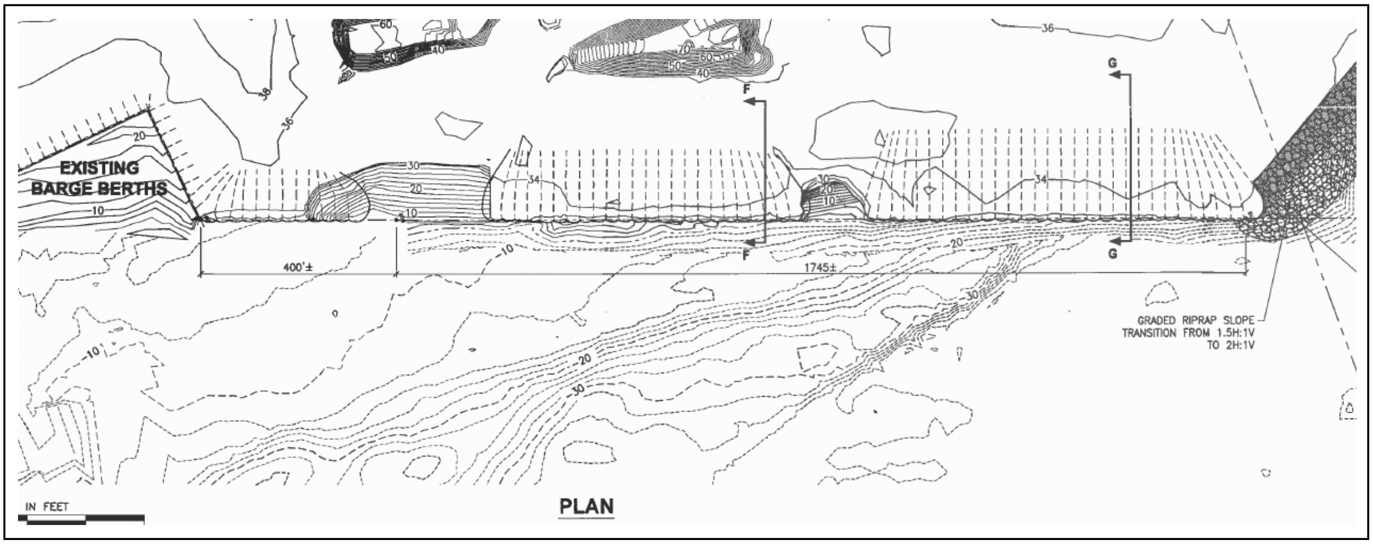


FIGURE 6.1-8. North Extension 1&2 As-Built Layout (PND, 2010b)

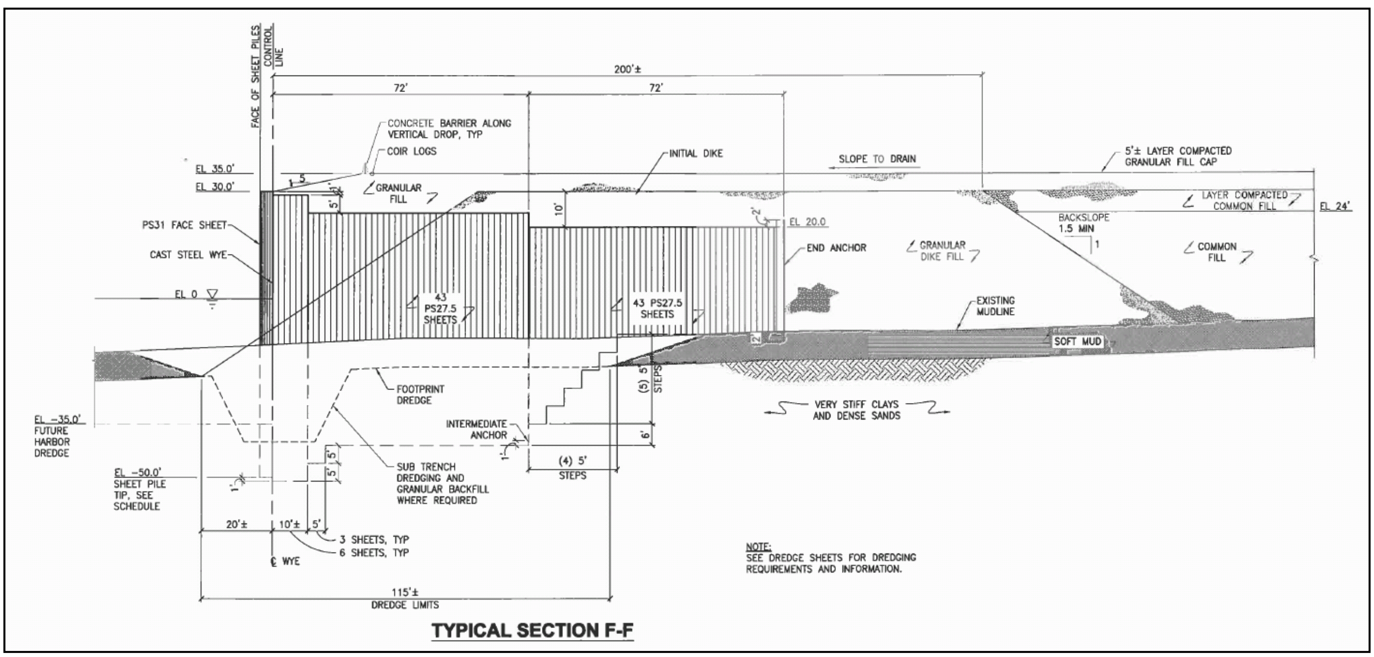


FIGURE 6.1-9. North Extension As-Built Typical Section F-F (PND, 2010b)

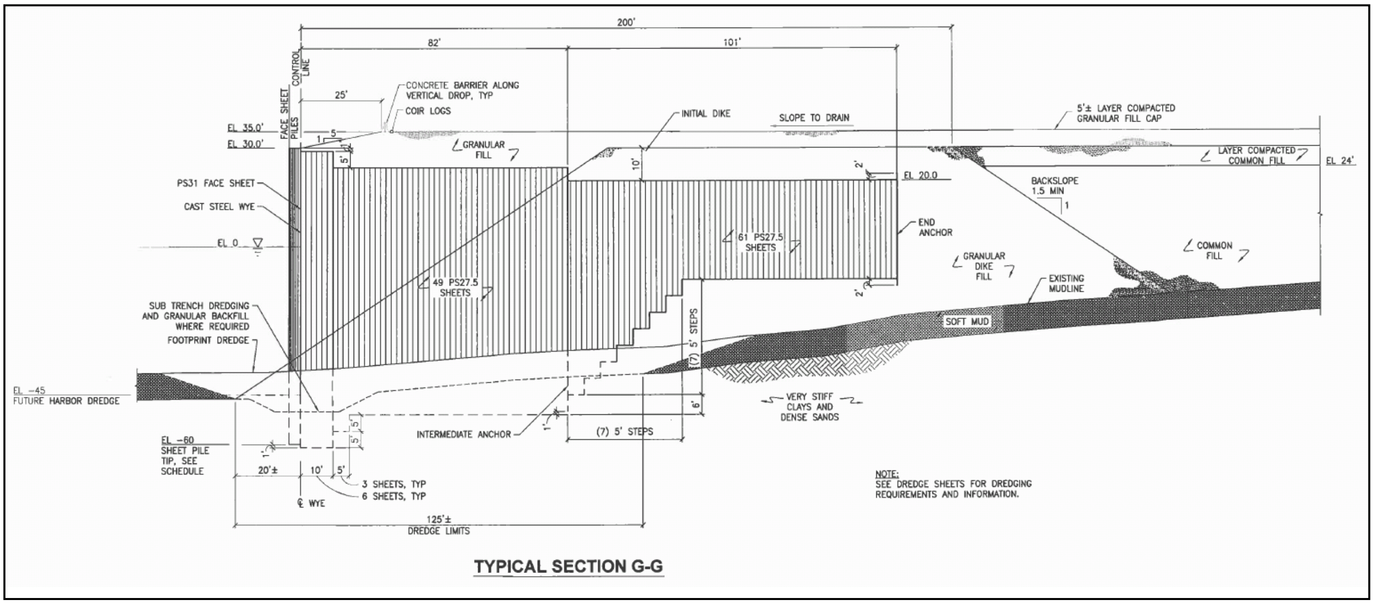


FIGURE 6.1-10. North Extension As-Built Typical Section G-G (PND, 2010b)

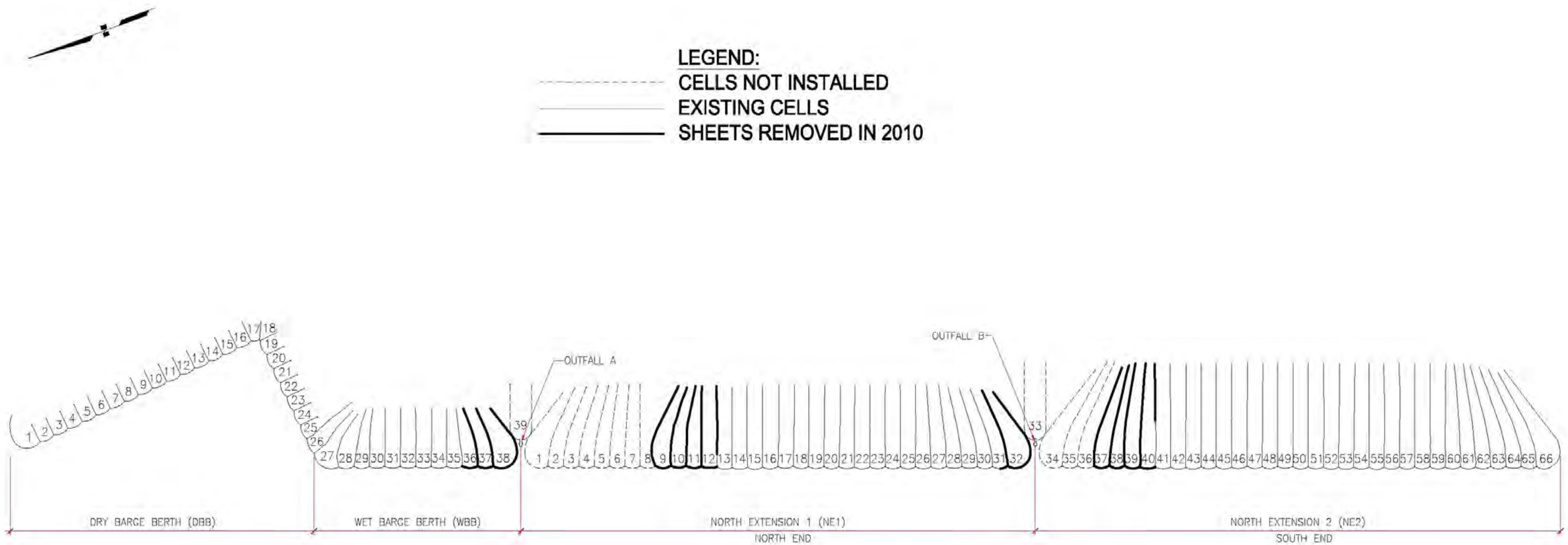


FIGURE 6.1-11. Cell Numbers in Barge Berth and North Extension areas (ICRC, 2010)

FIGURE 6.1-12. Pile Tip Damage – Case 1 (Photo Courtesy of ICRC)

FIGURE 6.1-13. Pile Tip Damage – Case 2 (Photo Courtesy of ICRC)

FIGURE 6.1-14. Curled Pile Tip (Photo Courtesy of ICRC)

FIGURE 6.1-15. Pinched Piles (Photo Courtesy of ICRC)

FIGURE 6.1-16. Bent Pile – Case 1 (Photo Courtesy of ICRC)

FIGURE 6.1-17. Bent Pile – Case 2 (Photo Courtesy of ICRC)

FIGURE 6.1-18. Disengaged Interlock – Case 1 (Photo Courtesy of ICRC)

FIGURE 6.1-19. Disengaged Interlock – Case 2 (Photo Courtesy of ICRC)

FIGURE 6.1-20. Threading of Sheet Pile (Photo Courtesy of PND and ICRC)

FIGURE 6.1-21. Threading of Sheet Pile and Wye (Photo Courtesy of PND and ICRC)

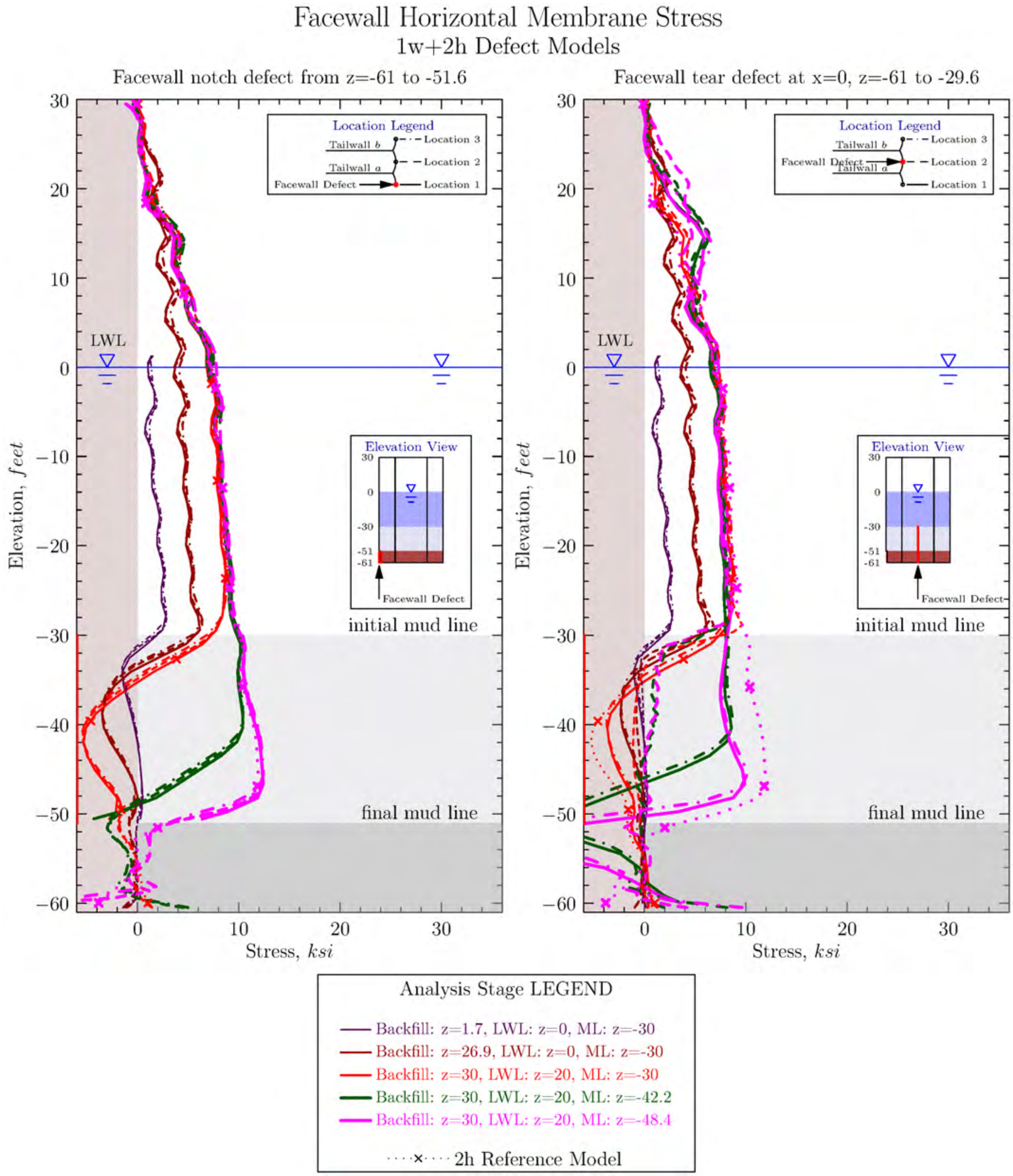


FIGURE 6.1-22. Facewall Horizontal Tensile Stress – Face Sheet Defect Models 1 & 2

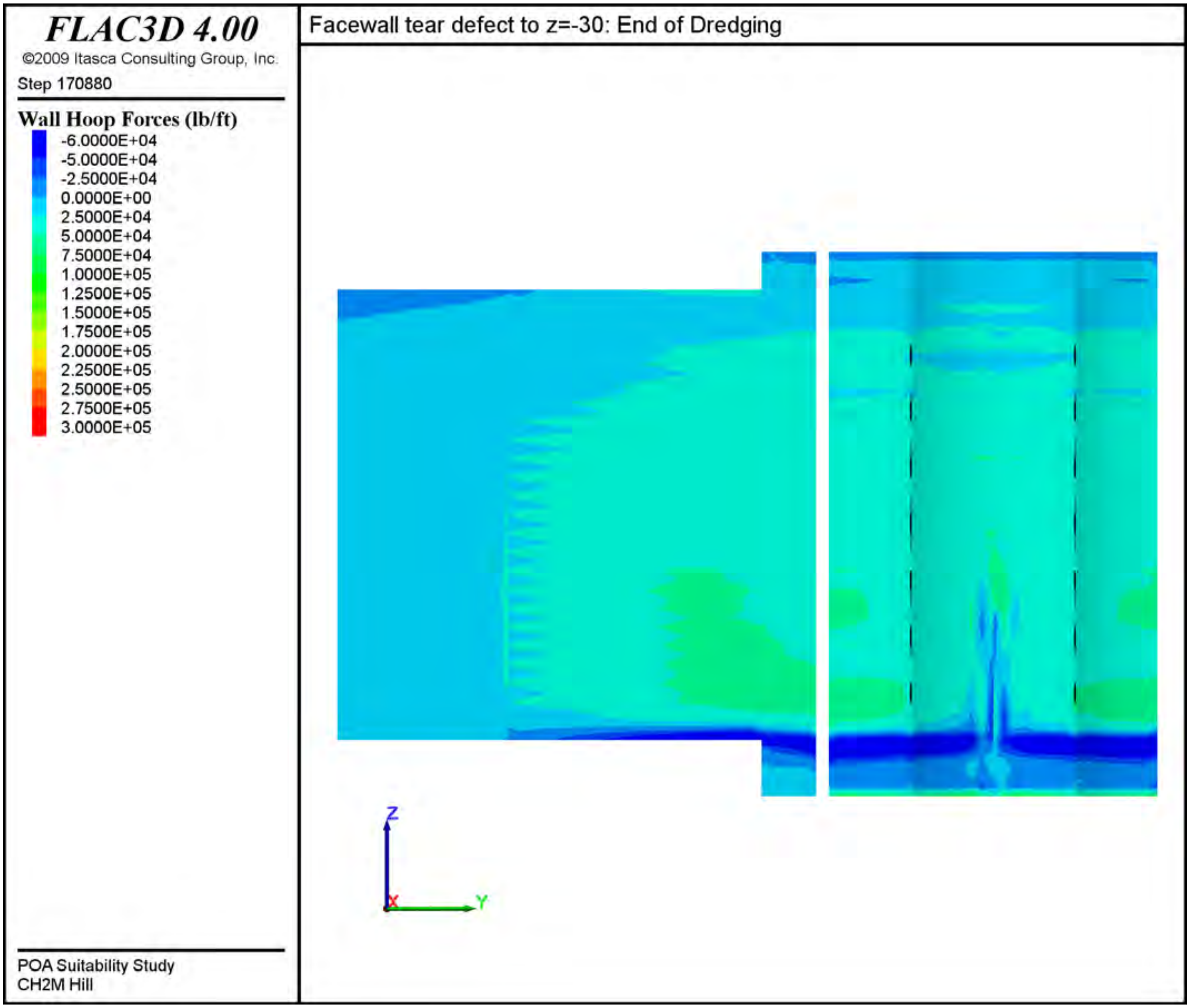


FIGURE 6.1-23. Facewall Horizontal Tensile Stress Contour – Face Sheet Defect Model 2 (-30')

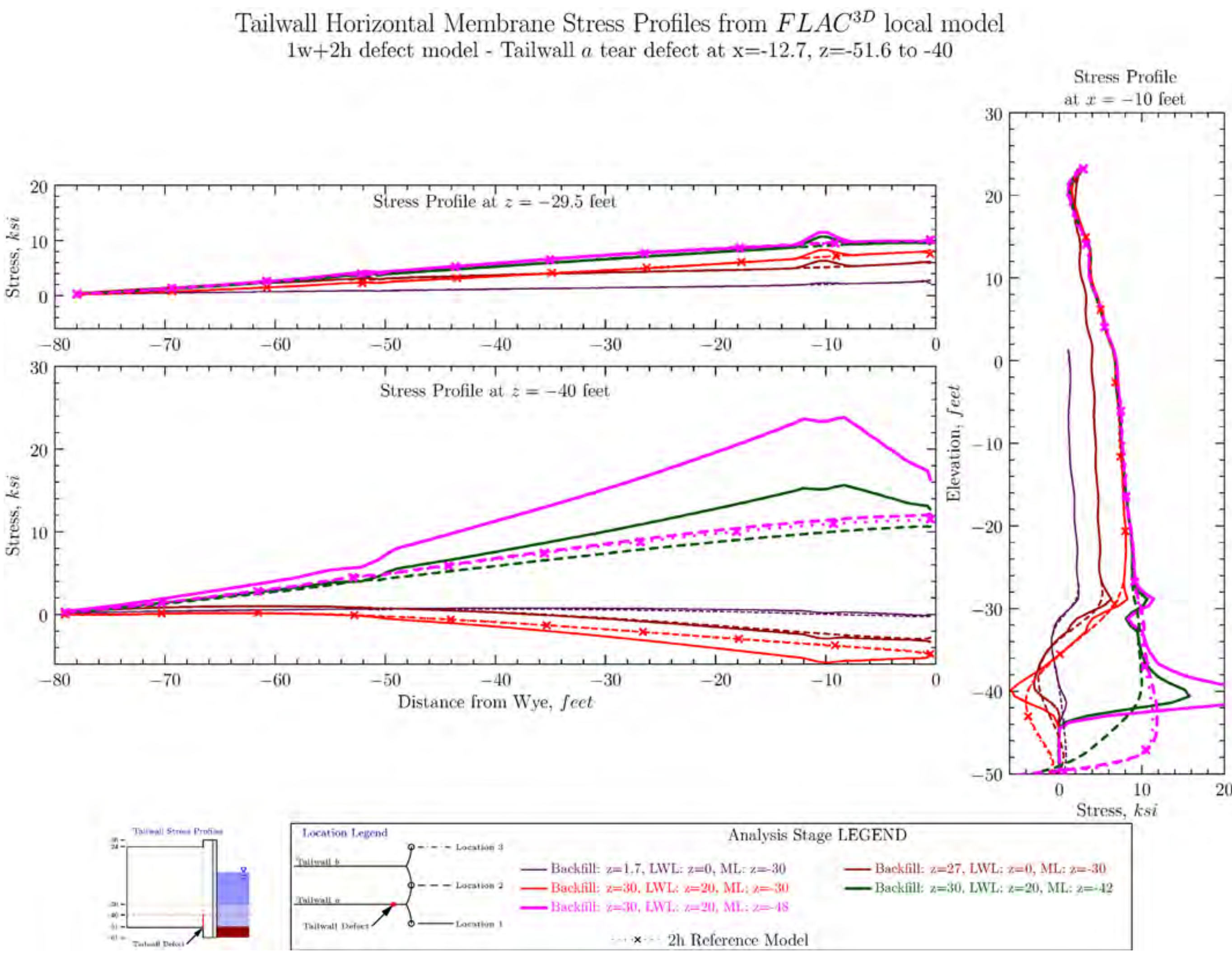


FIGURE 6.1-24. Horizontal Tensile Stress – Tailwall Defect Model 1

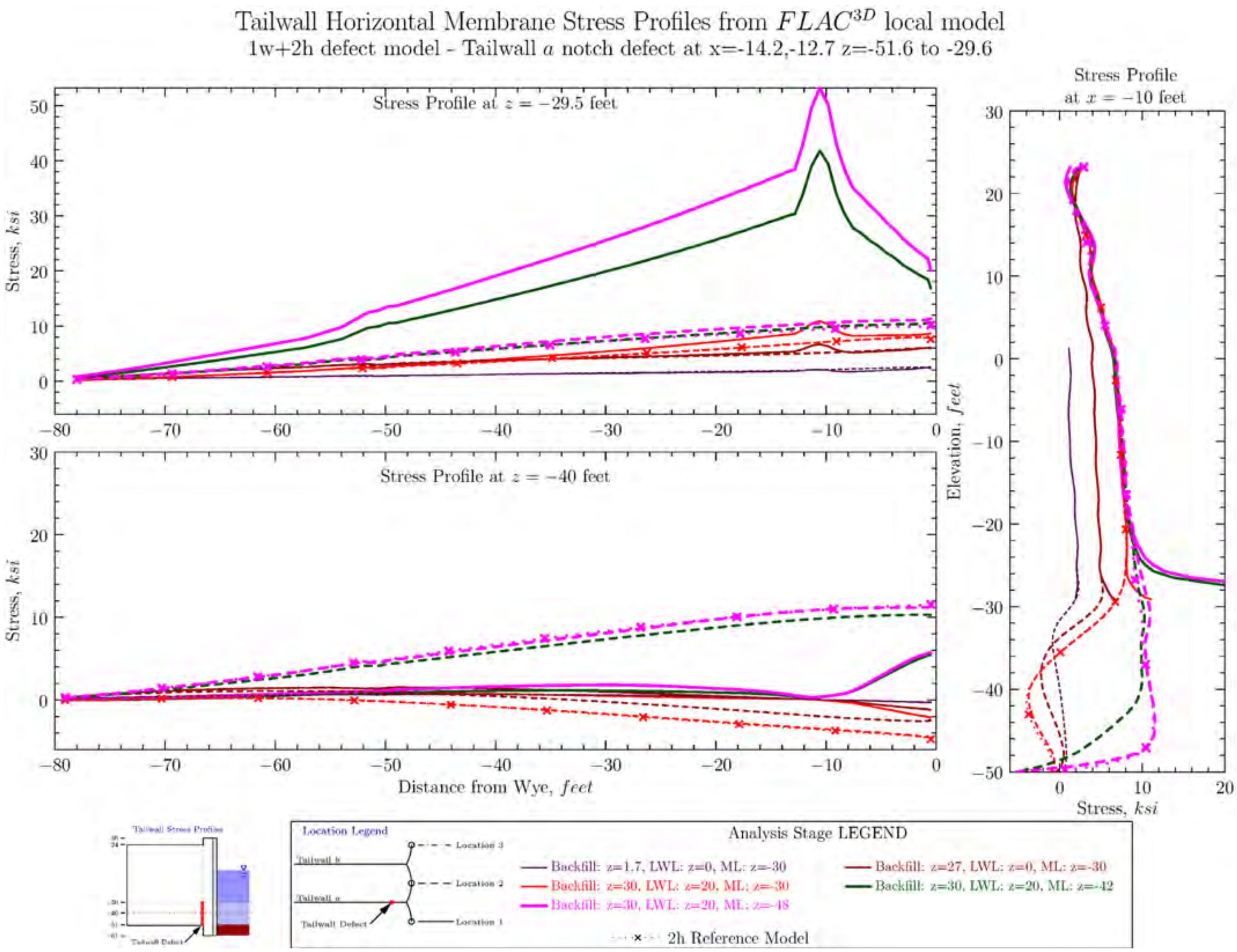


FIGURE 6.1-25. Horizontal Tensile Stress –Tailwall Defect Model 2

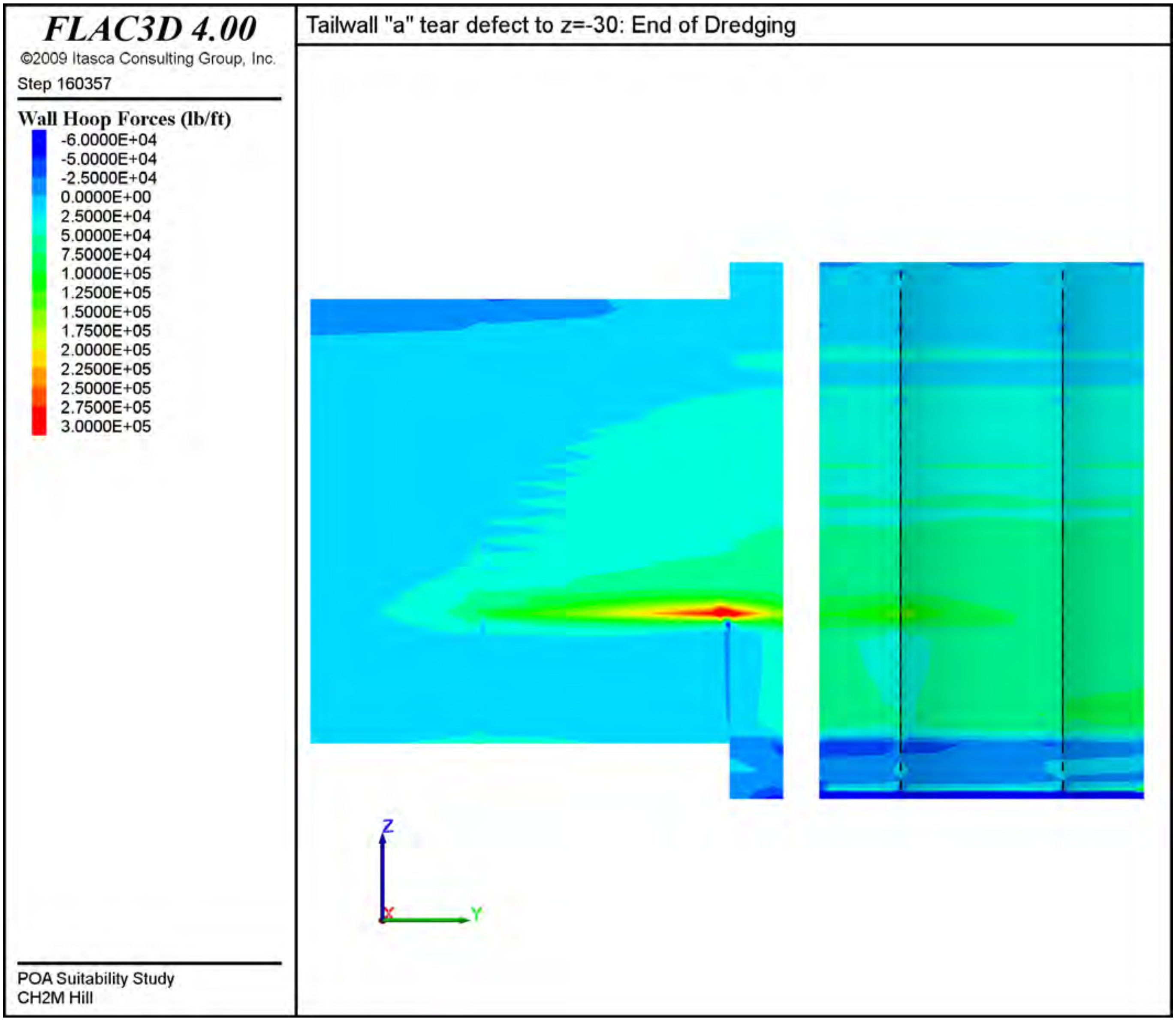


FIGURE 6.1-26. Horizontal Tensile Stress Contour –Tailwall Defect Model 2

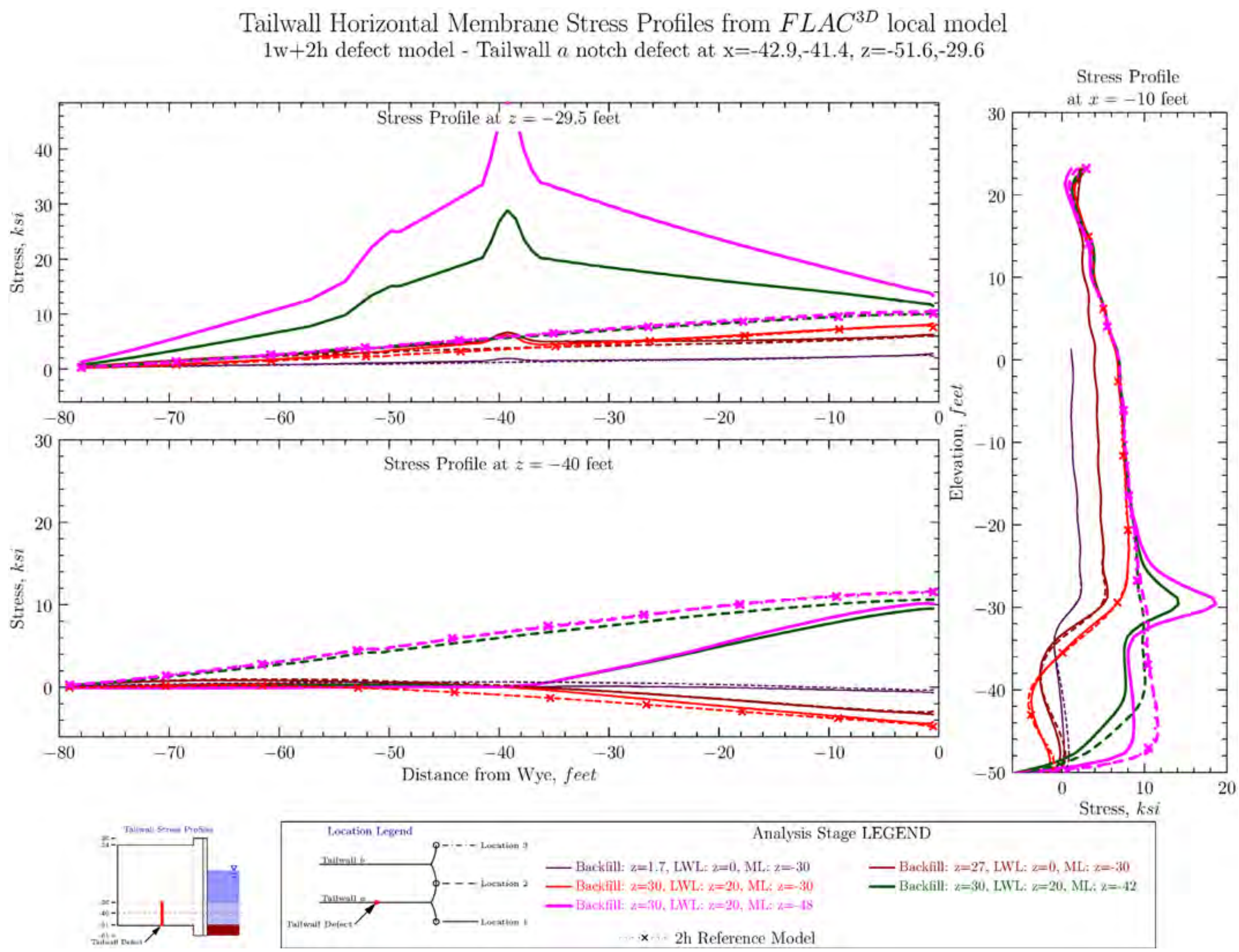


FIGURE 6.1-27. Horizontal Tensile Stress –Tailwall Defect Model 3

Numerical Modeling

This section summarizes the numerical modeling of the OCSP® system that has been completed by CH2M HILL for the PIEP. The numerical modeling involved three-dimensional modeling with the computer program FLAC^{3D}. The FLAC^{3D} analyses were carried out to evaluate the performance of the OCSP® structures at the North Expansion under gravity (static) loads and under three design earthquake levels: OLE, CLE, and MCE. The numerical modeling was used to analyze the as-designed and as-built geometry of the OCSP® system in order to assess expected performance of the system beyond what can be determined when using conventional design methods as described in Section 5. Special considerations included soil-structure interaction of the OCSP® system, as well as the inertial effects of loads during seismic events. This section describes the components of the numerical models developed and presents the results of FLAC^{3D} analyses of the as-designed and as-built models, results of FLAC^{3D} modeling of defects within the OCSP® system, and a comparison of FLAC^{3D} numerical modeling results with the results of the limit-equilibrium analyses discussed in Section 5 of this report.

7.1 Numerical Modeling Methods

The three-dimensional explicit finite-difference program FLAC^{3D} (Itasca, 2009) was used to conduct the analyses described in this section. In addition to summarizing the basic need for additional numerical modeling, this section summarizes past numerical modeling that has been performed for the OCSP® system at the POA, the rationale behind the selection of FLAC^{3D}, and the general approach used for numerical modeling.

7.1.1 Need for Numerical Modeling

There are no specific requirements to use numerical modeling for performance evaluation of the OCSP® bulkhead. However, numerical modeling appears to be well suited for application to the OCSP® at the POA, as this analysis technique integrates the behavior of several components—the soil, the environmental loads, and the sheet pile wall structure—into an overall system response that accounts for interaction between each component and provides results in terms of stresses and deformations that satisfy compatibility requirements and achieve equilibrium. These stresses and deformations can be used to determine whether individual members are being overstressed and whether the permanent deformation of the system is within required limits. The need for this type of analysis tool is usually driven by applications, such as with the OCSP® system, where decoupling performance into the response of each component does provide a sufficient understanding of system response.

Given the complexity of the OCSP® system, numerical modeling was used to obtain estimates of deformations and internal stresses within the OCSP® structure, as the system responded to and interacted with environmental loads and earth movement. The environmental loads included static earth pressures, groundwater pressures, and tidal fluctuations on the face of the wall, as well as inertial response of the soil mass within the OCSP® structure as the system responded to seismic events. Loads from ground shaking were transferred to the OCSP® facewall in the form of earth pressures; the tailwall developed reaction to the forces on the facewall from the same soil that caused the loads. By implementing the numerical modeling approach, FLAC^{3D} offered the opportunity to understand the interaction process as the OCSP® was constructed, as tidal and groundwater fluctuations occurred, and as the ground was shaken at different levels during the three levels of seismic loading.

Three-dimensional modeling methods within FLAC^{3D} were used for this modeling effort, rather than a two-dimensional model, because of the unique configuration of the OCSP® system. The tailwalls used to support the facewall are spaced approximately 30 feet apart. Although a two-dimensional representation of this configuration can be made, and in fact was used by the PND design team in their final design work, the two-dimensional model fails to account for load redistribution that occurs at the tailwall, particularly under seismic loading. A decision was made by CH2M HILL at the start of the suitability study to apply three-dimensional modeling methods, similar to what was originally used by Terracon. It was felt that the three-dimensional modeling would avoid uncertainties associated with simplification to a two-dimensional model and would better capture the seismic deformations that could occur. The three-dimensional model also offered the opportunity to evaluate the potential for localized failure of the sheet pile wall from "unzipping" of interlocks that had been opened during construction.

7.1.2 Past Numerical Modeling

The deformation analyses conducted for the evaluation of the OCSP® bulkhead by Terracon and the PND design team were based on using two different types of two- and three-dimensional numerical modeling. These methods differed depending on the phase of design:

- **Modeling for Predesign**. During the predesign stage of the project (2004-2007), numerical modeling was performed by Terracon to evaluate deformations of a pile-supported wharf and the OCSP® system for gravity and seismic loading. The finite-difference programs FLAC (Itasca, 2008) and FLAC^{3D} (Itasca, 2005) were used for the analysis of the pile-supported deck (PSD) and the OCSP® wharf structures (Terracon, 2004, 2005a, 2005b, 2006b, 2006c, 2007a, and 2007b).
- **Modeling for Final Design.** For final design, the PND design team used the finite element software PLAXIS-2D and -3D (PND, 2008) to conduct deformation analyses, as part of their performance evaluation of the OCSP® system. Most of the PLAXIS analyses relied on two-dimensional models after calibrating their input parameters to closely match the results obtained from the three-dimensional model.

None of the previous numerical modeling efforts specifically addressed issues related to the interlock behavior of the sheet piles or the sheet pile-soil interface. In the current CH2M HILL evaluation, the effects of the interlock behavior and the shear resistance of the sheet pile-soil interface are included in the numerical models developed for the as-built and as-designed systems, as will be described later in this section.

7.1.3 Software Selection

Software selection was a critical initial step in the numerical analysis in order to successfully achieve the goals of the modeling requirements without significantly compromising the productivity and the efficiency of the modeling efforts. There are potentially several requirements that can influence the selection of the analysis software. Key requirements for the intended PIEP modeling included capabilities for two- and three-dimensional analysis, sequential analysis for staged construction, availability of structural elements with interaction capabilities with the soil elements, and the availability of constitutive soil models suitable for static and dynamic analysis.

Several commercial software programs are available to perform numerical analysis; however, only a few can fulfill the key requirements for the intended numerical modeling of the project. After a rigorous screening process of commercial software, such as FLAC, FLAC^{3D}, PLAXIS, DIANA, and Midas GTS, it was decided that FLAC^{3D} (Fast Lagrangian Analysis of Continua in 3 Dimensions) is best suited to achieve the numerical modeling goals of the project. In addition, the CH2M HILL project team was familiar and experienced with the use of FLAC and FLAC^{3D}, and these programs had been used during modeling efforts in design stages by Terracon. For these reasons Version 4.0 of FLAC^{3D} from Itasca Consulting Group was selected and used in the evaluation of the POA OCSP® system.

7.1.4 Numerical Modeling Approach

The performance assessment of the existing OCSP® system for the North Expansion has two major components: (1) an evaluation of the system as it was designed and (2) an evaluation of the OCSP® system in its constructed condition. The intent of these evaluations was to establish whether the OCSP® system would meet the displacement design criteria for the North Expansion structures at the POA while also satisfying structural design criteria relative to stresses within the OCSP® system and interlock performance. The differences between these models are as follows:

- **As-Designed Model.** In the as-designed case evaluation, the numerical model was developed to replicate as close as practical and feasible the geometric layout and the properties of the OCSP® bulkhead for one of the design sections in the North Expansion used by the original designers. This evaluation for the as-designed case utilized soil properties and profiles as close as practical to those used during the design phase by the Terracon and PND design teams.

- **As-Built Model.** In the as-built case, the numerical model was developed to replicate as close as practical the actual constructed OCSP® geometry at the North Expansion. This evaluation used CH2M HILL's interpretation of the geometry and soil properties at the location of the cross-section selected based on the as-built condition.

The two models used for the evaluation of the as-designed and the as-built cases were based on common features and functionalities, which are combined into one basic model that is referred to as the primary model. However, modification of the primary model was required to include specific features pertinent to each evaluation, or separate development of other models was deemed necessary to evaluate and characterize a localized behavior or response to include its effect, in a simplified manner, in the primary model.

The following approach was taken in developing the initial test model, the implementation of the test model in the as-designed and as-built evaluations, and the use of a local model to evaluate specific questions regarding OCSP® performance:

- **Initial Test Model.** As a first step in the numerical modeling approach, an initial test model was developed using $FLAC^{3D}$ to gain a basic understanding of the fundamental elements used in previous numerical models of the OCSP® system. The geometry of the twin half-cell model developed by Terracon (2006c) using $FLAC^{3D}$ was considered the most suitable representation for the OCSP® structure and was selected as the conceptual base model. The geometry and the basic elements of the twin half-cell representation for the OCSP® system are shown in Figure 7.1-1. For expediency, development of the initial test model was restricted to recreating the basic elements in the geometry of the twin half-cell with no attempt to duplicate the analysis results reported by Terracon. Once developed and checked, this model was referred to as the primary model. Details of the primary model are discussed in Section 7.2.
- **As-Designed and As-Built Models**. Following the initial test model development, the numerical modeling effort focused on the refinement of the initial test model for use in the as-designed and as-built analyses. The implementation of the model in the as-designed evaluation is described in Section 7.3, and the implementation for the as-built evaluation is described in Section 7.4.
- **Local Model.** Additional models were also developed to investigate the pullout mechanisms of the OCSP® tailwall and the effect of construction defects on system performance. These models, which are called local models, are discussed in Section 7.5.

7.2 Primary Model

The following subsection describes the model used to represent the OCSP® system. This discussion covers the basic model components and features, and then describes the loading cases considered in the model.

7.2.1 Basic Model Components and Description

The OCSP® system is a large plane strain retaining structure consisting of sheet pile walls installed in an arched layout to form facewall elements tied to tailwall sheets. The area bounded by the sheet pile walls in the final layout is ultimately filled with compacted granular backfill to form cells for the wharf structure. The OCSP® wall system has the basic elements of an earth retaining wall system, with facewall sheet piles to retain the backfill and tailwall sheet piles providing reaction to the loads from the facewall. This idealization for the OCSP® system is similar to mechanically stabilized earth (MSE) walls, but with vertically oriented reinforcement elements rather than the horizontally oriented reinforcement elements of an MSE wall system.

7.2.1.1 Sheet Pile-Interlock Load-Deformation Mechanism

Relative to an MSE wall, the OCSP® system has a slightly different behavior when the pullout resistance of the tailwall is mobilized, or when the lateral earth pressure is active behind the wall face. For both cases the deformation behavior in its simplest form is characterized by stretching of the sheet piles followed by load resistance from the engagement of sheet piles at interlocks. Further complication arises in this deformation mechanism of sheet pile-interlock system when other factors such as the confining pressure and the slippage between sheet piles and soil are involved. Figure 7.2-1a shows a typical response for a 1-inch-wide and 10-inch-long section of unconfined PS31 sheet piles with interlocks in axial tensile testing (PND, 2008b). It is evident from this figure that the response has a unique shape with three distinct ranges: (1) a stretching range with no load transfer, (2) an elastic range for the deformation of the sheet piles with interlocks, and (3) an inelastic range when the yielding occurs and the ultimate capacity is reached.

The constitutive material model for the structural elements in FLAC^{3D} assumes non-yielding behavior within the elements, and their mechanical structural response is calculated based on elasticity. Modeling of structural yielding in FLAC^{3D} can be considered in the analysis by using links with deformation laws at the connection between the structural elements. However, for the type of analysis and the model size being used for the evaluation of the OCSP® system, the links impose restrictions on the spatial discretization of the solution domain (that is, the mesh used in the analysis) and require the size of the elements to be small in order to capture the mechanisms associated with soil-structure interaction without significantly affecting the pull-out capacity of the structure elements. To overcome this modeling restriction, an alternative approach is used by adopting a deformation criterion for the structural response to capture the effects observed in the load-displacement behavior of sheet piles with interlocks.

As will be explained later in this section, the deformation criterion relies on the proper selection of structural properties to predict the combined deformation in tension and to idealize the response observed for the load-deformation curve of the sheet pile-interlock system, as shown in Figure 7.2-1b. This simplified modeling approach predicts the deformation of the sheet pile-interlock system with reasonable accuracy, but will over-estimate the load recovered from the element stresses if not adjusted at the given deformation. This adjustment was not included in the FLAC^{3D} primary model used by CH2M HILL, but can be roughly estimated, as shown in Figure 7.2-1b, by matching the load on the actual stretching-load-displacement curve at the displacement predicted by the idealized curve. The FLAC^{3D} primary model included the deformation mechanism for sheet pile interlock in the analysis up to the end of the backfill placement during the construction of the cells, beyond which full stretch of the sheet pile-interlock system is assumed in the analysis for the deformation behavior of these structural elements.

7.2.1.2 Structural Elements

FLAC^{3D} offers a wide range of structural elements for modeling retaining structures, and the selection of which element type is suitable for the analysis of the OCSP® system depends on whether bending or membrane stresses are important in the physical behavior of the structural response. The FLAC^{3D} liner element type DKT-CST was selected for the facewall where both the bending and the membrane stresses are considered important, and the geogrid element type CST was selected for the tailwall where only the membrane stress controls the structural response. These elements are essentially plane stress elements with mechanical behavior that has two components: (1) the structural response of linear elastic material with no failure limit, and (2) the interaction with grid through coupling springs to control the relative movement and the failure limit.

For the structural response of these elements, the material properties were selected as follows:

- **Facewall and Tailwall Sheets.** Elastic properties using Young's Modulus of the steel were selected. These properties were assigned to the facewall and tailwall elements for the calculation of bending stresses.
- **Interlocks.** Elastic properties using secant Young's Modulus from the interlock tensile test results were used. The secant modulus is the slope of the stress-strain curve from the axial tensile test of sheet pile sections with interlocks obtained from the idealized load-displacement curve shown in Figure 7.2-1b.

The elastic properties obtained using the secant modulus were assigned to the facewall and the tailwall elements for the calculation of the membrane stress. Different sheet pile sizes were used for the structure elements in the evaluations, depending on the wall location and whether it was for an as-designed or as-built analysis. For example, PS31 sheet piles were used for the facewall and tailwall for the as-designed model; PS31 and PS27.5 sheet piles were used for the facewall and tailwall, respectively, in the as-built model. The same PS31 sheet piles were assumed for the facewall and tailwall in the as-designed model to allow direct comparison to FLAC^{3D} results obtained by Terracon (2004c). The PS31 tailwall was later changed to a PS27.5 sheet pile by the PND design team, and this smaller sheet was used in the as-built model.

The selection of material properties mentioned above allows the effect of the stretching behavior to be included uniformly along the curved wall face and along the length of the tailwall, without the need to combine this effect at the structural element connection. The modeling method uses anisotropic properties for the shell element to allow in-plane deformation to capture the interlock behavior of the sheet piles of the facewall and the tailwall, and still preserves their actual out-of-plane deformation to capture the bending stiffness normal to the shell element surface for the facewall. This feature was used until the fill was placed, then the property of the shell elements was switched back to the isotropic properties, assuming the sheet piles' interlock of the facewall and the tailwall has fully stretched. Therefore, only the structural elements affected by the applied loads from backfill placement will engage in the stretching behavior of the OSCP system. Table 7.2-1 summarizes the properties selected for the structural elements used in the primary model.

TABLE 7.2-1
Structural Element Material Properties

Stress Type	Thickness (inch)	Young's Modulus[a] (psf)	Poisson's Ratio
Bending	0.5	4.18×10^9	0.2
Membrane	0.5 (Facewall), 0.5 (Tailwall) – as-designed 0.5 (Facewall), 0.4 (Tailwall) – as-built	2.06×10^8	0.2

[a] Young's Modulus used for the membrane stress was derived from the slope of the idealized stretch-load-displacement curve.

For the interaction with the grid, FLAC^{3D} allows the liner and the geogrid elements to interact with the grid using coupling springs at the element-soil interface. This feature allows for modeling the relative movement between the sheet piles and the soil, and limits the ultimate load transfer with a Coulomb-type strength criterion. The stress-displacement relationship and the shear strength criterion for the coupling springs of the structural elements are shown in Figure 7.2-2. Table 7.2-2 summarizes the interface properties selected for the coupling springs of these elements.

TABLE 7.2-2
Structural Element Interface Properties

Element Type	cs_nk (pcf)	cs_sk (pcf)	cs_ncut (psf)	cs_scoh (psf)	cs_scohres (psf)	cs_sfric[a] (degree)
Liner	2.5×10^7	2.5×10^7	0	0	0	30 (as-designed) 34 (as-built)
Geogrid	NA	2.5×10^7	NA	0 (as-designed) 500 (as-built)	NA	30 (as-designed) 34 (as-built)

[a] Shear resistance is assumed equivalent to 0.8 times the tangent friction angle of the soil.

The stiffness values of the coupling springs (that is, cs_nk and cs_sk in Table 7.2-2) in the normal and the tangential directions of the structural elements define the slope of the stress versus the relative displacement shown in Figure 7.2-2, and their orders of magnitude were estimated based on the horizontal modulus of subgrade reaction of the retained soil. When the stiffness values of the coupling springs are selected greater than the stiffness of the surrounding media (that is, the soil and the structural element), a rigid boundary with small interpenetration is created at the soil-structure element interface. Likewise, when the stiffness values for the springs are selected smaller than the stiffness of the surrounding media, a soft boundary with large interpenetration is created. Generally, selection of very high or low stiffness values for the coupling springs should be avoided, and the best selection would be the values that allow for the deformation of the soil and the structural elements to predict the appropriate stiffness for the wall system.

7.2.1.3 Soil Elements and Constitutive Model

The eight-node hexahedral zones with conventional Mohr-Coulomb (MC) constitutive model were selected to represent the soil during the loading stages of the analysis. For the loading conditions under which the soil is subject to drained shear, effective shear strength parameters are appropriate for both coarse-grained and fine grained soils. Under rapid loading conditions for which the rate of porewater pressure generation exceeds the rate of dissipation, undrained shear strength parameters are used for low-permeability soils, while the effective shear strength parameters are used for high-permeability soil.

To include the effect of strength anisotropy of the BCF clay, the shear strength was divided into two regions at the wall face: triaxial extension strength parameters on the sea side, and triaxial compression strength parameters on the land side. This two-zone model was used rather than the three-zone model discussed in Section 5 for computation efficiency. The average strength along a failure surface with the two-zone model was similar to the average strength for the three-zone model. This similarity in average strength for like failure surface produces equivalent loading conditions. Comparisons between limit-equilibrium FS values and FS values from c-ϕ reduction methods in FLAC^{3D} were comparable, suggesting again that the two-zone model was reasonable.

The undrained shear strength was assigned to the soil elements as a function of the effective vertical stress and the OCR of the soil. Selection of the strength parameters for the backfill soil and the BCF clay during loading stages was made based on the interpreted soil properties for the selected cell cross-sections and the construction time of the OSCP® system, as discussed below:

- **Short-term Static Loading.** This is the end-of-construction analysis, and the shear-strength parameters for the BCF clay are based on the short-term consolidated undrained (CU) shear strength after the construction of the dike and the placement of the common fill for the backland area. Based on construction and instrumentation records, it was assumed that the BCF clay beneath the backfill will have fully consolidated under the backfill load. This case is applicable when the current dredge elevation is deepened to the final dredge depth. A live load of 200 psf is included in the strength determination to account for construction loads that are likely to exist. Effective shear strength parameters are used for the granular backfill soil.
- **Long-term Static Loading.** The effective shear strength parameters and the long-term CU shear strength are used for the drained and the undrained shearing of the BCF clay, respectively. In this case, the full weight of the live load is added to the backfill load to simulate an operational loading condition. The load for the undrained case is associated with a low tide. The assumption is also made that the rate of loading and the rate of soil failure will be rapid, precluding dissipation in porewater pressures. Effective shear strength parameters are used for the granular backfill soil.
- **Seismic Loading.** In the seismic analysis the shear-strength parameters are based on the long-term CU shear strength for the BCF clay and the effective shear strength parameters for the granular backfill soil. However, as discussed in Section 5, the undrained cyclic strength of the BCF clay is taken as 0.9 times the long-term CU strength, assuming a reduction in the shear strength due to the cyclic straining. For granular soil susceptible to liquefaction, the effective friction angle was reduced to include the effect of the strength loss caused by cyclic liquefaction. The reduced friction angle used for these soils in the seismic analysis was equivalent to the liquefaction shear strength, and was estimated from the ratio of the residual shear resistance to the effective vertical stress. The reduction in shear strength for the BCF clay was only considered in the evaluation of the as-built conditions. No strength reduction was considered in the evaluation of the as-designed conditions. A 200-psf live load was included in the strength determination to account for potential container storage in the area.

The elastic properties used for the deformation modulus of the BCF clay in the static analysis were selected from the ratio between the undrained modulus and the undrained shear strength, and are assigned to the soil elements as a function of the effective vertical stress and the OCR. The undrained modulus is used for the short-term static load conditions, and the drained modulus is used for the long-term load condition. In the seismic analysis, modulus-reduction curves are assigned to the soil elements using the FLAC^{3D} sigmoidal model (sig4) to include the effect of the soil response to cyclic straining. The parameters for the sig4 model were selected to match the shape of the Derendeli (2001) modulus-reduction curves using the PI of the soil layers. The small strain-shear modulus for the soil elements used in the dynamic analysis is based on the interpreted shear wave velocity at the site.

7.2.1.4 Free-standing Water and Groundwater

Free-standing water on the sea side is included in the analysis as a uniform pressure boundary applied normal to the top surface of the model on the sea side, and as a pressure boundary varying linearly with depth applied normal to the wall face. A unit weight equivalent to 64 pcf was used for the water during calculation of the effective stresses of the soil elements and the pressure boundaries of the free-standing water.

Groundwater was included in the analysis as hydrostatic pressure without modeling the effects of the seepage pressures. Hydrodynamic forces were determined following Westergaard's method (Westergaard, 1933; Kramer, 1996). See Section 7.2.2.3 for further discussion.

7.2.1.5 Boundary Conditions

The sections analyzed in FLAC^{3D} are representative of an infinitely long OSCP® wharf structure which would exhibit plane-strain conditions, that is, all displacements occur in the plane perpendicular to the long axis of the structure. In this idealization of the structure the materials, geometry, construction sequence, and loading are identical at every cell. Each FLAC^{3D} model developed for the analysis of the OCSP® is a slice from the idealized structure and spans from the center of one open cell to the center of an adjacent cell, and thus includes one tailwall. Fixed-boundary conditions were applied to the mesh points of the soil elements on the external boundaries to restrain the translational degree-of-freedom (DOF) in the normal direction consistent with the assumed plane-strain condition. These boundary conditions allowed the twin half-cell model to include the effect of the large lateral extension of the OCSP® structure in the analysis.

For the structural elements, boundary conditions were assigned to the nodes as follows:

- **Facewall (Liner Elements)**. At the nodal points located along the centerline of the cells a fixed boundary condition is used to restrain the translational DOF normal to the outer boundary of the model, and a fixed boundary condition is used to restrain rotation around the vertical axis. All other nodes for the liner elements are unrestrained.
- **Tailwall (Geogrid Elements).** Free boundary conditions were assigned to all the nodes for the geogrid element in the three translational DOF.

At the wye connection, the DOF for the nodes of the liner and the geogrid elements were slaved together using rigid links to connect the facewall and the tailwall elements in all directions except the vertical, where free movement is allowed between the facewall and the tailwall. In the as-built evaluation, additional continuum elements were constructed at the top of the wall face with linear elastic properties selected to match those of the utilidor.

7.2.2 Model Loading Stages

The loading stages included in the FLAC^{3D} analyses were based on the construction sequence for the dike, backland, and the OCSP® wall installation. These stages were simulated in the analysis in an effort to replicate the key features of the actual loading during construction. For ease of reference to the position of a particular construction or loading stage, all elevations in the numerical models were measured relative to MLLW. The loading stages are shown in Figure 7.2-3, and included the following steps (the elevations in parentheses are those used for the as-designed and the as-built case evaluations, respectively):

1. Establish the initial in situ total stresses for the offshore pre-construction conditions of the BCF clay between elevations (-30, -19) and -150 feet MLLW with seawater at elevation -5 feet MLLW.
2. Dredge the clayey silt sediments between elevations (-30, -19) and (-51, -41) feet MLLW and replace it with granular backfill.
3. Apply load increments for the placement of the granular backfill to complete the construction of the dike between elevation (-30, -19) and +30 feet MLLW.
4. Apply load increments for the placement of the common fill on the land side to complete the construction of the backland between elevation (-30, -19) and (+30, +24) feet MLLW.
5. Apply load increments for the placement of the granular backfill between the dike and the OCSP® wall back face between elevations (-30, -19) and +30 feet MLLW followed by the placement of the backfill cap between elevation (+30, +24) and (+35, +38) feet MLLW. The end of this loading stage completes the construction of the OCSP® system.
6. Dredge offshore at the mudline to increase the depth of water adjacent to the wharf structure from elevation (-30, -19) to -51 feet MLLW.

7.2.2.1 Short-Term Static Loading

The short-term static analysis represents the end-of-construction state, and the wall deformation is estimated assuming short-term, undrained loading conditions. In this analysis the undrained shear strength of the BCF clay is assumed to correspond to the consolidated strength under the backfill weight of the dike and the backland area. The analysis involved loading stages of steps 1 through 4, as described above, then resetting the deformations of the structural and the soil elements to zero, and then continuing the analysis with loading stages of steps 5 and 6.

The elevation of the water level on the land side behind the wall was increased from elevation -5 to +20 feet MLLW between step 5 and 6, and a live load of 200 psf was added to the model at the top of the backfill behind the wall. The 200 psf live load is included to represent construction vehicles and other equipment that could occur immediately after the backfill reaches elevation +35 feet MLLW.

7.2.2.2 Long-Term Static Loading

The long-term, static analysis represents the state where the primary consolidation of the BCF clay has been completed following construction, and wall deformations are estimated under the long-term drained and undrained loading conditions. For the long-term drained analysis, following the end of loading stage of step 6, the shear strength parameters for BCF clay are changed from the short-term undrained parameters to the effective stress parameters. Similarly, the BCF clay strength parameters are changed to the long-term undrained parameters at the end of loading stage 6 to estimate the long-term undrained condition.

After the material parameter changes are made, the elevation of the water level on the sea side is increased from -5 to +7.5 feet MLLW, and a live load of 1,000 psf is added to the model at the top of the backfill behind the wall to represent maximum container and other planned operational loads.

7.2.2.3 Seismic Loading

The initial state for the dynamic analysis is taken as the end of the static long-term loading. The displacements for the soil mesh points and the structural nodes are reset to zero to define a reference for the additional deformation resulting from the seismic loading.

Two main assumptions are made for estimating the wall deformations under the seismic loading conditions:

- The seismic loading during an earthquake will be rapid and the shear strength of the BCF clay mobilized is the long-term consolidated undrained shear strength. As stated earlier, the long-term shear strength used in the seismic analysis is reduced by 10 percent (that is, 0.9 S_u) in the as-built case evaluation to account for the effect of the cyclic straining. As discussed in Section 5, a much larger reduction in undrained strength may be warranted based on results of ring shear tests conducted as part of this project; however, the reduction in strength for the FLAC^{3D} analyses was limited for the as-built analyses to evaluate response with FLAC^{3D} under optimistic strength assumptions.

- Based on the cyclic simple shear test results, significant porewater pressure generation in the BCF clay during the seismic loading is unlikely as long as limited deformations occur; therefore, no porewater pressure generation is considered in the seismic analysis. As noted in the previous comment, this assumption was believed to be optimistic, based on ring shear tests conducted as part of the project. However, by evaluating deformations under optimistic conditions, the severity of cyclic loading and large permanent deformations could be identified and a decision on whether to use a more complex degrading strength model made.

Additionally, the seismic deformation analyses conducted for the as-designed case and the as-built case included consideration of the following loads:

- The effect of the tidal variation by increasing the water level on the sea side from elevation -5 to +7.5 feet MLLW.
- The effect of the hydrodynamic pressure, which is included in the analysis as an added mass to the vertical face of the wall using Westergaard (1933) pressure distribution for a depth of 58.5 feet.
- The addition of live load of 200 psf at the top of the backfill behind the wall for seismic loading in the as-built case. This load is defined as 20 percent of the 1,000 psf live load. Section 2 provides the rationale for selecting 20 percent of the live load. For the as-designed case, the live load was specified as zero, consistent with the approach taken by the original designers.

In the dynamic analysis, a free-field boundary condition is applied to the side boundaries of the model mesh to absorb the reflected horizontal waves, and a compliant base was used at elevation -150 feet MLLW of the model to apply the ground motions for the design events. This combination of boundaries approximately provides a source of radiation damping and is used to simulate the effect of infinite media while reflecting the influence of the media underlying the base of the model on the ground motion used for the analysis.

The absorbing boundaries along with the hysteretic damping are the primary damping mechanism in the dynamic analysis. A small amount of Rayleigh type damping is also used in order to reduce the effect of high-frequency modes. A critical damping ratio of 0.07 percent was used for Rayleigh damping and is applied at frequencies of 1.07 (seaside) and 1.4 (landside) Hertz. These frequencies were estimated from the fundamental site period, and a period for a higher mode was taken as five times the fundamental site period. In the seismic deformation analyses, the outcrop ground motions at elevation -150 feet MLLW, shown on Figures 7.2-4 through 7.2-6, for the OLE, CLE, and MCE seismic events, respectively, are used and input as a shear stress boundary condition at the base of the model. The shear stress applied at the base is calculated from the shear wave velocities at the base and half the amplitude of the outcrop motion for the design events.

7.3 As-Designed Evaluation

The intent of the as-designed evaluation is to develop a numerical model using FLAC^{3D} for one of the representative sections in the North Expansion (PND, 2008b) to verify that the original design assumptions were adequate and that the estimated deformations are generally consistent with those obtained during the original design. For this purpose, the tallest wall section of the OCSP® system in the North Extension, Section F, was selected and analyzed. A typical cross-section for as-designed case Section F is shown in Figure 7.3-1.

Although the intent of the as-designed evaluation was to model the methods used by the PND team as closely as possible, some assumptions used for the as-designed analysis differ from those reported by PND in their 2008 report, as follows:

- The CH2M HILL analysis is based on using a three-dimensional analysis, compared to the two-dimensional analysis used for most of the PND evaluations.
- The tailwall geometry in the current analysis differs slightly, as noted in Section 7.3.1, to accommodate as many details for Section F as possible with a simplified geometry for the analysis.
- The tailwall used a PS31 sheet pile rather than a PS27.5 sheet pile to match FLAC^{3D} analyses conducted by Terracon (2004c).
- The depth of the BCF clay on the land side is taken at elevation -51 feet MLLW compared to -50 feet MLLW in PND's report.
- The elevation for the tidal level on the sea side is at elevation +7.5 feet MLLW for the long-term static and seismic analysis compared to elevation +11.5 feet MLLW in PND's report, and on the land side is at elevation +20 MLLW compared to +18 feet MLLW. These changes were made after discussing PND's as-designed assumptions with USACE and concluding that the changes should be made to reflect more likely conditions. The effects of other water levels are evaluated in the sensitivity analysis included in Section 5.3.
- The undrained shear strength variation of BCF clay with depth is approximated in this FLAC^{3D} analysis with a SHANSEP function. The parameters for the shear strength function were selected to provide a shear strength profile similar to the shear strength profile used by the original designer.
- The modulus variation of BCF clay with depth used in the CH2M HILL FLAC^{3D} static analysis is a function of the undrained shear strength and approximates the modulus profile used by the original designer.
- The low-strain shear modulus used for the dynamic analysis is based on the interpreted shear wave velocities reviewed for the current evaluation.

The idealized wall geometry, soil layers, water level, and dredge depth are as described in Section 7.3.1. The estimated permanent seismic deformation in CH2M HILL's FLAC^{3D} analyses is based on the maximum response of the OCSP® wall using the ground motions of at least three earthquake records in the CLE and the MCE evaluations, and two earthquake records in the OLE evaluation.

7.3.1 Model Description

The FLAC^{3D} model for the as-designed analysis is 689 feet long, 27.5 feet wide, and extends from elevation -150 feet MLLW to +35 feet MLLW. The geometry of the model is as follows:

- The model length includes a lateral distance from the wall face to the side boundaries of 197 feet on the sea side and 492 feet on the land side. These distances were selected to be far enough from the facewall to minimize the boundary effects on the results of the analysis.
- The model base was selected as the boundary between the BCF clay and the underlying dense sand.
- The facewall for the section analyzed has a total height of 90 feet extending from elevation +30 feet MLLW at the top of the sheet pile and elevation -60 feet MLLW at the tip of the sheet pile.
- The tailwall is 85 feet long (compared to 83 feet in PND 2008b, Section F) and extends between elevation 25 feet MLLW and -51 feet MLLW. The tailwall extension is 96 feet long (compared to 98 feet in PND 2008b, Section F) and extends between elevation 20 feet and -10 feet.
- The dredge limits at the base of the wall extend a distance of 105 feet on the land side from the wye connection.
- The side slopes of the granular fill dike are assumed to have a slope of 1.5H:1V (horizontal:vertical) on both sides, with the landside hinge point at a distance of 200 feet from the wye connection.

The as-designed FLAC^{3D} model has the same basic element types as described in Section 7.2.1.1 and Section 7.2.1.2 for modeling the facewall, tailwall, and soil. Figure 7.3-2 shows the mesh size and the soil zones used for the evaluation of the as-designed conditions. The soil properties used in the analyses are very similar to those presented by PND in their 2008 report. Table 7.3-1 summarizes the soil properties used for the as-designed case evaluation.

TABLE 7.3-1
Material Properties for As-Designed Evaluation

Elevation (ft)				Effective Shear Strength Parameters						
From	To	Soil Type	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degree)	Undrained Shear Strength (psf)	Drained Poisson's Ratio	E (ksf)	Shear Wave Velocity (fps)	Modulus Reduction Curve[a]
+35	-30	Granular Backfill	130	0	36	-	0.3	1400[b]	1000	PI=0
		Common Backfill	120	0	32	-	0.3	900[b]	1000	PI=0
-30	-51	Granular Backfill	130	0	36	-	0.3	1400[b]	1000	PI=0
		Silt Sediments	120	0	32	-	0.3	700b	1600 – 1700	PI=0
-51	-150	BCF	120	400	27	2100 (23)[c] 2800 (18)[c]	0.35	1200 (9)[d] 2400 (11)[d]	600 – 970	PI=15

[a] Modulus reduction curve is based on Derendeli 2001 for the given PI (plasticity index).
[b] Average value to reflect the modulus variation with depth.
[c] Undrained shear strength on land side is given for short-term and long-term, respectively. Value in parenthesis is the increase per one foot of depth below elevation -50 feet. Undrained shear strength for the sea side is 1,600 psf with 17 psf increase per one foot of depth below elevation -50 feet for short-term and long-term loading.
[d] Young's Modulus is given for short-term and long-term, respectively. Value in parenthesis is the increase per one foot of depth below elevation -50 feet.

The stages for the static and dynamic analyses of the as-designed model are as described in Sections 7.2.1.6 through 7.2.1.9. Dredging in the as-designed model is between elevations -30 MLLW and -51 feet MLLW. The water levels considered for the short-term loading conditions were at elevation -5 feet MLLW on the sea side and +20 feet MLLW on the land side, and for long-term static and seismic were at elevation +7.5 feet MLLW on the sea side and +20 feet MLLW on the land side. The live load ground surcharge for the long-term static loading conditions is 200 psf for a distance of about 200 feet from the face of the wall and then 1,000 psf to the landside model boundary.

7.3.2 Results of FLAC^{3D} Analyses

The results from CH2M HILL's FLAC^{3D} analyses for the as-designed case are presented for the static and the dynamic loading conditions in Table 7.3-2, and the plots for the predicted stresses and displacements in the structural elements and soil zones are shown in Figures 7.3-3 through 7.3-17. Results from the static and seismic analyses are briefly reviewed below.

7.3.2.1 Static Analyses

The global stability of the as-designed model was initially evaluated by defining the factor of safety (FS) for short-term and long-term loading conditions. This evaluation was performed to examine the shear strength profile for the BCF clay zones and the boundary conditions used in the static analyses and to compare to the original designer's FS values. The global stability evaluation involved searching for the minimum FS at the end of each loading case using FLAC^{3D} c-phi reduction technique. This method of evaluation is described in Itasca (2009).

The results show that the static FS values vary between 1.2 and 1.4 for the static short-term and long-term loading conditions, respectively. These values are slightly lower than those obtained by the PND design team in Table 6-2 of PND (2008b), where PND reports FS values of 1.3 and 1.5 for short-term static and long-term static loading, respectively. The difference most likely represents a combination of effects from the limit-equilibrium methods used by PND versus the three-dimensional methods used by CH2M HILL, different assumptions regarding the shape of the failure surface, and other small differences in modeling assumptions as discussed above.

TABLE 7.3-2
Summary of As-Designed FLAC^{3D} Analysis

Parameter		Short-Term Static [a]	Long-Term Static Drained	Long-Term Static Undrained	Seismic		
					OLE	CLE	MCE
Maximum Membrane Stress on Facewall							
Horizontal Stress (ksi)[b]	Tension	11.8	9.4	9.4	15.3	32.3	40.3
	Compression	7.6	6.6	6.6	6.9	9	8.3
	Allowable Horizontal Tensile Stress	26.7	20.0	20.0	30.8	36.4	40.0
Maximum Membrane Stress on Tailwall[c]							
Horizontal Stress (ksi)	Tension	13.9	13.2	13.5	27.1	30.6	33.3
	Compression	-	-	-	-	-	6.9
	Allowable Horizontal Tensile Stress	26.7	20.0	20.0	30.8	36.4	40.0
Factor-of-Safety[d]		1.2	1.4	1.4			
Facewall Maximum Horizontal Displacement[e] (inches)		10	10	10	2 – 14	20 – 120[f]	93 – >120
Tailwall Maximum Horizontal Displacement (inches)		9	9	9	-	-	-
Maximum Vertical Settlement (inches)		5	5	5	-	-	-

[a] Short-term static includes harbor dredge to Elevation -51 feet.
[b] Membrane horizontal stress along the curved face of the wall.
[c] Stresses in tailwall based on PS31 sheet pile. For PS27.5 sheet pile adjust stress based on difference in wall thickness (0.5 inch versus 0.4 inch for PS31 and PS27.5 sheet piles, respectively).
[d] Global stability factor-of-safety.
[e] Displacement at the top of the wall. For seismic, displacement is permanent wall deformation at the top of the wall.
[f] Estimated range does not include Synthetic EQ record. Permanent wall deformation from the Synthetic EQ record exceeded 120 inches.
OLE = operating level earthquake
CLE = contingency level earthquake
MCE = maximum considered earthquake

For the static loading conditions, results from CH2M HILL's FLAC^{3D} analyses show that lateral wall movements at the top of the facewall sheet piles are about 10 inches, with no significant difference between the lateral movements at the wye connection compared to the lateral movements at the cell centerline. The maximum lateral movement for the facewall is about 10 inches and generally occurred near the cell centerline at the original mudline before the dredging in this analysis. The corresponding lateral movement for the wye connection at the maximum movement was about 9 inches. This behavior is consistently observed for all the loading conditions considered in the analyses. The 9-inch lateral wall movement at the wye connection includes the sheet pile interlock stretch and deflection.

Section 6 shows that the allowable horizontal sheet pile tensile stress is controlled by the sheet pile interlock strength, and can be calculated by dividing the ultimate interlock strength of 20 kips/inch by the sheet pile wall thickness, and then by the appropriate FS for each loading case (Table 2.2-2). For the as-designed model, with facewall and tailwall thickness of 0.5 inch, the allowable horizontal tensile stress for the long-term static case is 20 ksi for both the facewall and the tailwall (20 kips/inch / 0.5 inch / 2.0 = 20 ksi). Similarly, the allowable tensile stress for the short-term static case can be calculated to be approximately 26.7 ksi, and the allowable horizontal tensile stress for the OLE, CLE, and MCE cases are approximately 30.8 ksi, 36.4 ksi, and 40 ksi, respectively.

Based on the FLAC^{3D} results, the maximum membrane tensile stresses developed in the facewall and the tailwall under the static loading conditions are about 12 and 14 ksi, respectively. According to the idealized load-displacement curve in Figure 7.2-1b, the actual stresses at the sheet pile interlocks in the facewall and the tailwall are expected to be somewhat lower than these values. Using these values as a conservative upper bound, the estimated sheet pile horizontal stresses are well below the allowable tensile stress limits for the short- and long-term static cases. The location of the maximum membrane stresses in the facewall and the tailwall are observed to occur near the original mudline prior to the dredging. It is also observed that the zone of the maximum membrane stresses in the tailwall are developed along a line extending between a point near the wye connection at the original mudline before dredging, and near the tailwall centerline at the top of the wall.

7.3.2.2 Seismic Analyses

The lateral permanent wall movements of the sheet pile walls for the seismic loading range between 2 and 14 inches at the end of strong motion for the OLE, between 20 and 120 inches for the CLE, and between 93 and greater than 120 inches for the MCE. Figures 7.3-18 through 7.3-23 show the x-displacement-time histories for the three design earthquake ground motions. In all three cases, a sliding block was developed along a non-circular failure surface extending between the toe of the facewall and near the rear end of the tailwall extension. Additional contour plots of stresses and displacement in the structural elements and the soil zones recorded at different times during the seismic shaking are shown in Appendix F.

The estimated permanent lateral deformations based on the results of CH2M HILL's seismic analyses exceed the threshold for the maximum permanent deformation of the design criteria for all three design level earthquakes. Estimated ground displacements for the three levels of earthquake loading are also significantly higher than those estimated by PND, particularly for upper bound values. For example, upper bound values of displacement reported in Appendix N of PND (2008b) are approximately 25 inches compared to more than 120 inches found by CH2M HILL. This large difference in displacement estimates is attributed to the larger tidal elevation difference used by CH2M HILL (for example, 11.5-foot difference used by CH2M HILL versus 5-foot difference used by PND), differences in the two- and three-dimensional modeling, and differences in some of the boundary condition assumptions.

It should be noted that membrane stresses in Table 7.3-2 excluded some inconsistencies for the tailwall stress during seismic loading, where the tension stresses for the OLE case are higher than those estimated for the CLE case. This inconsistency is caused by stress concentrations near the tailwall/tailwall extension. At this location there are only a small number of elements that resulted in unreasonably high stresses. This type of problem could have been remedied with a finer mesh, or perhaps by introducing some structural elements damping.

After correcting for the inconsistency mentioned above, the maximum membrane stresses for the OLE, CLE, and MCE cases are generally smaller than the allowable horizontal tensile stress, as shown in Table 7.3-2. The maximum interlock stresses were observed to occur in the structural elements between the tailwall and the tailwall extension.

7.4 As-Built Evaluation

The numerical model for the as-built evaluation is intended to establish OCSP® system performance in the North Expansion considering the interpreted wall-specific soil profile, the layout of the sheet piles and their depth of penetration, imposed seismic ground motions, and modeling (to the extent possible) of the effects of the defective sheet piles as they exist at the site. For this purpose, Section 2-2 (see Section 5 of this report) was selected as a representative section to evaluate the impact on the deformation of the structure for the as-built condition. A typical cross-section for the as-built case Section 2-2 is shown in Figure 5.1-3. No attempt is made in these as-built evaluations to account for sheet piles that are out of interlock. This issue is dealt with in Section 7.5.

7.4.1 Differences between As-Designed and As-Built Modeling

There are differences between the as-designed and the as-built models, as summarized below:

- The soil profile for the as-built case is interpreted at the location of Section 2-2, where the clayey silt sediments are located between elevations -19 and -41 feet MLLW compared to between elevations -30 and -51 feet MLLW in the as-designed case.
- The shear strength profile of the BCF clay and the backfill granular fill for the as-built condition is different than the shear strength profile used for the as-designed condition.
- The dredge limits in the as-built condition are between elevations -19 and -51 feet MLLW, whereas for the as-designed condition, dredging lowers the seafloor from elevation -30 to -51 feet MLLW.
- The live load representing backlands operations was modified for both gravity and seismic loading conditions to be a constant 1,000 psf for gravity loading and 200 psf for seismic loading. These values compare to reduced loading of 200 psf within 200 feet of the bulkhead for gravity loading and no live loads for seismic events during the as-designed case.

The differences noted above are considered important parameters to consider in the analysis because they are better representations of actual operating conditions and because they provide additional insight into the response of the OCSP® system in terms of the stresses and displacement.

7.4.2 As-Built Structural Defects

The predominant structural defects in the installed OCSP® walls include damaged tips and broken interlocks that occurred during installation of the sheet piles. The potential effect of damaged tips was not considered in the design as this construction issue was not anticipated. Realizing that damage has occurred to the sheets, concerns have been raised about the initiation of progressive failure in the sheet piles along interlock connections during gravity or seismic loading.

From the numerical modeling standpoint, these structural defects are difficult to include in the numerical model unless simplifying assumptions are made to include their effects in a practical way. In addition, the twin half-cell model of the primary model is a symmetric model, and introducing the defective structural elements implies the repetition of these defects in the actual OCSP® structure. Therefore, the evaluation of the as-built condition did not include the structural defects in the global stability model. Instead, the effects of sheet pile defects were addressed with local models described in Section 7.5.

7.4.3 As-Built Model Description

The as-built model dimensions and boundary conditions are the same as the as-designed model; however, the geometry differs from the as-designed model in the following ways:

- The facewall for the section analyzed has a total height of 91 feet extending from elevation +30 feet MLLW at the top to elevation -61 feet MLLW at the tip of the sheet piles.
- The tailwall has a width of 82 feet, and extends between elevation 24 feet MLLW at the top to elevation -41 feet MLLW at the tip. The tailwall extension has a length of 82 feet and extends between elevation 20 feet MLLW and elevation -10 feet MLLW.
- The sheet pile tip elevation at the end of the tailwall gradually changes in elevation from -41 to -10 feet MLLW within a distance of about 20 feet.
- The dredge limits at the base of the wall extend a distance of 105 feet on the land side from the wye-connection.

The as-built FLAC^{3D} model has the same basic element types as described in Section 7.2.1.1 and Section 7.2.1.2 for modeling the facewall, tailwall, and soil. Figure 7.4-1 shows the mesh size and the soil zones used for the evaluation of the as-built conditions. The soil parameters used in the analyses are summarized in Table 7.4-1.

TABLE 7.4-1
Material Properties Selected by CH2M HILL for As-Built Evaluation

Elevation (ft)				Effective Shear Strength Parameters						
From	**To**	**Soil Type**	**Unit Weight (pcf)**	**Cohesion (psf)**	**Friction Angle (degree)**	**Undrained Shear Strength (psf)**	**Poisson's Ratio**	**E (ksf)**	**Shear Wave Velocity (fps)**	**Modulus Reduction Curve[a]**
+38	-19	Granular Backfill	130	0 (SA) 100 (DA)	40	-	0.3	1400	1100 – 1700	PI=0
		Common Backfill	120	0	32 (SA) 27 (DA)	-	0.3	900	950 – 1100[b]	PI=0
-19	-41	Silt Sediments	120	0	32	2600	0.3	700	1600 – 1700	PI=0
-41	-150	BCF	120	0	30	SHANSEP[c]	0.35	TX[d]	920 – 1250 (L) 350 – 1025 (S)	PI=15

[a] Modulus reduction curve is based on Derendeli (2001) for the given PI (plasticity index).
[b] Shear wave velocity is based on using low-strain shear modulus equivalent to 50 percent the low-strain shear modulus for granular fill.
[c] Undrained shear strength variation with depth is approximated with SHANSEP function to match the strength profile given by equation 5.1-2.
[d] The modulus variation with depth is selected as a ratio of the undrained shear strength profile to match the values based on the triaxial shear as given in Table 5.1-3.
SA = Static Analysis
DA = Dynamic Analysis
L = Land side
S = Sea side

The static and dynamic analyses for the as-built-built model generally follow the modeling procedures described in Sections 7.2.1.6 through 7.2.1.9. The dredge depths in the as-built model are between elevations -19 and -51 feet MLLW. The water levels considered for the short-term loading conditions are at elevation -5 feet MLLW on the sea side and +20 feet MLLW on the land side, and for long-term static and seismic are at elevation +7.5 feet MLLW on the sea side and elevation +20 feet MLLW on the land side. A live load ground surcharge of 200 psf is used for the short-term static condition, while 1,000 psf is used for the long-term static conditions. For the seismic evaluation, a live load ground surcharge of 200 psf is used in the analysis.

7.4.4 Results of FLAC^{3D} Analyses

Results of CH2M HILL's FLAC^{3D} analyses for the as-built case are presented for static and dynamic loading conditions in Table 7.4-2, and plots for predicted stresses and displacements in structural elements and soil zones are shown in the Figures 7.4-2 through 7.4-16.

The global stability of the as-built model was initially evaluated by defining the FS for short-term and long-term loading conditions. This evaluation was performed to examine the shear strength profile for the BCF clay zones and the boundary conditions used in the static analyses. The global stability evaluation involved searching for the minimum FS at the end of each loading case using FLAC^{3D} c-phi reduction technique. This method of evaluation is described in Itasca (2009). The results indicate that the FS values vary between 1.1 and 1.2 for the static undrained and drained conditions, respectively. A detailed comparison between FLAC^{3D} results and the results from the limit-equilibrium methods previously described in Section 5 is addressed in Section 7.6.

For the static loading conditions, the lateral wall movements at the top of the sheet pile walls range between approximately 10 and 14 inches, with no significant difference between the lateral movements at the wye connection compared to the lateral movements at the cell centerline. The maximum lateral movement for the facewall is about 14 inches and generally occurred near the cell centerline at elevation -1 foot MLLW for the long-term, undrained loading conditions. The corresponding lateral movement for the wye connection at the maximum movement is about 12 inches. This behavior was consistently observed for all the static loading conditions considered in the analyses. The amount of horizontal movement is less than 2 percent of the exposed height of the OCSP® facewall, and therefore not considered unusual for such a high, flexible wall.

TABLE 7.4-2
Summary of As-Built FLAC^{3D} Analysis

Parameter		Short-Term Static[a]	Long-Term Static Drained	Long-Term Static Undrained	Seismic		
					OLE	CLE	MCE
1. Maximum Membrane Stress on Facewall							
Horizontal Stress (ksi)[b]	Tension	11.5	17.7	16.3	23.6	35	38.9
	Compression	-	-	-	-	-	-
	Allowable Horizontal Tensile Stress	26.7	20.0	20.0	30.8	36.4	40.0
2. Maximum Membrane Stress on Tailwall							
Horizontal Stress (ksf)	Tension	25	28.8	28.8	31.3[c]	41.7[c]	43.8[c]
	Compression	-	-	-	-	-	-
	Allowable Horizontal Tensile Stress	33.3	25.0	25.0	38.5	45.5	50
3. Factor-of-Safety[d]		1.1	1.2	1.1			
4. Facewall Maximum Horizontal Displacement (inch)		10	12	14	10 - 20	82 - > 120	> 120
5. Tailwall Maximum Horizontal Displacement (inch)		9	11	13	-	-	-
6. Maximum Vertical Settlement (inch)		9	11	11	-	-	-

[a] Short-term static includes harbor dredging to elevation -51 feet MLLW.
[b] Membrane horizontal stress within the curved face of the wall.
[c] Localized stress concentration effects from few elements are excluded.
[d] Global stability factor-of-safety.
OLE = operating level earthquake
CLE = contingency level earthquake
MCE = maximum considered earthquake

The allowable horizontal tensile stresses for the long-term static case are 20 ksi for the facewall and 25 ksi for the tailwall based on a facewall thickness of 0.5 inch and a tailwall thickness of 0.4 inch. The allowable horizontal tensile stresses for other load cases are shown in Table 7.4-2. Based on the results presented in Table 7.4-2, the maximum membrane tensile stresses developed in the facewall and the tailwall under the static loading conditions are about 18 and 29 ksi, respectively. According to the idealized load-displacement curve in Figure 7.2-1b, the actual stresses in the facewall and tailwall sheet piles are expected to be somewhat lower than these values. Using these values as a conservative upper bound, the estimated horizontal stresses in the facewall are within allowable limits. The maximum membrane stresses in the facewall and the tailwall occur near the original mudline prior to the dredging. It is also observed that the zone of the maximum membrane stresses in the tailwall develops along a line extending between a point near the wye connection at the original mudline before dredging, and near the tailwall centerline at the top of the wall.

For the seismic loading conditions, the lateral permanent wall movements at the top of the sheet pile walls at the end of the shaking ranges between 10 and 20 inches for the OLE, between 82 and greater than 120 inches for the CLE, and greater than 120 inches for the MCE. The maximum horizontal tensile stresses at the facewall are generally within allowable limits for all three cases.

The stresses computed for the tailwall exceed allowable levels for several cases. These exceedances are the result of stress concentrations caused by mesh sizes used in the FLAC^{3D} modeling. If the high stress caused by local stress concentration is discarded, the tailwall stress are also within the allowable limits.

The undrained shear strength of the BCF clay on the sea side was the primary difference from the as-designed case that affected the magnitude of the permanent deformation in the FLAC^{3D} analyses. Figures 7.4-17 through 7.4-23 show the x-displacement-time histories for the three design earthquake ground motions. In all three cases, a sliding block develops along a non-circular failure surface extending between the toe of the facewall and through the tailwall extension. Additional snapshot plots for the stresses and displacement contours in the structural elements and the soil zones recorded at different times during the seismic shaking are shown in Appendix F.

7.4.5 Performance Evaluation Relative to Design Criteria

The estimated maximum deformations of the OCSP® system for the as-built case are shown in Table 7.4-2 and are summarized as follows:

- **Static Loading**. The estimated static deformations range between 14 and 10 inches for long- and short-term static loading conditions, respectively. The deformation estimate for the short-term and the long-term static loading condition is less than the threshold of 18 inches for the maximum lateral displacement defined in the design criteria for static loading conditions. The interlock tension stresses in the facewall and the tailwall are generally observed to have FS values above the requirement.
- **Seismic Loading**. The estimated permanent lateral deformations based on the results of the seismic analyses indicate that the threshold for the maximum permanent deformation in the design criteria is likely to be exceeded for all three design level earthquakes. The estimated permanent seismic deformation is based on the maximum response of the OCSP® system using the ground motions of at least two earthquake records for the OLE, CLE, and the MCE events. The FS for the interlock tension stresses were generally greater than 1.0, except in localized areas near the tip of the tailwall sheet piles during the OLE, CLE, and MCE events.

Based on the observed behavior of the as-built OCSP® system during the seismic loading, it appears that larger magnitudes of the interlock stresses are likely to develop in the tailwall sheet piles than in the facewall sheet piles.

7.4.6 Comparison of As-Built Model Performance to Geotechnical Instrumentation Data

The results obtained from the analysis of the as-built model were compared with the instrumentation data for Cell #61 (Terracon, 2011). In Figure 7.4-23, displacement measurements from the in-place inclinometer installed on the interior face of the wall were plotted versus elevation, along with the calculated deflected shape from the FLAC^{3D} analysis of the as-built model. In this comparison, the calculated wall deflection was referenced to a fixed point at elevation -45 feet MLLW before dredging to elevation -51 feet MLLW.

The comparison between the recorded and estimated deformations shows that the amount of predicted movement was generally consistent; however, the shape of the settlement differed. The primary reason for the difference was the elevation and stage of loading for the comparison. The inclinometer provided information before the fill was built above elevation +5 MLLW, while the FLAC^{3D} estimate continued to elevation +30 MLLW. The FLAC^{3D} estimate also included the concrete utilidor that was planned for final design of the wall, which stiffened the top of the OCSP® wall system relative to the inclinometer readings. This stiffening resulted in the different shapes for the two cases.

This comparison between the instrumentation and the FLAC^{3D} was done to confirm general conformance between the two, and therefore a more direct check for the lower fill height and without the utilidor was not made. In the interest of modeling efforts, the comparison between elevations -45 MLLW and -15 MLLW was sufficient to demonstrate the similarity between predicted and observed response.

The predicted vertical displacement from FLAC^{3D} analysis at the end of construction and the long-term static conditions was about 9 and 11 inches, respectively. Although the FLAC^{3D} analysis gave higher settlements, the difference is not viewed as being significant.

7.5 Local Models for Interlock Pullout and Wall Defect Study

Local FLAC^{3D} models were developed to address (1) the contribution of tailwall interlock geometry on pullout resistance, and (2) the effect of defective sheet piles on the performance of the OCSP® system. The interlock evaluation involved using FLAC^{3D} to simulate a physical model test of the interlock knuckle as it was pulled through granular soil, similar to the mechanism that develops resistance at each interlock along the OCSP® tailwall. The defect model considered the effects of defects located at different locations on the facewall and the tailwall. The objective of the defect evaluation was to determine whether a defective sheet pile could "unzip" during future loading, resulting in the rupture of one of the OCSP® system cells. These local models differed from the global stability model described in previous sections by the amount of the OCSP® system included in the evaluation and the type of applied loading.

7.5.1 Tailwall Interlock Pullout Model

One point of uncertainty about the technical design of the OCSP® system was the pullout resistance of tailwalls and the influence of the sheet pile knuckles on this resistance. PND considers the development of interlock resistance to be proprietary information. In the absence of details regarding PND's design basis for the interlock pullout resistance of the knuckles, this issue was investigated in CH2M HILL's suitability study by developing a local model of the tailwall pullout mechanism using FLAC^{3D}. The benefits of investigating performance with a local model using FLAC^{3D}, rather than a physical model, were as follows:

- First, in contrast to physical model tests, numerical modeling does not require the fabrication or purchase of any testing equipment or the labor to form test samples and conduct tests.
- Second, numerical modeling is conducive to parameter studies and investigations of boundary conditions on test results.
- Finally, development of a numerical model specific to the sheet pile interaction with the backfill provides a means to gain insight into how to model this interaction in the primary model.

The FLAC^{3D} model used to investigate the influence of interlocks on pullout resistance was based on pulling connected sheet piles from a box containing soil. Specifically, the model represented a series of connected sheet pile elements 1-inch high that were pulled from a 1-inch-thick box containing soil. The numerical model allowed the sheet piles to pass through openings in opposing walls in the box. The box sides facing the sheet pile elements in the model were open so that the upper and lower parts of the box could be filled with soil.

This model had the ability to apply a uniform normal stress to the surface of the soil on the open sides of the box through a device similar to an inflatable diaphragm, which is held against the two sides of the sheet pile elements. The sheet pile geometry used in the models was based on PS31 sheet pile section geometry. Figure 7.5-1 illustrates the sheet pile section geometry used in the pullout models. Figure 7.5-2 shows a section view of the test device when configured to contain one full sheet pile and two half-sheets.

Three different sheet pile-interlock configurations were investigated:

- Two sheet pile halves
- One full sheet pile joined with two sheet pile halves
- Two full sheet piles with each one connected to the other and a sheet pile half

Figures 7.5-3, 7.5-2, and 7.5-4 illustrate the three configurations, respectively. These configurations are subsequently referred to by the number of interlocks contained in the test specimen; that is, one, two, and three.

Details for this local interlock resistance modeling are presented in Appendix G. The following four primary conclusions were drawn from the pullout test modeling:

- The interlock knuckles increased the force that the tailwall could transmit to the soil above the force that would be calculated from the normal force on the wall and the interface friction angle. This effect diminished as the steel/soil interface friction angle approached the soil friction angle. In other words, the effect of interlocks was to increase the effective steel/soil interface friction, but the upper bound resistance was that of a soil/soil interface.
- For the model cases analyzed, the limiting pullout resistance was the maximum friction force that could develop on a surface with the same dimensions of the tailwall if the interface friction angle equaled the soil friction angle.
- For the pullout test device modeled, the potential to develop the full pullout resistance was limited by the depth of soil in the device. In other words, the depth of soil should probably be about the same as the length of the sheet pile sample.
- Using interface elements on curved surfaces in FLAC^{3D} required careful consideration and examination of model behavior before accepting results.

Finally, the model tests performed have assumed a dilation angle of zero in the soil surrounding the tailwall. However, the results of this modeling effort show that the upper bound for tailwall pullout resistance is the soil shear strength. Furthermore, a weak interface between the soil and steel sheet reduces the pullout resistance. Therefore, although introducing a soil dilation angle will increase pullout resistance, it is unlikely that this will offset the reduction resulting from the steel/soil interface, which is weaker than a soil/soil interface.

7.5.2 Local Defect Model

It is generally not possible to construct a single model for a large system like the OCSP® that provides answers to all questions about the system performance and behavior. This is because modeling usually involves tradeoffs for simple reasons such as that large numerical models can require so much computer memory as to become impractical to run. In order to address some detailed questions about the stresses in the OCSP® system and the influence of construction defects such as “sheet piles out of interlock,” another set of FLAC^{3D} local models was developed. The purpose of these local models was to examine the consequences of defects in the sheet pile walls, and therefore the models are called local defect models. The purpose was not to model the actual system performance, and therefore some simplifications were used and the model results are not directly comparable to the primary model results.

Two basic models with different geometry were developed; the first model served as a reference or baseline model for the second model:

- The first model represented an infinitely long OCSP® wall at the maximum design section without any defects. Because of the symmetry of an infinitely long wall, this model was limited to two half cells and one tailwall, the same as the previously presented FLAC^{3D} primary model. This first model was referred to as the “2h” model, which was shorthand for “two half-cell”. Figure 7.5-5 illustrates the FLAC^{3D} mesh for the 2h model. The 2h model contains 65,400 zones (soil elements) and 5,656 structural elements.
- The second model was used to model isolated defects in a facewall and tailwall. Because the defects were intended to be isolated, the model encompassed a greater reach of the wall than the reference model; however, a large model meant increased model run time and memory requirements. Therefore, the need to model a larger reach of wall was balanced with the need to obtain a model that did not exceed the memory capacity of the software and modeling platform and ran in a reasonable period of time. In this case, this balance meant limiting the second model to two cell widths; that is, a width of 55 feet. The second model was arranged so that one complete cell and two half cells with two tailwalls were modeled. This model required about 32 hours of run time and 3 GB of memory. The nomenclature “1w+2h” was used as shorthand for “one

whole cell and two half-cells" to identify the second model in this section. Figure 7.5-6 illustrates the FLAC^{3D} mesh for the 1w+2h model, which contains 130,800 zones (soil elements) and 11,344 structural elements.

Because the focus of the local defect modeling effort is on OCSP® stresses, emphasis was placed on attaining a high resolution of the numerical model mesh in the vicinity of the OCSP® face and tailwalls. There are 17 PS31 steel sheet piles in the facewall of each of the ±27-foot-wide cells. The aspect ratio of elements influences the accuracy of the numerical solution and a hexahedron element with height-to-width and width-to-depth ratios of 1 provides the most accurate solution. Therefore, in order to maintain an aspect ratio as close to 1 as possible, an element width slightly less than one PS31 sheet was used in order to divide each whole cell into 16 elements and each half cell into 8 elements.

Maintaining this mesh resolution throughout the model required many elements and thus quickly increased run time and memory requirements. Therefore, the mesh size away from the wall was increased. FLAC^{3D} allows unconforming meshes (that is, meshes with different element sizes) to be attached together along common planar boundaries. The gridpoints (nodes) of the finer mesh were slaved to the displacements of the coarser mesh along the common boundary. This slaving works best when the coarse and fine meshes are related by an integer multiple of elements along each boundary segment. In the 2h and 1w+2h models, coarse meshes with element sizes double that of the finer mesh around the OCSP® bulkhead were attached between the inner finer mesh and the model boundaries.

The actual unzipping and plastic deformation of an interlock is a complex failure mechanism, which is not easily modeled with the linear structural shell elements in FLAC^{3D}. Therefore, in the interest of cost and schedule, a simpler model was used to obtain a first-order understanding of the primary parameters controlling the potential unzipping process.

Initially, the defect model involved deleting shell elements from the bottom of a column of elements in the rectangular mesh corresponding to a facewall or tailwall. In this case the defect appeared as a gap (for example, 11 feet high by the width of one column of elements, which was approximately 17 inches), resulting in two corners at the top of the gap for stress concentrations to occur rather than one crack tip.

To better represent the unzipping process, the defect model was revised to represent a linear defect by leaving all the elements in place but disconnecting the shell elements along one column of nodes. This was done by replacing each node with two nodes, which occupy the same point in space. Above the defect the elements on either side of the node column are connected to the same node. The result is a static model of a crack; that is, the crack cannot propagate but the singularity which causes a stress concentration is present. Appendix G provides additional details on the defect model.

This modeling of the transition from full interlock to no interlock at a single point (that is, the crack tip) may not be as severe as the real condition. For example, consider a 50-foot-high sheet pile tailwall with an interlock that is completely disengaged over the bottom 5 feet and transitions to fully interlocked over a distance of 2 feet. This results in 43 feet of "good" (that is, full strength, interlock). In this situation, a 5-foot crack model is unconservative; however, modeling the defect as a 7-foot crack appears to be conservative. In reality, the partially zippered portion may have a tendency to propagate upwards under interlock loads less than the rated interlock strength. In this situation, it is unclear whether the tendency of the interlock to unzip further can be adequately assessed by modeling the defect as a 7-foot crack.

Details for this local interlock resistance modeling are further summarized in Appendix G.

A series of FLAC^{3D} analyses was conducted using the defect model described above. The following conclusions were drawn from the local defect modeling effort:

- The local defect model results showed that when dredging occurs there is minimal influence on the wall stresses above the mudline; rather, there is a significant increase in wall stresses within the elevation span of dredging. The model consistently showed the maximum wall stress in the dredge zone to be slightly greater than the maximum stress above the initial mudline.

- The base case and parametric cases of the 2h reference model display instability when the dredging proceeds to a final mudline at elevation -51 feet MLLW. This instability is manifested as a pullout of the tailwall and seaward movement of the base of the facewall. Comparison with model cases in which the BCF clay shear strength is increased to the parameters for the long-term condition show that the instability goes away when the long-term undrained shear strength of BCF clay is used. In addition, these comparisons also show that the base case models are stable for dredging to elevation -42 feet MLLW and that the model results for dredging down to elevation -51 feet MLLW are not dramatically different. Therefore, the base case of the local model is useful and valid for assessing the impact of defects on the system performance.
- Because of symmetry, the local defect models represent regularly repeating defects in an infinitely long wall system. Therefore, the increase in wall displacement and stresses from these defects can be expected to decrease if single defects are modeled in similar FLAC^{3D} models that encompass a longer reach of the OCSP® structure.
- The distribution of relative shear displacement vectors on the tailwall is complex and does not correspond to a simple horizontal pullout of the tailwall.
- A primary consequence of facewall defect is to allow retained soil to escape, which relieves the load on the affected open cell face. Therefore, in contrast to a tailwall defect, there is little tendency to develop a stress concentration at the top of the model interlock defect.
- Unlike the loss of interlock in the face of an open cell, there is a definite tendency to develop a stress concentration above such a defect when loss of interlock occurs in a tailwall. This is because the horizontal load transferred to a tailwall by the open cell faces is largely unaffected by the tailwall defect since the material cannot escape to release the stress. Therefore, the same amount of load must be transferred over a smaller length of interlock, which could lead to unzipping.
- The defect models are based on the assumption of no compliance in the interlocks; that is, joints are tight throughout construction. Comparing the no-defect 2h reference model to a 2h model in which the interlock compliance is emulated by reducing the horizontal stiffness of the sheet pile walls indicates that including interlock compliance in the defect models would not significantly alter the results. The principal effect in the results examined is to decrease the magnitude of the compressive hoop stress that develops in the OCSP® walls below the mudline in the model.

7.6 FLAC^{3D} Analyses Comparison with Conventional Methods

This section provides a comparison between the results obtained from FLAC^{3D} analyses and the results from the conventional design methods presented in Section 5 of this report. The comparison was made as a check on FS values and deformations obtained by each method (that is, limit equilibrium and FLAC^{3D}) and to confirm the shape of the critical slip surface used in the limit-equilibrium stability analysis.

7.6.1 Static Global Stability

The comparison described in the following paragraphs focuses on the global stability results for the static loading cases. These static loading cases involve (1) the short-term, undrained case, also referred to as the end-of-construction case; (2) the long-term drained case, representing very slow ground movement during operations; and (3) the long-term undrained case, representing rapid ground movement during operations. For these comparisons, the results from the limit-equilibrium analyses using the computer program Slide, as discussed in Section 5, are compared to the results calculated by FLAC^{3D} using the three-dimensional model and a c-ϕ reduction method of analysis.

The following observations were made from these comparisons:

- **Short-Term Static, Undrained (End-of-Construction) Case.** This case represents the condition immediately after dredging to the final dredge depth and prior to the application of the loads from port facility operations. For both analyses, the landside BCF clay was assumed to be fully consolidated under the backfill load with undrained shear strength estimated from the TXC-based SHANSEP correlation. The fully consolidated condition was assumed because of the extended duration between the placement of backfill material during OCSP® construction and the time at which final dredging will occur. The seaside BCF clay was assumed to be unconsolidated with the undrained shear strength determined from the TXE-based SHANSEP correlation.
  - The as-built OCSP® structure was assumed to be fully built to elevation +35 feet MLLW with a construction surcharge of 200 psf on top of the fill. The construction surcharge was used to represent activities required to complete construction of the facility. The sea level was assumed to be at the lowest elevation of -5 feet MLLW. The groundwater table behind the bulkhead was assumed to be at elevation +20 feet MLLW. The over-dredge elevation was assumed to be at elevation -51 feet MLLW.
  - In the limit-equilibrium analysis, the critical slip surface was assumed to be located outside the main portion of the tailwall. However, trial slip surfaces were allowed to be within the extension part of the tailwall in the limit-equilibrium analysis. The Bishop's Simplified, Spencer, and Morgenstern-Price methods were used to calculate the FS in the limit-equilibrium analysis, and the critical slip surface was defined as the one that associates with the lowest FS.
  - The FS calculated for this loading condition was 1.14 with the limit-equilibrium method, which is very close to the FS value of 1.13 predicted by FLAC^{3D} using the c-ϕ reduction analysis. The location of the critical slip surface found by limit-equilibrium analysis is within the band of high shear strains calculated by FLAC^{3D}, as shown in Figure 7.6-1 (a).
- **Long-Term Static-Drained Case.** In this case, the global stability of the as-built OCSP® structures was evaluated at the long-term condition when excess porewater pressure in the BCF clay generated by the backfill surcharge has been fully dissipated. The shear strength of the BCF clay on both sides of the wall was modeled using a friction angle of 30 degrees with no cohesion; the rate of shear during movement was assumed to be slow enough that no excess porewater pressures develop. The sea level was assumed to be at elevation +7.5 feet MLLW. The as-built OCSP® structure was assumed to be fully built to elevation +38 feet MLLW (including the 3-foot surfacing), and a full live load of 1,000 psf from facility operations and container storage was assumed to be in effect. The over-dredge elevation was assumed to be at elevation -51 feet MLLW.
  - Similar to the short-term static-undrained case, the trial slip surfaces were allowed to be within the extension part of the tailwall in the limit-equilibrium analysis. The lowest FS in the limit-equilibrium method was determined from the FS values calculated by Bishop's Simplified, Spencer, and Morgenstern-Price methods. FS values from FLAC^{3D} were obtained using the c-ϕ reduction method of analysis.
  - Results obtained from both limit-equilibrium and FLAC^{3D} analyses indicate comparable FS values (for example, 1.26 versus 1.23). However, the critical slip surface identified by the limit-equilibrium analysis appears to be slightly deeper than that calculated by FLAC^{3D}, as shown in Figure 7.6-1 (b). This difference is attributed to a combination of effects, including the method used to solve for equilibrium in each method, the two- versus three-dimensional modeling methods being used in Slide versus FLAC^{3D} analysis, and assumptions regarding soil stiffness and stress development within each model. Given the small difference in FS values and the location of the slip surface, the comparisons are considered to be very good.

- **Long-Term Static-Undrained Case**. In this case, the global stability of the as-built OCSP® structures was evaluated when a rapid drop in tidal elevation occurs at the time that the facility is fully operational under the 1,000-psf operational loading from facility activities and container storage. The landside BCF clay was assumed to be fully consolidated with undrained shear strength estimated from the TXC-based SHANSEP correlation. The seaside BCF clay was assumed to be unconsolidated with the undrained shear strength determined from the TXE-based SHANSEP correlation. The sea level was assumed to be at elevation -5 feet MLLW. The as-built OCSP® structures were assumed to be fully built to elevation +38 feet MLLW (including the 3-foot surfacing). The over-dredge elevation was assumed to be at elevation -51 feet.
  - Assumptions regarding the shape and location of trial slip surfaces are consistent with those used in the above two cases.
  - Both limit-equilibrium and FLAC^{3D} analyses yielded the same FS value of 1.06 with critical slip surfaces located very close to each other, as shown in Figure 7.6-1 (c).

It was concluded from these comparisons that the two methods of analysis are consistent with one another. This consistency gives confidence that predictions by the two methods are reasonable based on the assumptions that were made regarding soil conditions, groundwater and tidal elevations, and external loads.

7.6.2 Seismic Performance

Response of the OCSP® structure was also compared for seismic loading from the OLE, CLE, and MCE based on results of limit-equilibrium/Newmark and the FLAC^{3D} analyses. These comparisons were more difficult because of the very low static FS values occurring in the as-built model for static loading, as discussed in the preceding subsection. In this case, little margin exists within the OCSP® structures to handle increased inertial loads from seismic events.

Both methods of analysis become very sensitive to assumptions on soil strengths and loading conditions in this situation, potentially leading to large differences in results. This was evident in Section 5 when the limit-equilibrium/Newmark method used strengths consistent with ring-shear test results. Results of these analyses suggested that the OCSP® structure would be unstable with seismic loading. Under this condition, the limit-equilibrium method was not able to estimate displacement for seismic loading, other than to indicate that the movement could be very large. Similar issues appear to affect results of deformation predictions in the FLAC^{3D} analyses.

Realizing this, the following observations were made for the seismic deformations from the analyses summarized in Section 5 and the FLAC^{3D} results discussed above in this section:

- **Deformations during OLE**. In general, deformations predicted by the limit-equilibrium and the FLAC^{3D} analyses were small during the OLE, not exceeding 20 inches. Both methods predicted permanent movements greater than the PIEP design criteria value of 3 inches. Although these movements are large, they do not suggest that very large 1964-like movements would result, even though ground motions reported for the 1964 Alaska earthquake in the Anchorage area are roughly similar to those predicted for the OLE.

- **Deformations during CLE**. Results of both methods indicate that deformations during the CLE can be very large—exceeding the PIEP displacement design criteria of 6 inches by an order of magnitude or more. Even with an optimistic assumption of limited soil strength loss from permanent ground displacement, large movements were predicted in both analyses. The large movements are attributed to the low static FS values, as noted above. These low static FS values leave the OCSP® system with little margin in capacity to handle the additional inertial forces from seismic loading. Long-duration earthquakes from a megathrust event (similar to the 1964 Alaska earthquake) would be particularly critical, as the long-duration seismic events were observed to cause large strength losses, similar to those on Fourth Avenue. While the results of the FLAC^{3D} modeling were unable to duplicate this movement because of the simplified material model being used, the large displacements that were predicted support the potential for many feet of movement, as was estimated by the limit-equilibrium method, and as observed during the 1964 Alaska earthquake.

- **Deformations during MCE.** Both the limit-equilibrium and FLAC^{3D} methods predict very large movements, in excess of those described above. Lower-bound displacements exceed the 18-inch criterion set for the PIEP, and a potential exists for very large movements similar to those observed during the 1964 Alaska earthquake. The cause of these large displacement estimates is the same as discussed for the CLE, which is the low margin of soil capacity to resist the loading demand. For the MCE, inertial forces are higher during the seismic event than for the CLE, and capacity of the soil to resist the seismic loads is unchanged. In this case, higher displacements are to be expected. Again, both the limit-equilibrium and the FLAC^{3D} methods are consistent in the prediction of displacements that exceed those estimated for the CLE and those used in the PIEP design criteria.

It was concluded from the comparison of estimated ground movement during seismic events that the CLE and MCE seismic events could result in very large movement of the OCSP® system, beyond what is required to meet the PIEP design criteria. Although the absolute comparison of displacement from limit-equilibrium/Newmark analyses and the FLAC^{3D} analyses was not as precise as observed for static stability, both predicted large movements. This consistency gives confidence that predictions by the two methods are reasonable based on the assumptions that were made regarding soil conditions, groundwater and tidal elevations, operational loads, and the level of seismic ground shaking.

7.7 Conclusions from Numerical Modeling

The computer program FLAC^{3D} was used to evaluate the response of the OCSP® structure under gravity and seismic loading conditions. These evaluations were conducted for two cases: as-designed conditions and as-built conditions. The as-design case modeled the response of the OCSP® structure using the original design assumptions for OCSP® geometry, seismic loads, and soil strengths. For the as-built case, the general geometry of the structure was maintained; however, parameters used in the numerical model were changed to represent CH2M HILL's best estimate of conditions after construction when the facility is in operation. Changes for the as-built evaluation included the strength of the soil used in the model, the elevation of groundwater and tidal elevations, and the amount of surcharge load. These changes resulted in more severe loading than was considered by the original designer. In addition to this evaluation of global response, the potential effects of construction-related defects in the installed sheet piles were also evaluated in this numerical modeling effort.

The following conclusions were reached from this numerical modeling effort:

- **As-Designed Analyses.** Results from as-designed FLAC^{3D} analyses for gravity and seismic loading showed that performance of the OCSP® structure would be generally consistent with performance predicted by the original designers. Estimates of seismic deformations were consistent with the lower bound of estimates; however, for the upper bound deformation estimates, CH2M HILL's FLAC^{3D} results suggest that higher deformations could occur, particularly during the larger CLE and MCE seismic events. These differences in displacement estimates for the as-designed case are believed to represent subtle differences in modeling methods. These differences are not surprising given the complexity of the overall soil-structure interaction process during seismic loading.

- **As-Built Analyses.** Results of the as-built FLAC^{3D} analyses for gravity and seismic loading showed much poorer performance than the as-designed analyses. FS values for FLAC^{3D} analyses using a c-ϕ reduction method were lower by 20 to 30 percent than obtained for the as-designed case. In general, these FS values were lower than normally accepted for design. The lower FS values can be attributed to the lower strengths assigned by CH2M HILL to the BCF clay on the sea side of the sheet pile wall, somewhat higher groundwater elevations behind the sheet pile wall, use of a live load of 1,000 psf starting at the bulkhead line, and lower tidal elevations used for seismic analyses. Of these factors, the change in strength was the most significant factor. The low FS values—or essentially the margin between capacity and demand—resulted in excessive seismic deformations, particularly during the CLE and MCE seismic events. Upper-bound displacements were predicted to be in excess of 10 feet, which is not unlike the displacements that occurred at Fourth Avenue, Government Hill, L Street, and Turnagain Heights during the 1964 Alaska earthquake. Levels of ground shaking for the CLE and MCE events will exceed those occurring in Anchorage during the 1964 earthquake, suggesting that large seismic-induced displacements would not be unexpected. Deformations predicted by $FLAC^{3D}$ did not account for the strength softening that was observed at Fourth Avenue and obtained from laboratory tests described in Section 5 of this report, further confirming the risk of large displacements predicted by $FLAC^{3D}$.

- **Local Defects Model.** Results of $FLAC^{3D}$ analyses of the tailwall pullout confirmed that higher reaction is developed by the sheet pile interlocks and this higher capacity is limited by the frictional strength of the soil. Results from an analysis of defects in the OCSP® structure, as constructed, showed that there is minimal influence from defects above the mudline when dredging occurs, although stresses increase as the soil is dredged. There is a potential for instability as dredging occurs if the strength of the BCF clay has not fully consolidated under the new backfill loads. This instability would involve wall pullout and wall face movement. Once the BCF clay consolidates under the backfill loads, the risk of this type of instability is low. The consequence of facewall defects is that it allows soil to escape, and this soil loss reduces loading to the facewall. However, for the tailwall, defects will result in stress concentrations above the defect, potentially leading to “unzipping” of the tailwall sheets.
- **OCSP® Structure Comparison with Conventional Methods**. Comparisons between limit-equilibrium and $FLAC^{3D}$ results were made for gravity and seismic loading of the as-built structure. These results were very comparable for static (gravity) loading, both having low FS values. Although zone of maximum shear differed for the long-term drained case, comparisons for the short-term undrained and long-term undrained cases were very comparable, thereby providing a separate check on the numerical modeling method. Although displacements estimated through the limit-equilibrium/Newmark approach were comparable for the OLE seismic case, the comparisons for the CLE and MCE were not as close as FS values from gravity loading, mainly because of the large deformations associated with both methods for some earthquake records. Both the limit-equilibrium/Newmark approach and the $FLAC^{3D}$ model have large uncertainties when deformations exceed a few feet, with these results indicating large to very large deformations. The results show, however, that the design criteria required by the PIEP are not being satisfied during gravity and seismic loading.

These numerical analyses indicate that for the as-built case, FS values are lower than are normally accepted for the design of port facilities and that deformations predicted under CLE and MCE events will potentially be very large, resulting in what appears to be an unacceptable risk to the development.

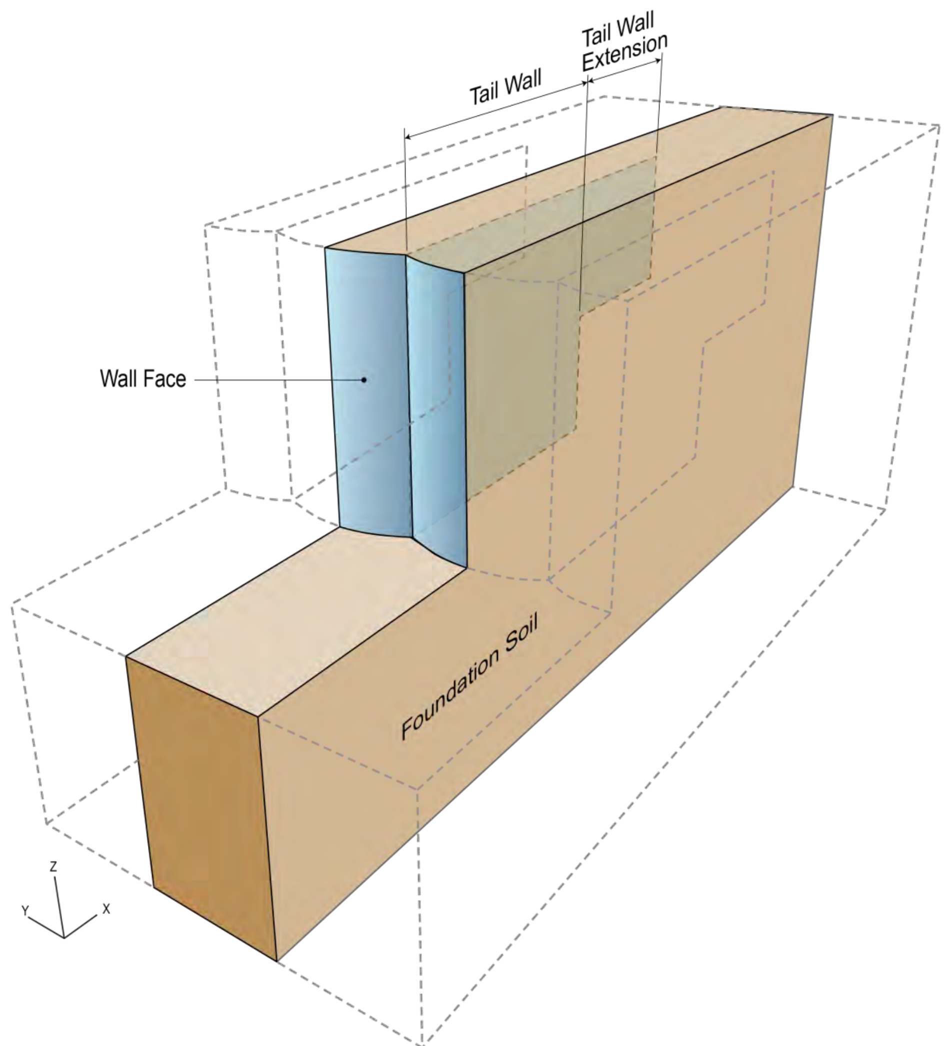


FIGURE 7.1-1. Twin Half-Cell Geometry and Components

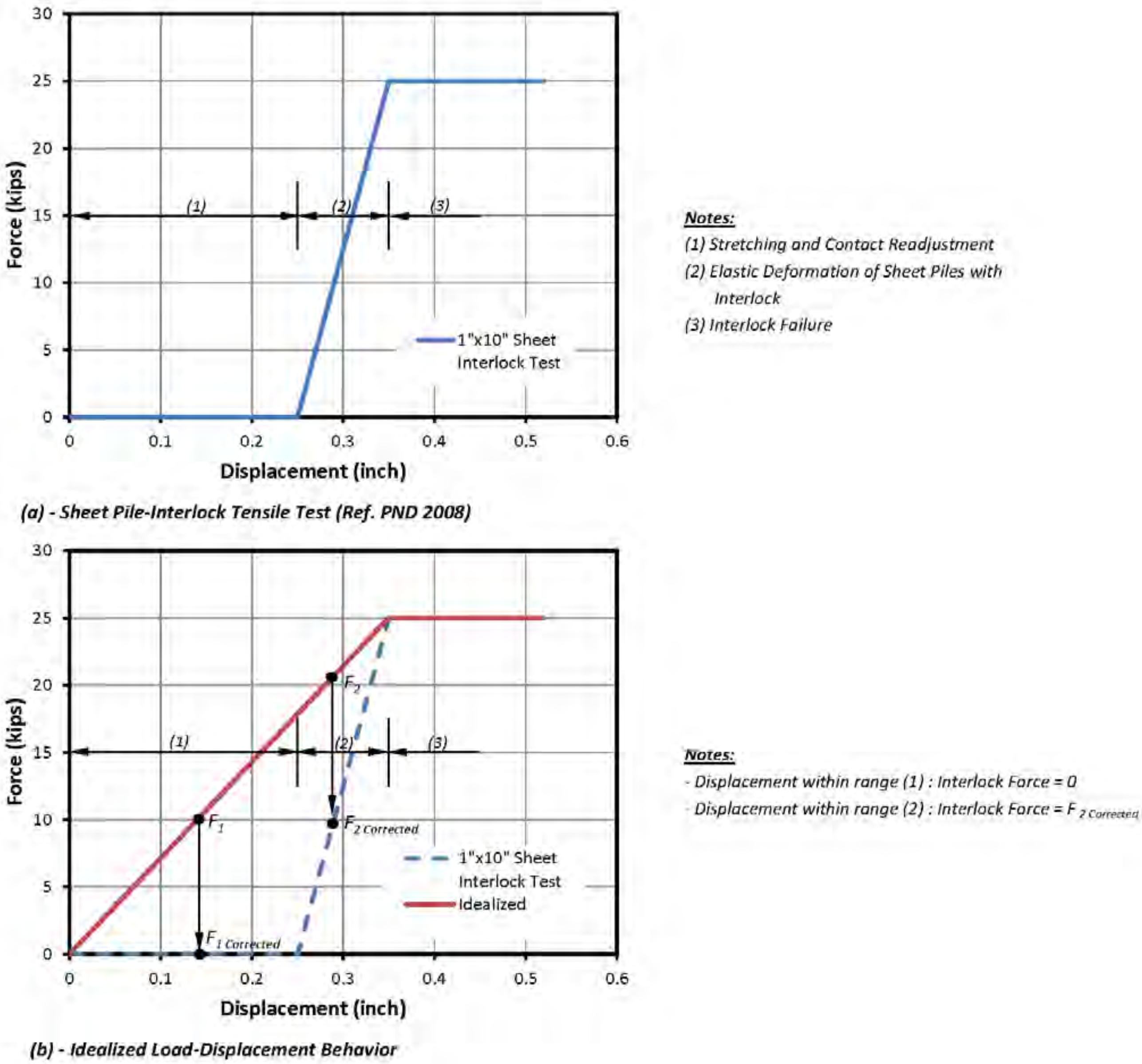
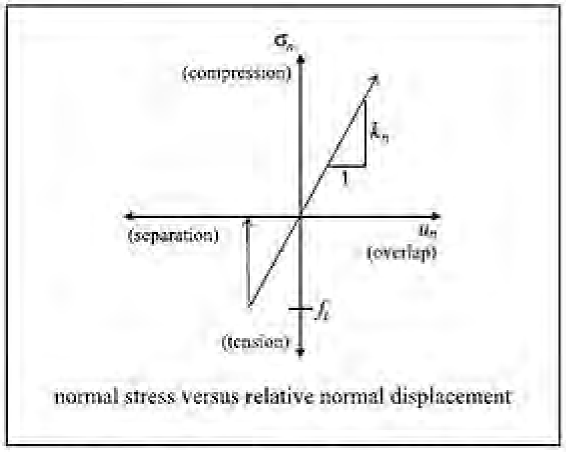
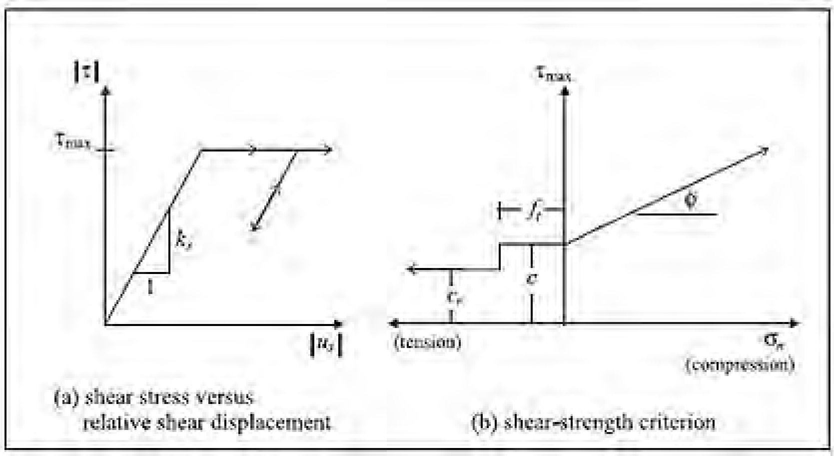


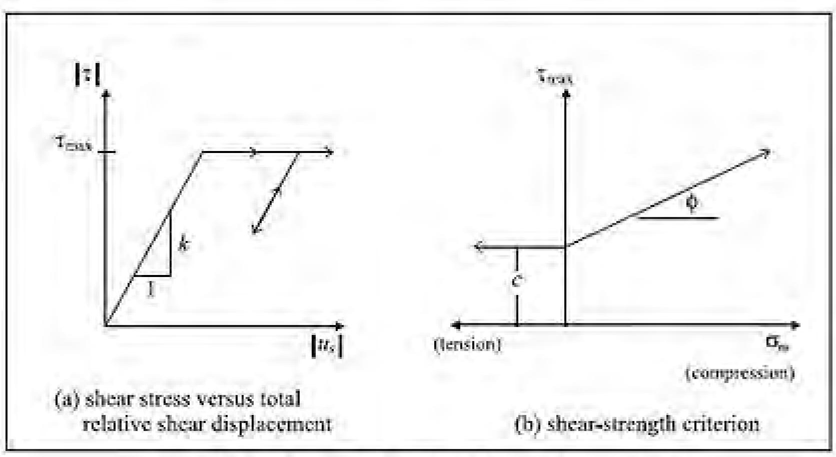
FIGURE 7.2-1. Sheet Pile-Interlock Behavior, (a) Sheet Pile-Interlock Tensile Test (PND, 2008), (b) Idealized Load-Displacement Behavior



(a)- ***Normal-directional interface behavior for linerSELs***



(b)- ***Shear-directional interface behavior for linerSELs***



(c) - ***Shear-directional interface behavior for geogridSELs***

FIGURE 7.2-2. Structural Element Coupling Springs Shear Stress Versus Relative Displacement Relationships and the Shear Strength Criterion

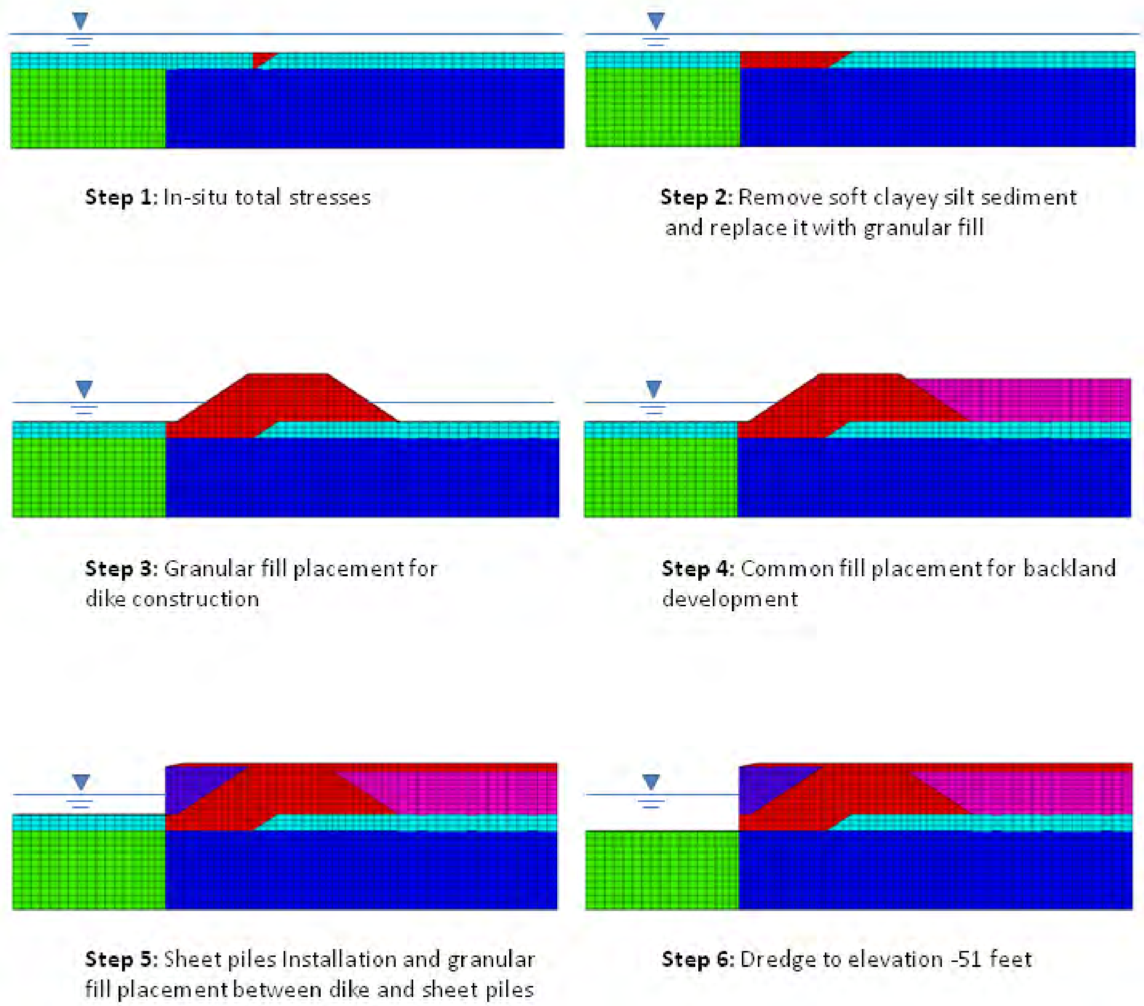
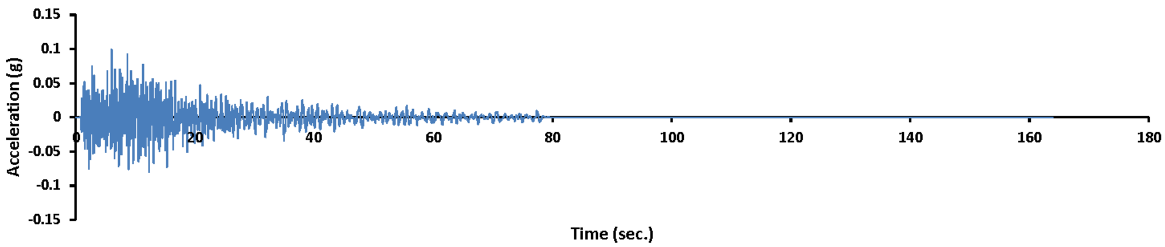
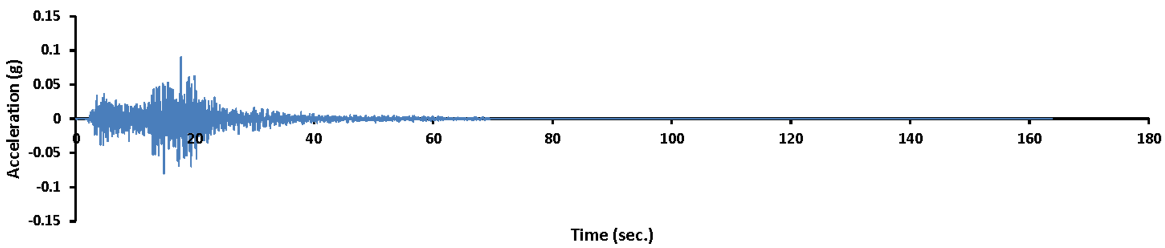


FIGURE 7.2-3. Primary Model Loading Stages

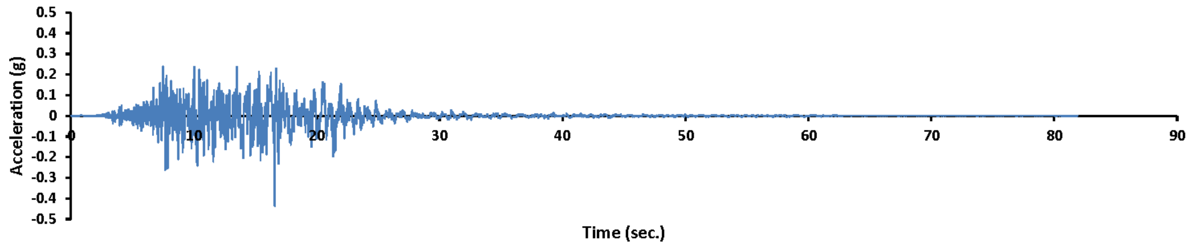


(a) - LFPS010H Record (Puget Sound 1965 EQ - M_w=6.5)

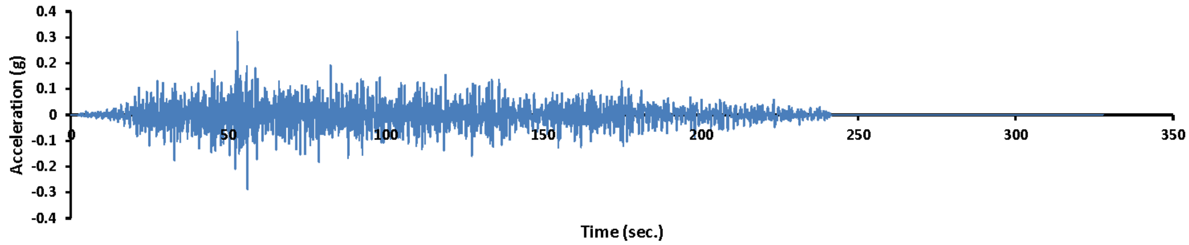


(b) - LFMCUNOH Record (Michoacan 1997 EQ - M_w=6.6)

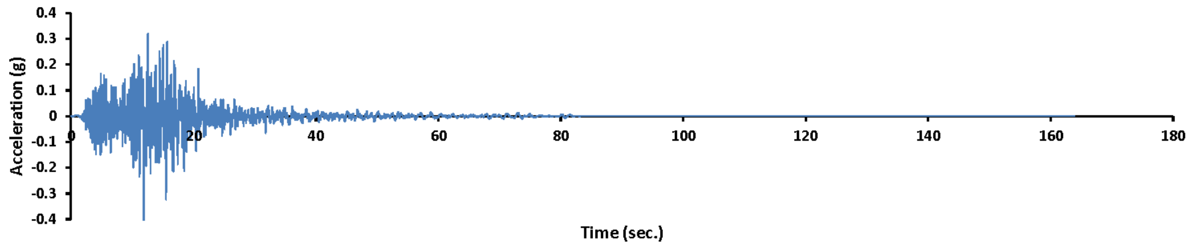
FIGURE 7.2-4. OLE Outcrop Ground Motions at Elevation -150 feet



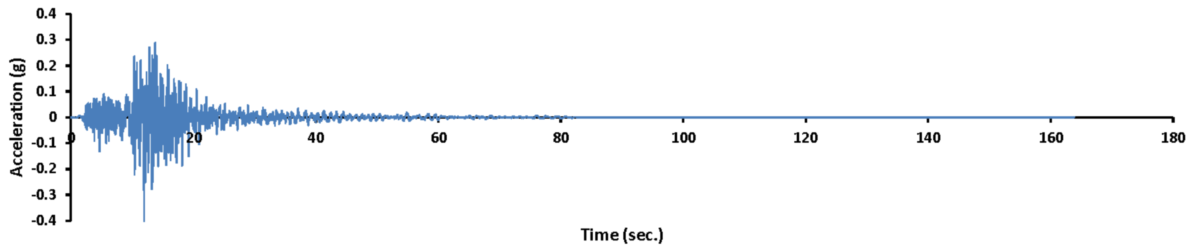
(a) - LFMC180C Record (Michoacan 1997 EQ - Mw=7.1)



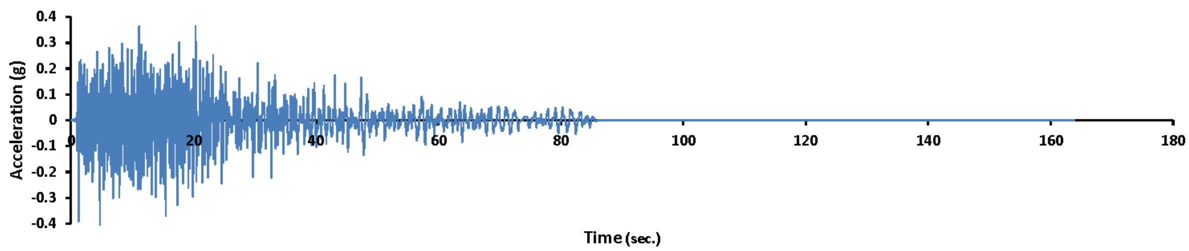
(b) - LFCAS05C Record (Synthetic - M_w=9.0)



(c) - LFNQP0C Record (Nisqually 2001 EQ - M_w=6.8)

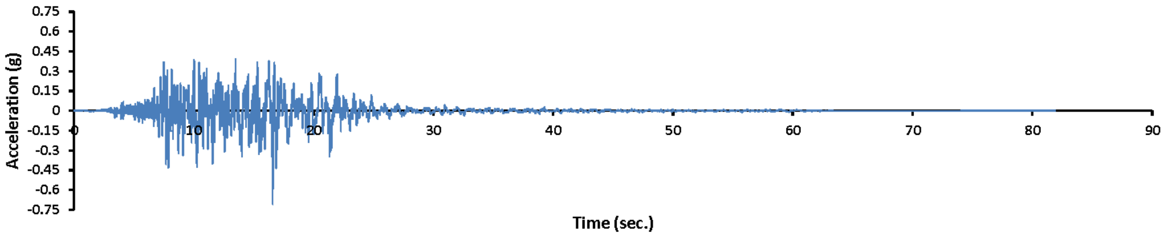


(d) - LFNQP90C Record (Nisqually 2001 EQ - M_w=6.8)

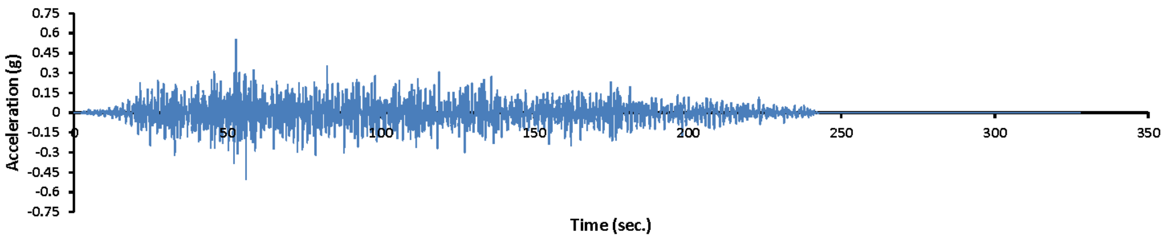


(e) - LFWO356C Record (Western Washington 1949 EQ - M_w=7.1)

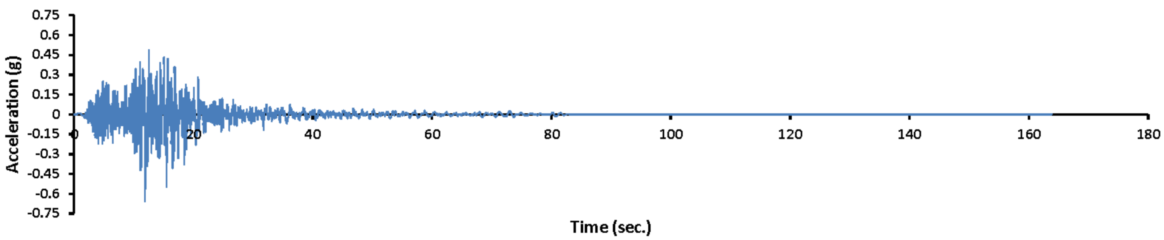
FIGURE 7.2-5. CLE Outcrop Ground Motions at Elevation -150 feet



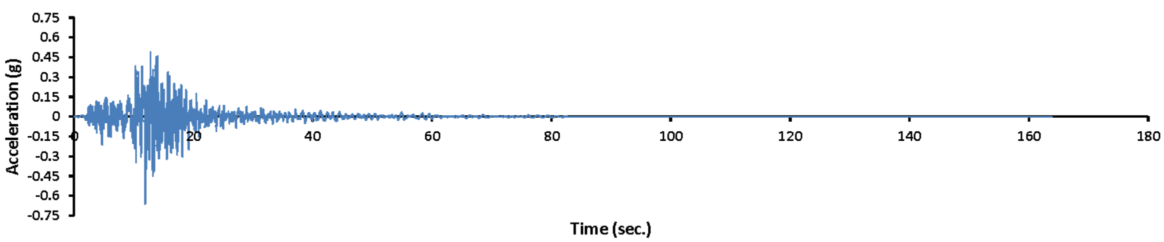
(a) - LFMC180M Record (Michoacan 1997 EQ - M_w=7.1)



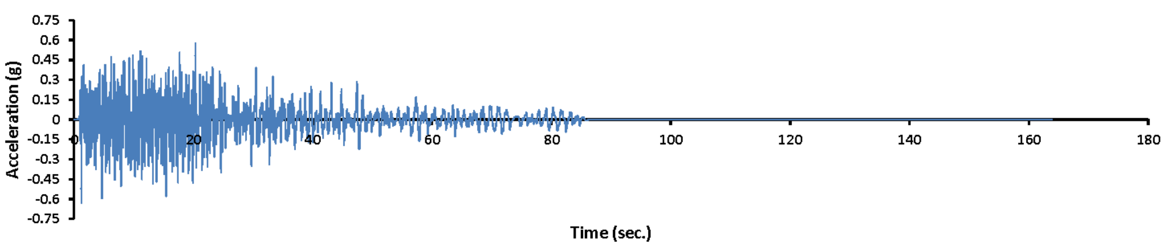
(b) - LFCAS05M Record (Synthetic - M_w=9.0)



(c) - LF_NQP0M Record (Nisqually 2001 EQ - M_w=6.8)



(d) - LFNQP90M Record (Nisqually 2001 EQ - M_w=6.8)



(e) - LFWO356M Record (Western Washington 1949 EQ - M_w=7.1)

FIGURE 7.2-6. MCE Outcrop Ground Motions at Elevation -150 feet

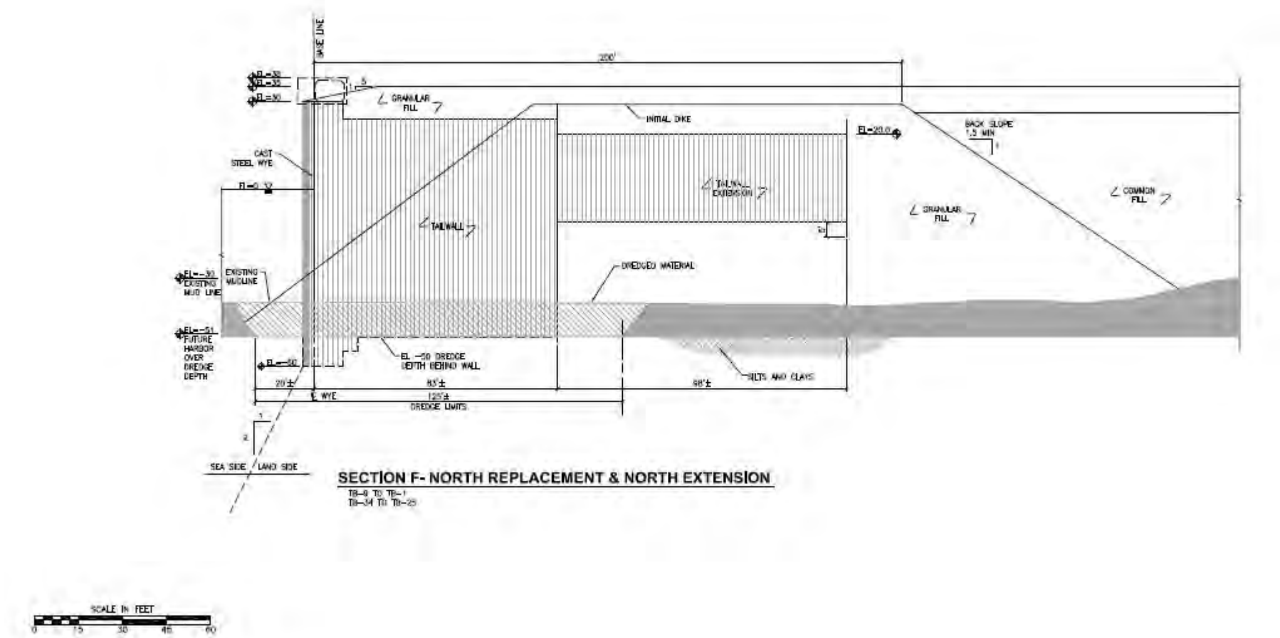


Figure 7.3-1. Typical Cross-Section for As-Designed Section F (PND, 2008)

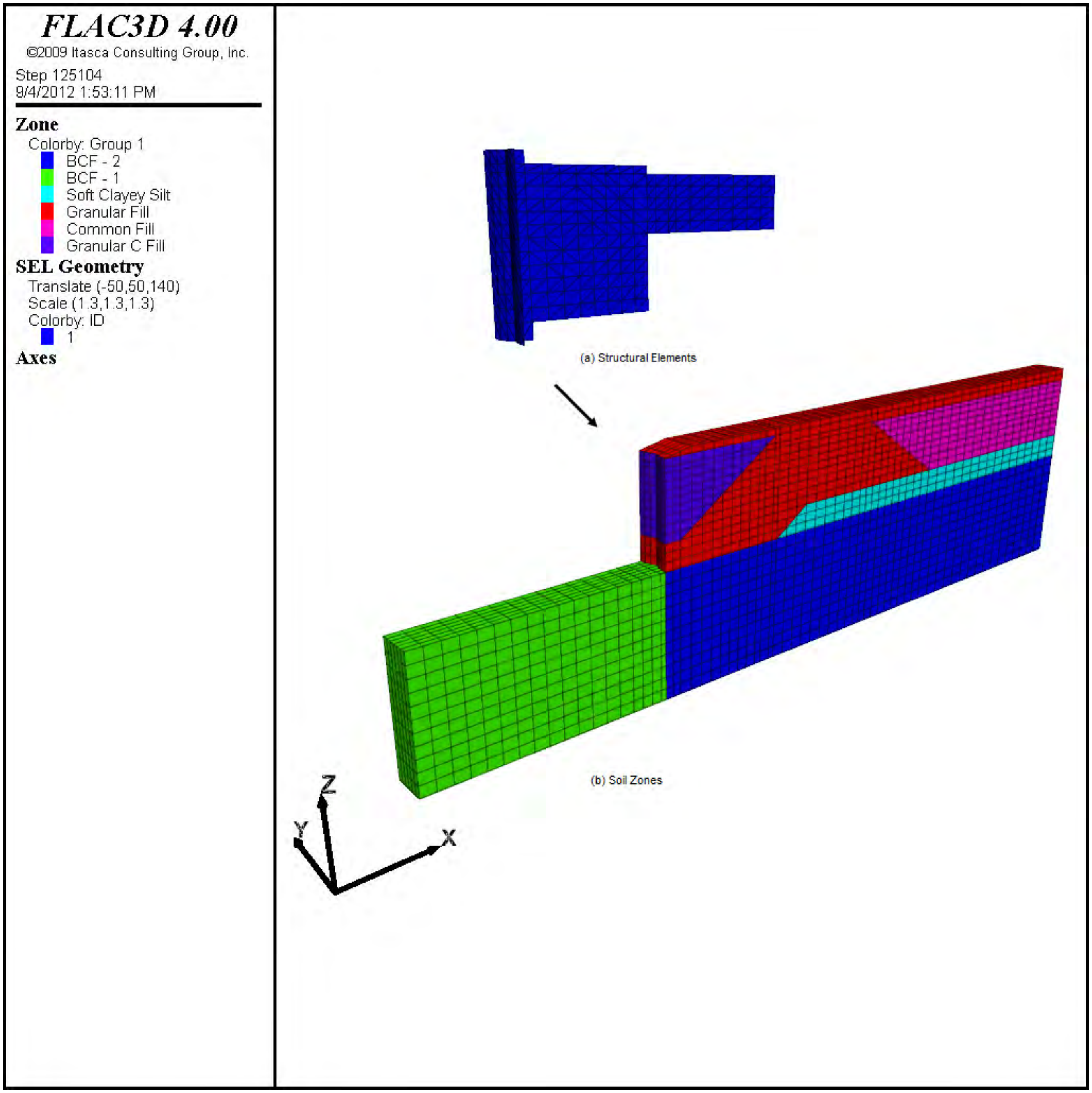


Figure 7.3-2. As-Designed FLAC^{3D} Analysis Mesh

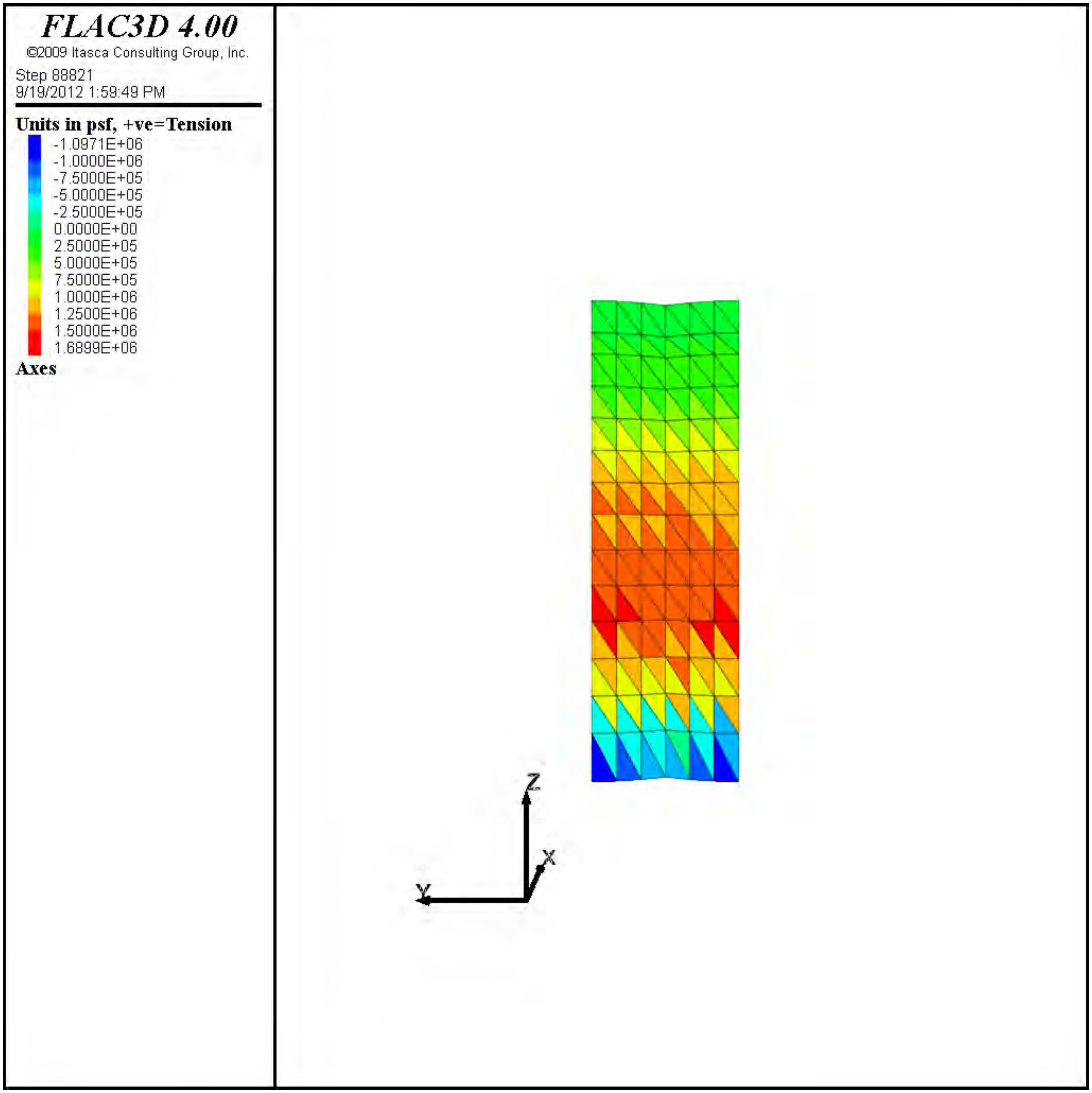


Figure 7.3-3. Facewall Membrane Stress – As-Designed Static Short-Term

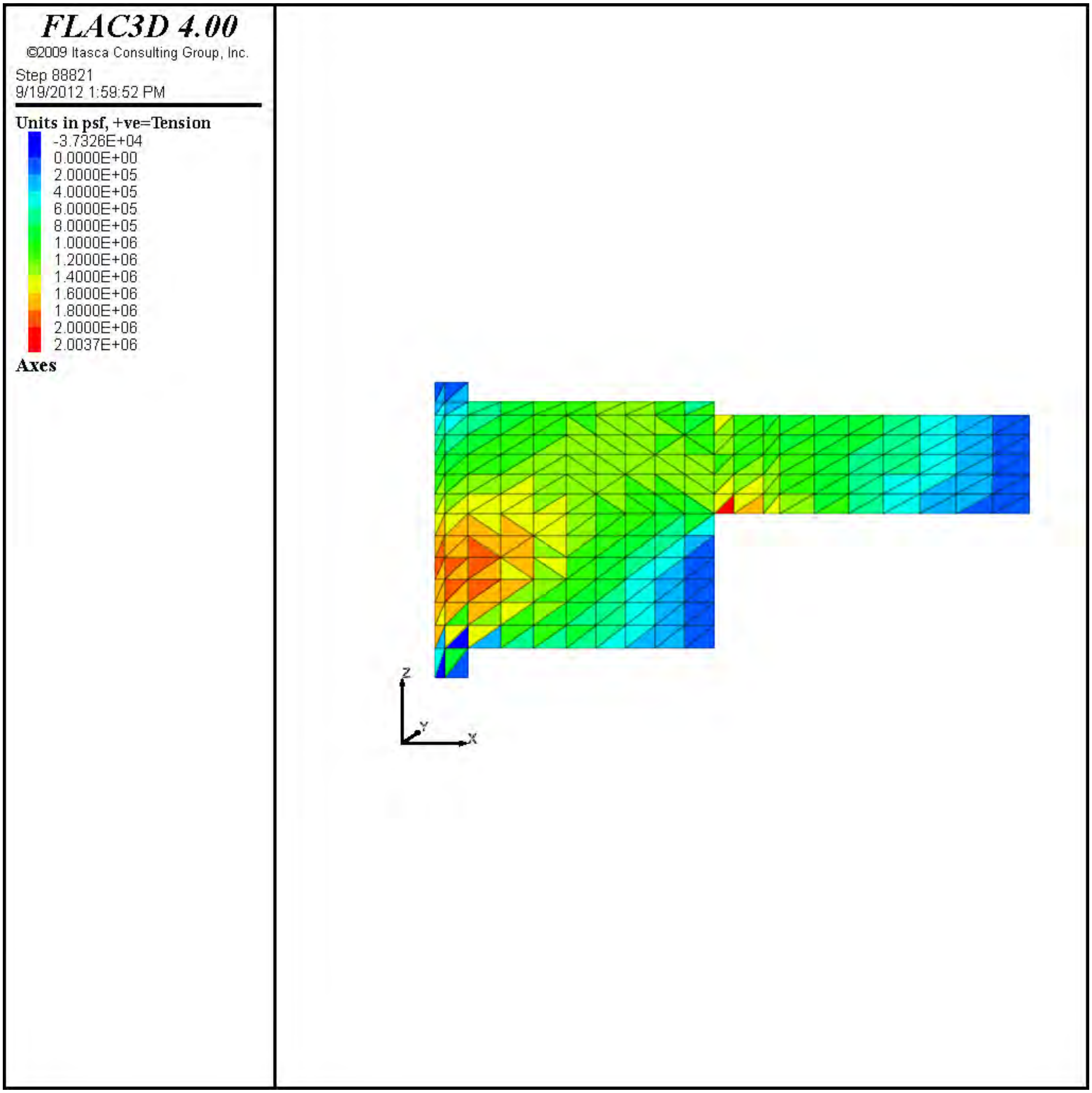


Figure 7.3-4. Tailwall Membrane Stress – As-Designed Static Short-Term

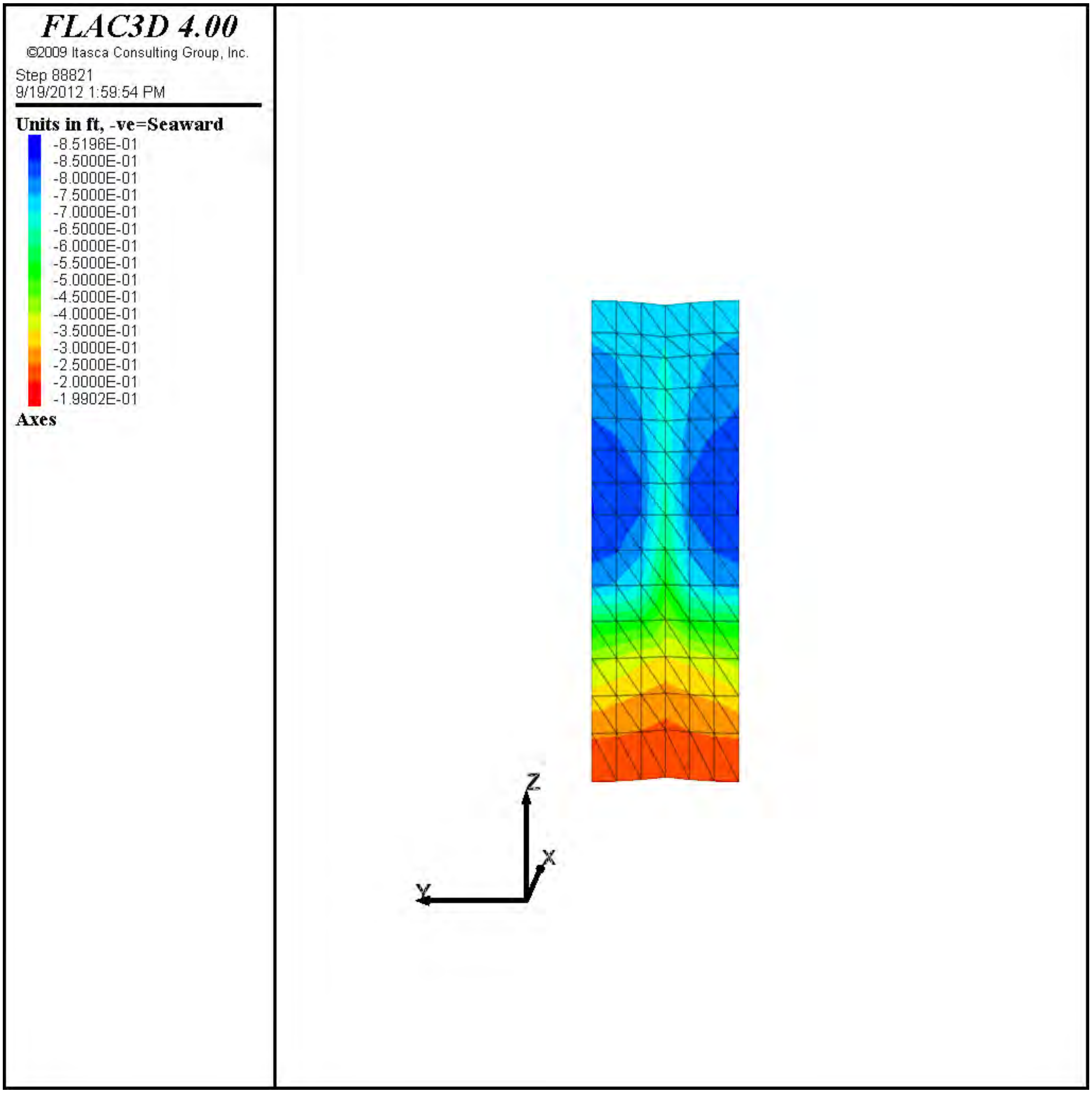


Figure 7.3-5. Facewall X-Displacement Contours – As-Designed Static Short-Term

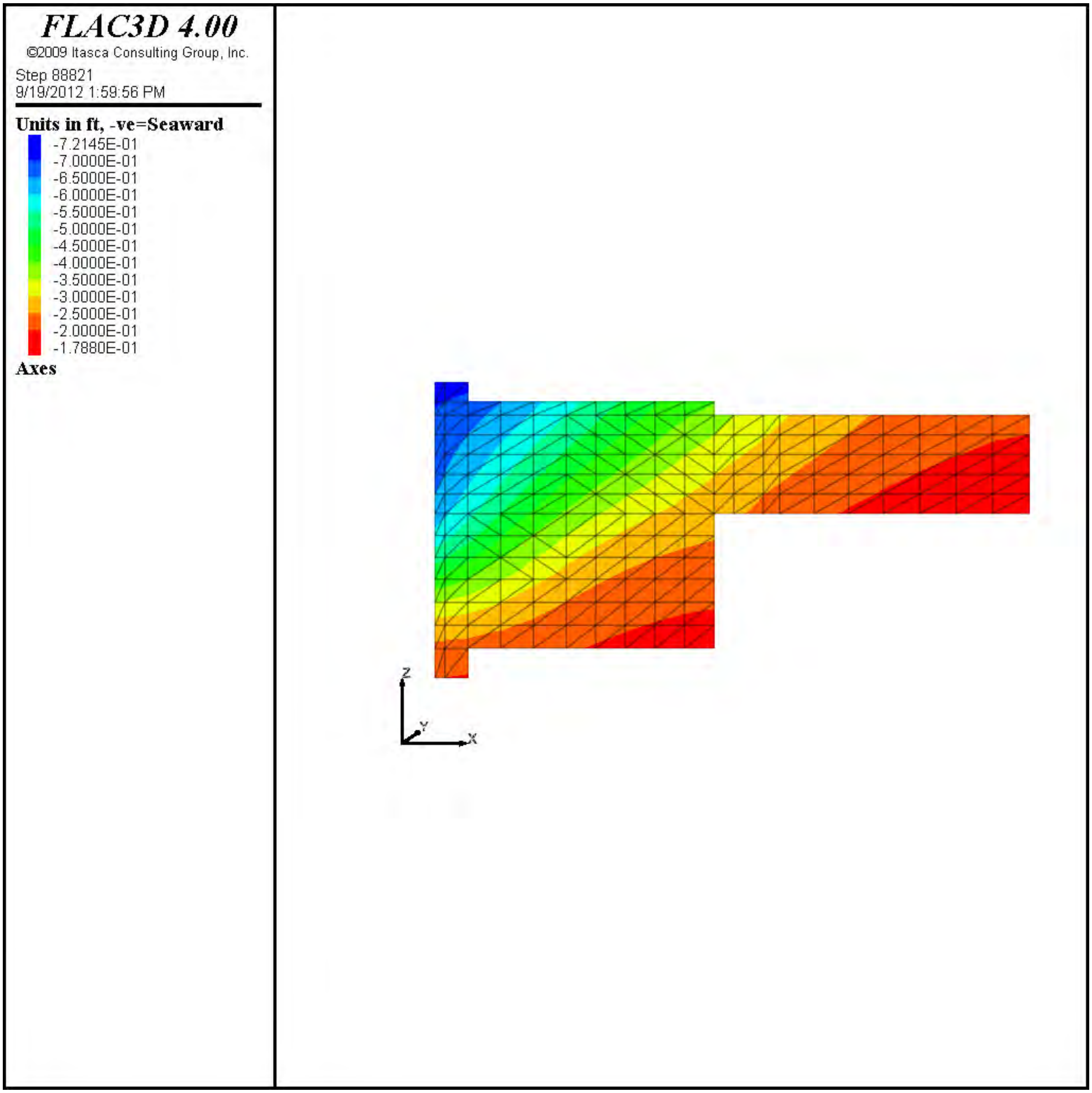


Figure 7.3-6. Tailwall X-Displacement Contours – As-Designed Static Short-Term

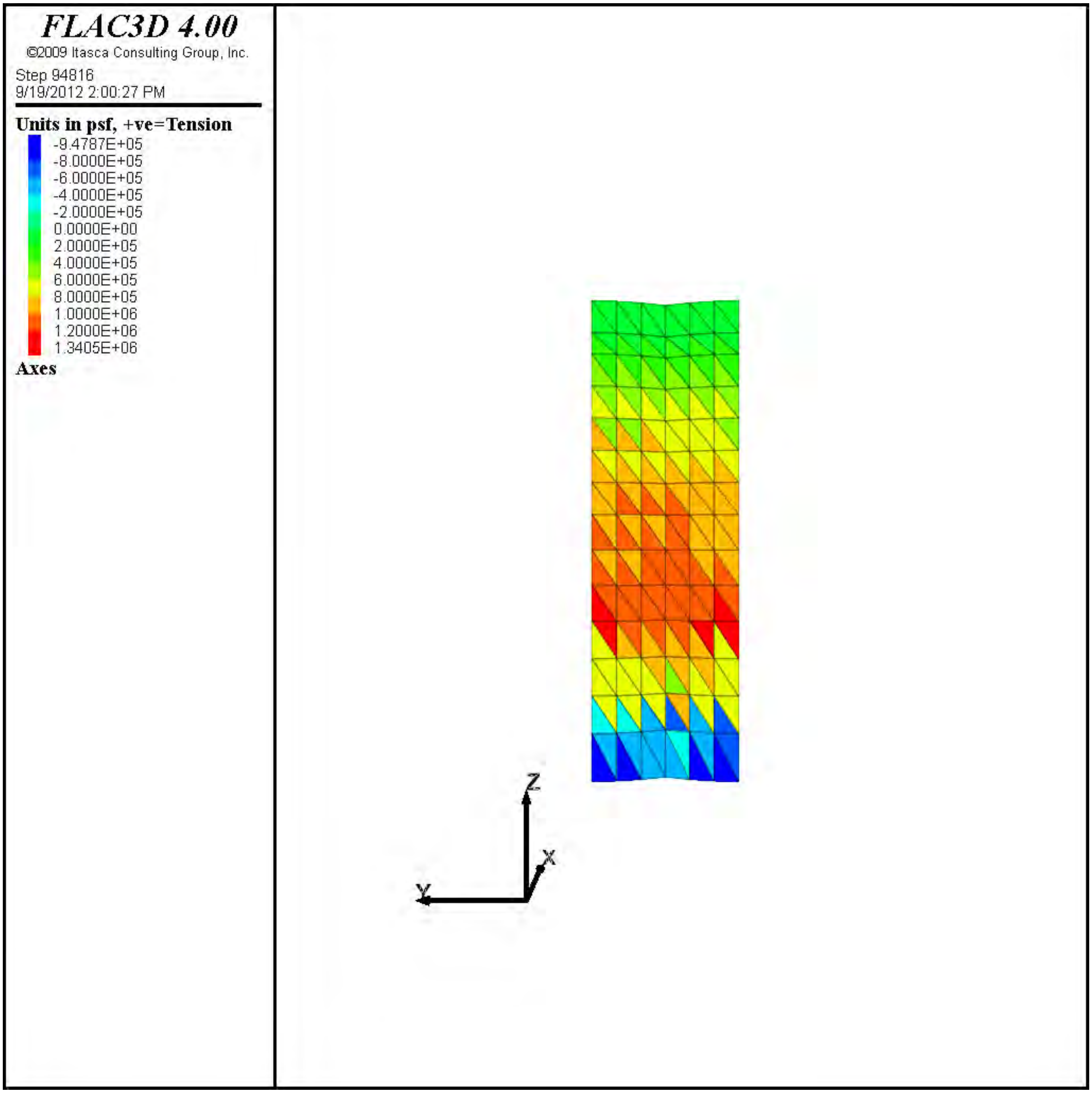


Figure 7.3-7. Facewall Membrane Stress – As-Designed Static Long-Term Drained

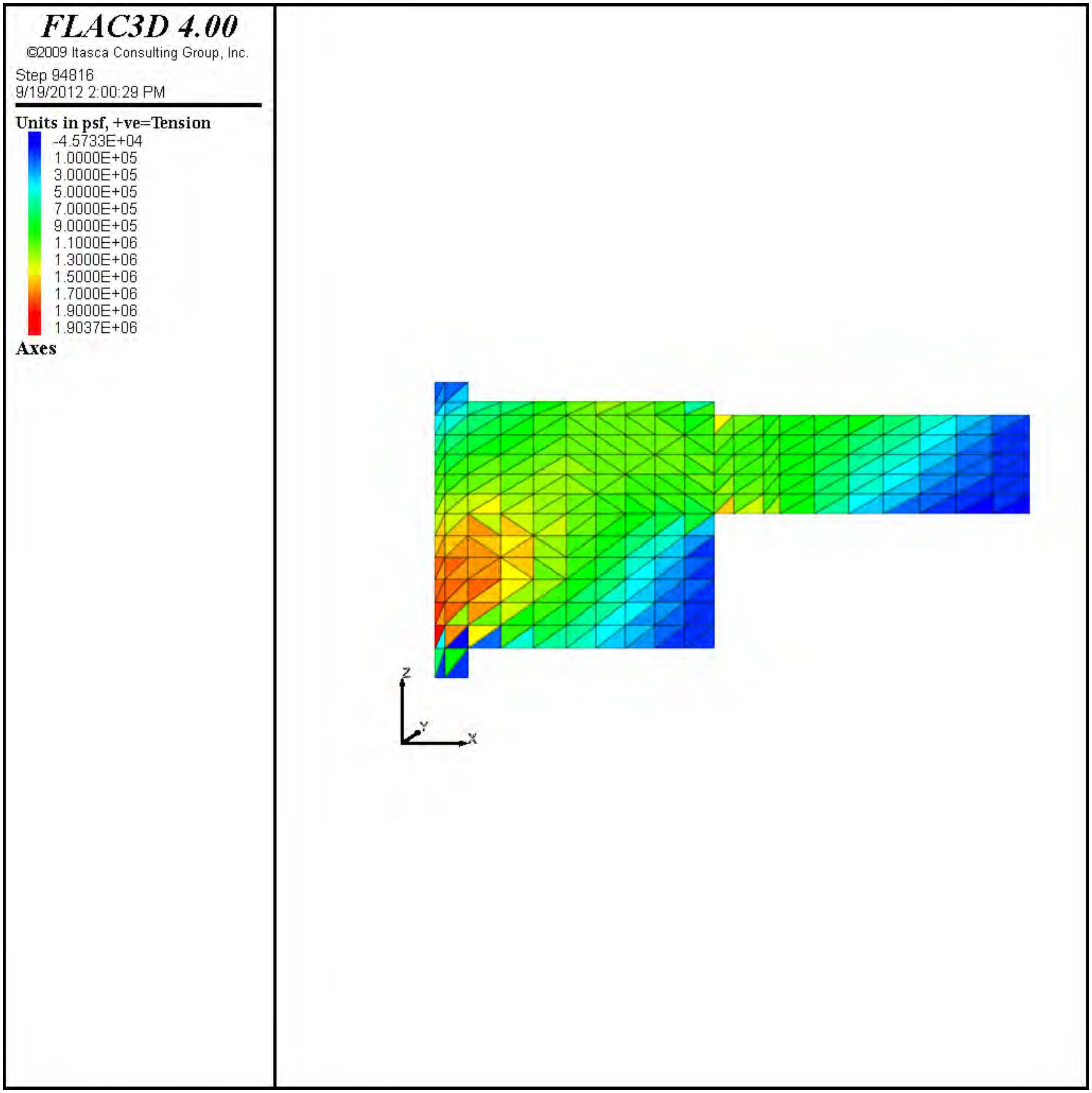


Figure 7.3-8. Tailwall Membrane Stress – As-Designed Static Long-Term Drained

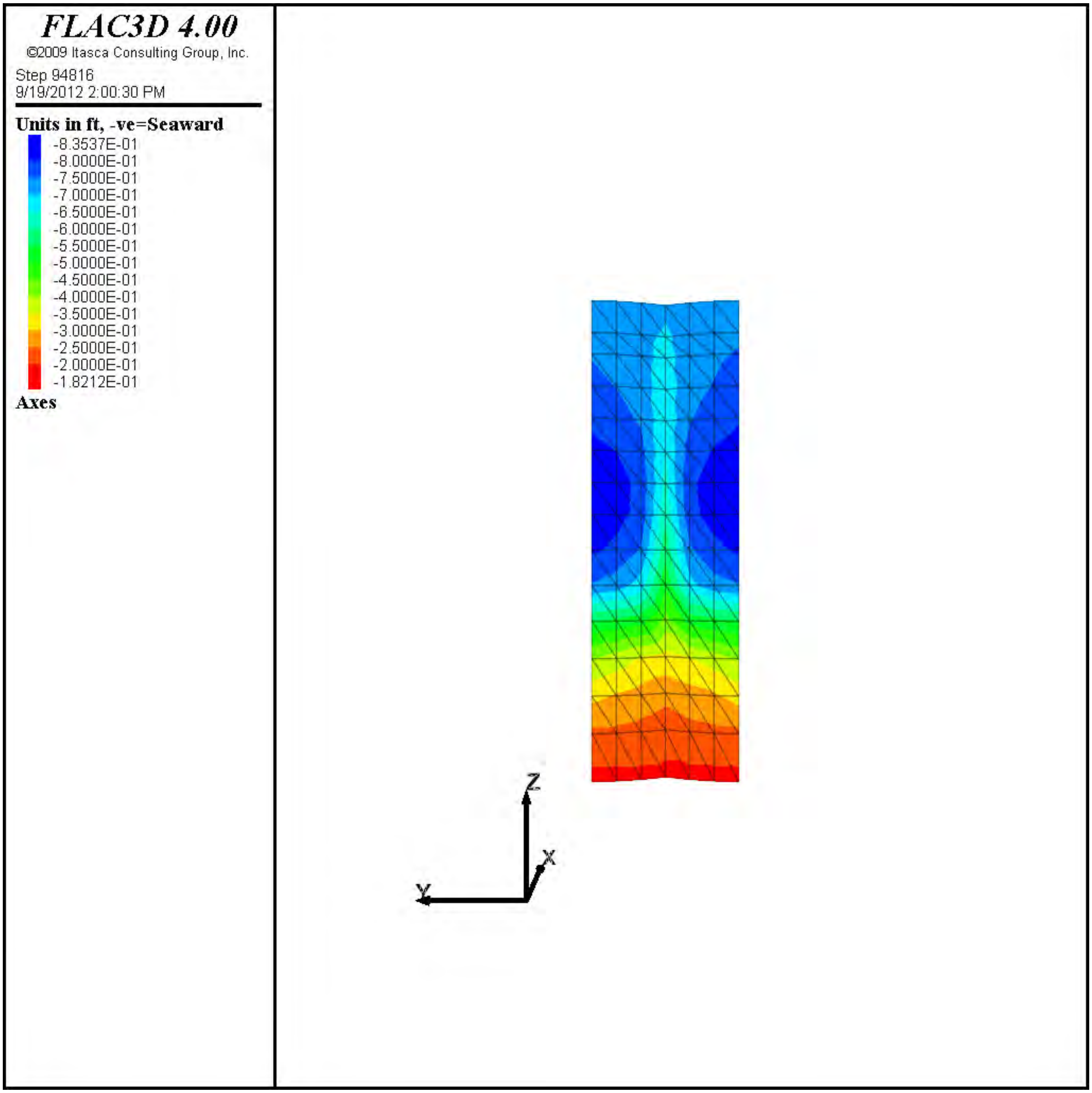


Figure 7.3-9. Facewall X-Displacement Contours – As-Designed Static Long-Term Drained

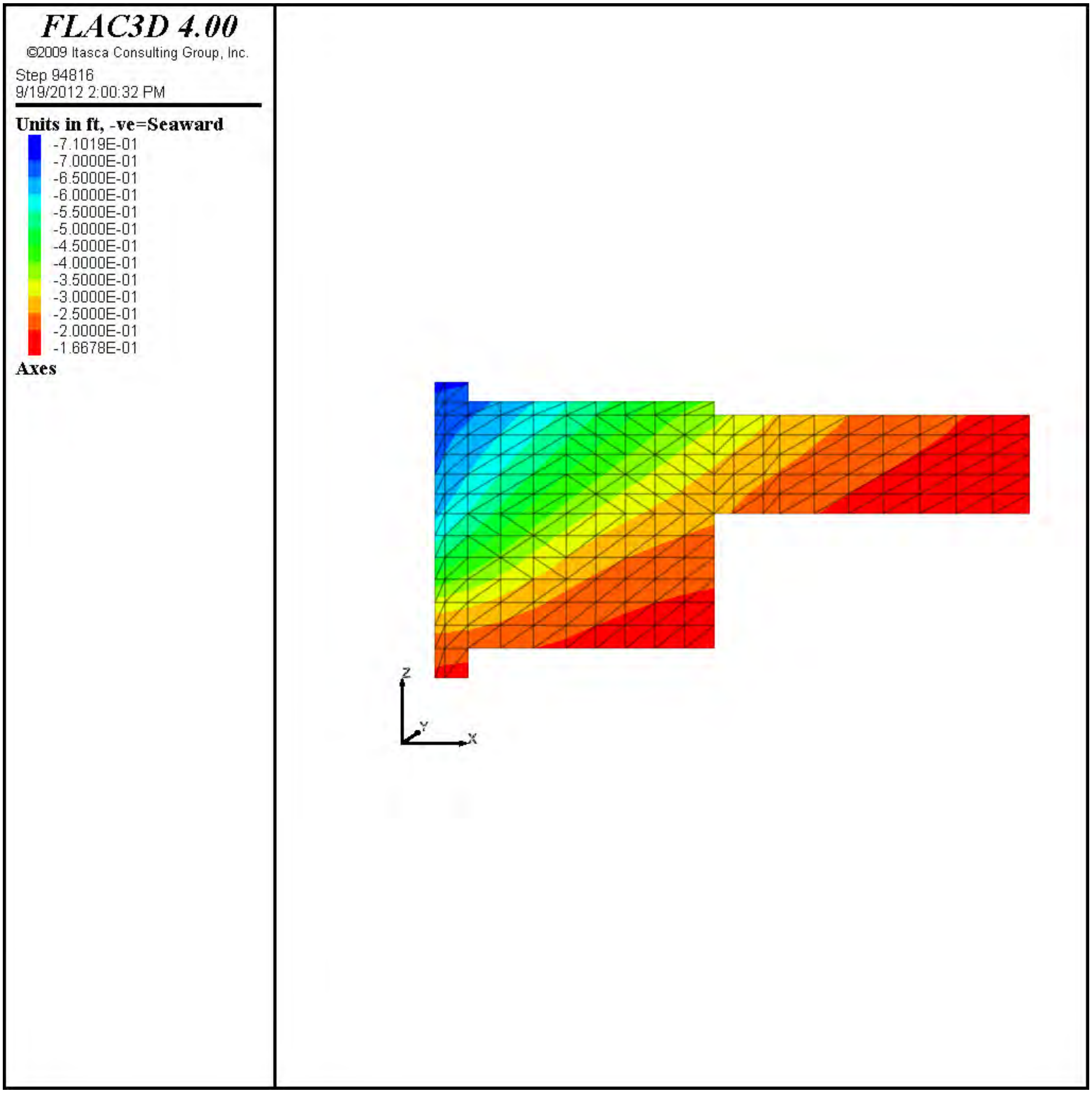


Figure 7.3-10. Tailwall X-Displacement Contours – As-Designed Static Long-Term Drained

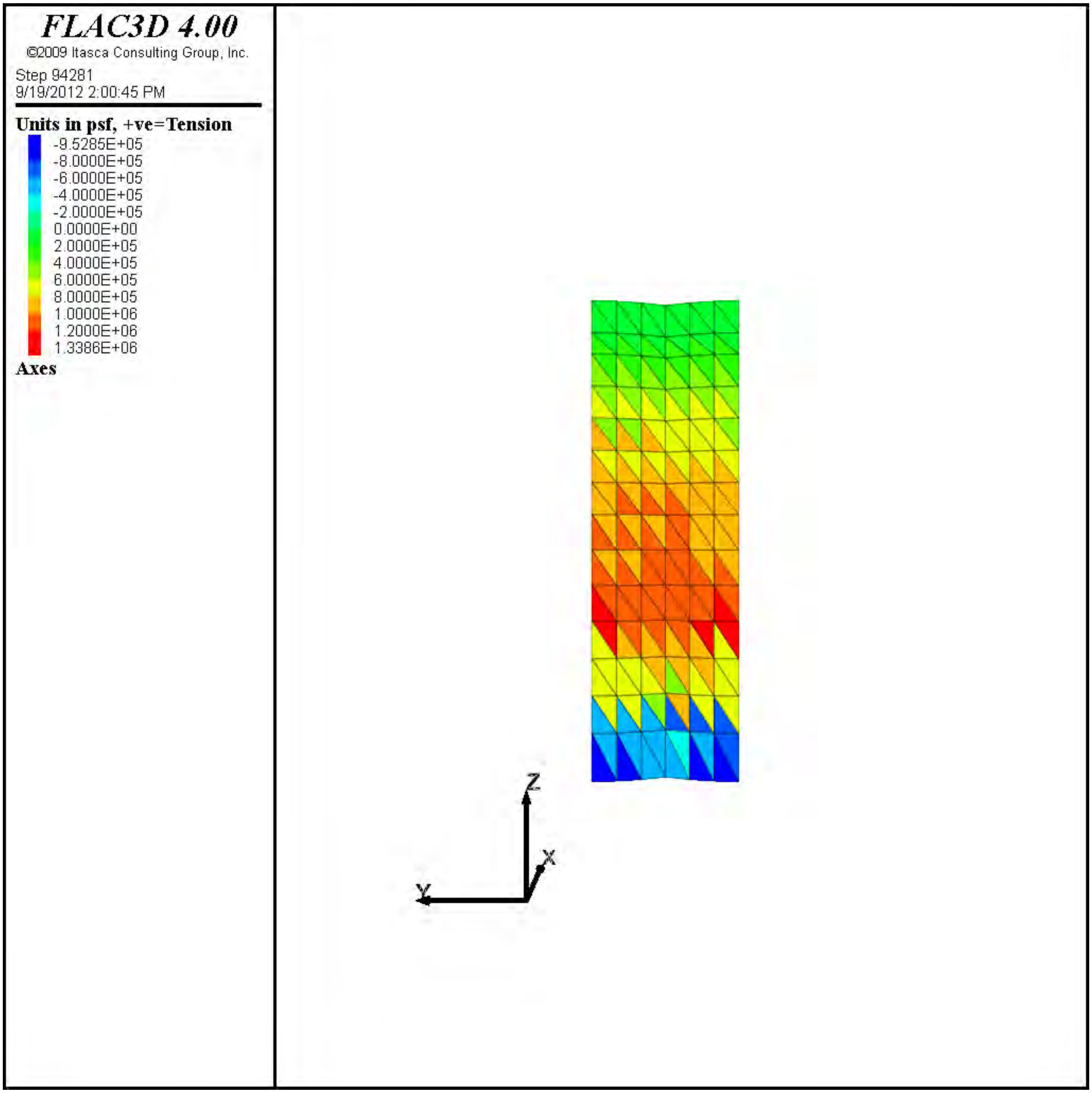


Figure 7.3-11. Facewall Membrane Stress – As-Designed Static Long-Term Undrained

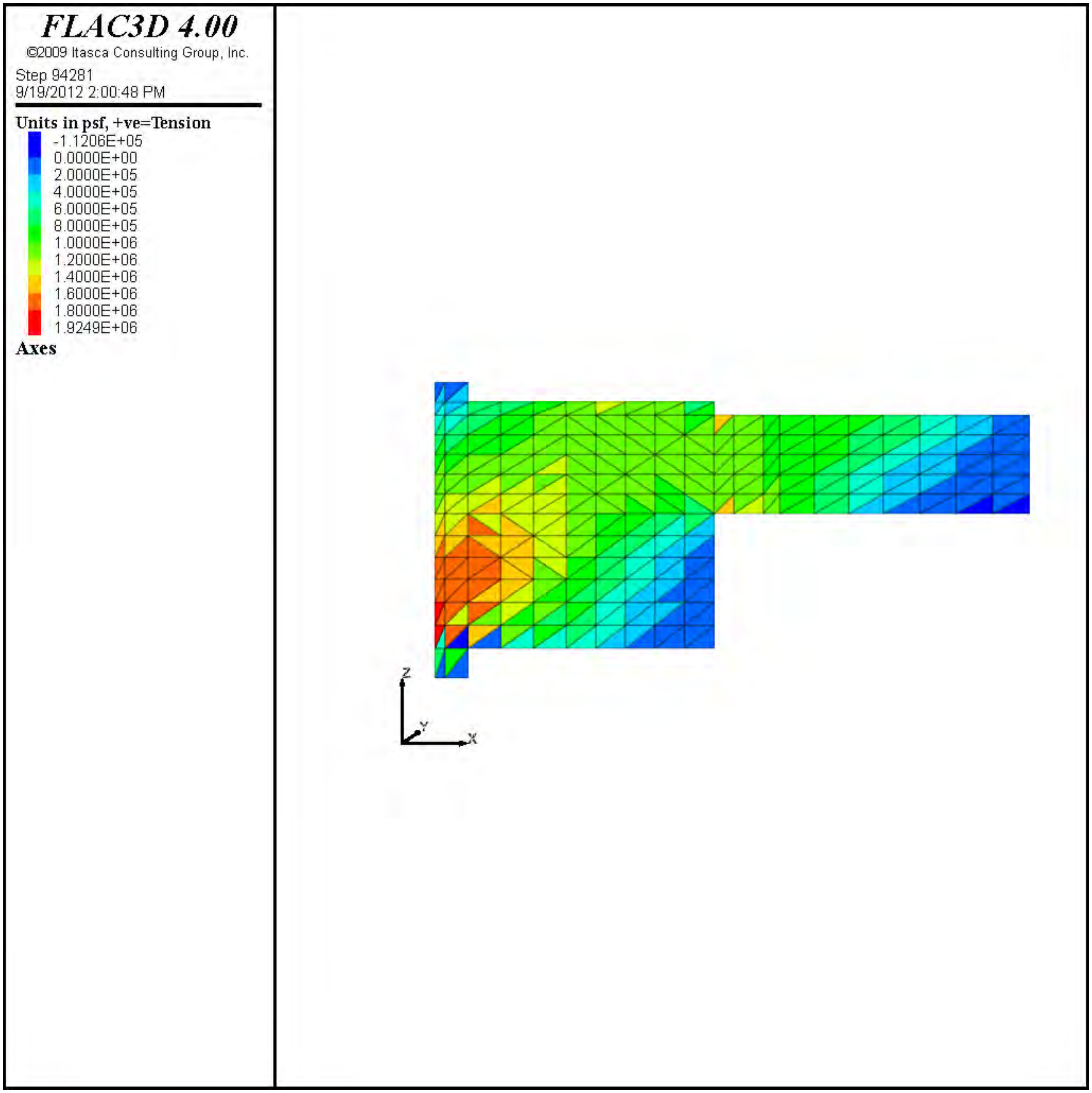


Figure 7.3-12. Tailwall Membrane Stress – As-Designed Static Long-Term Undrained

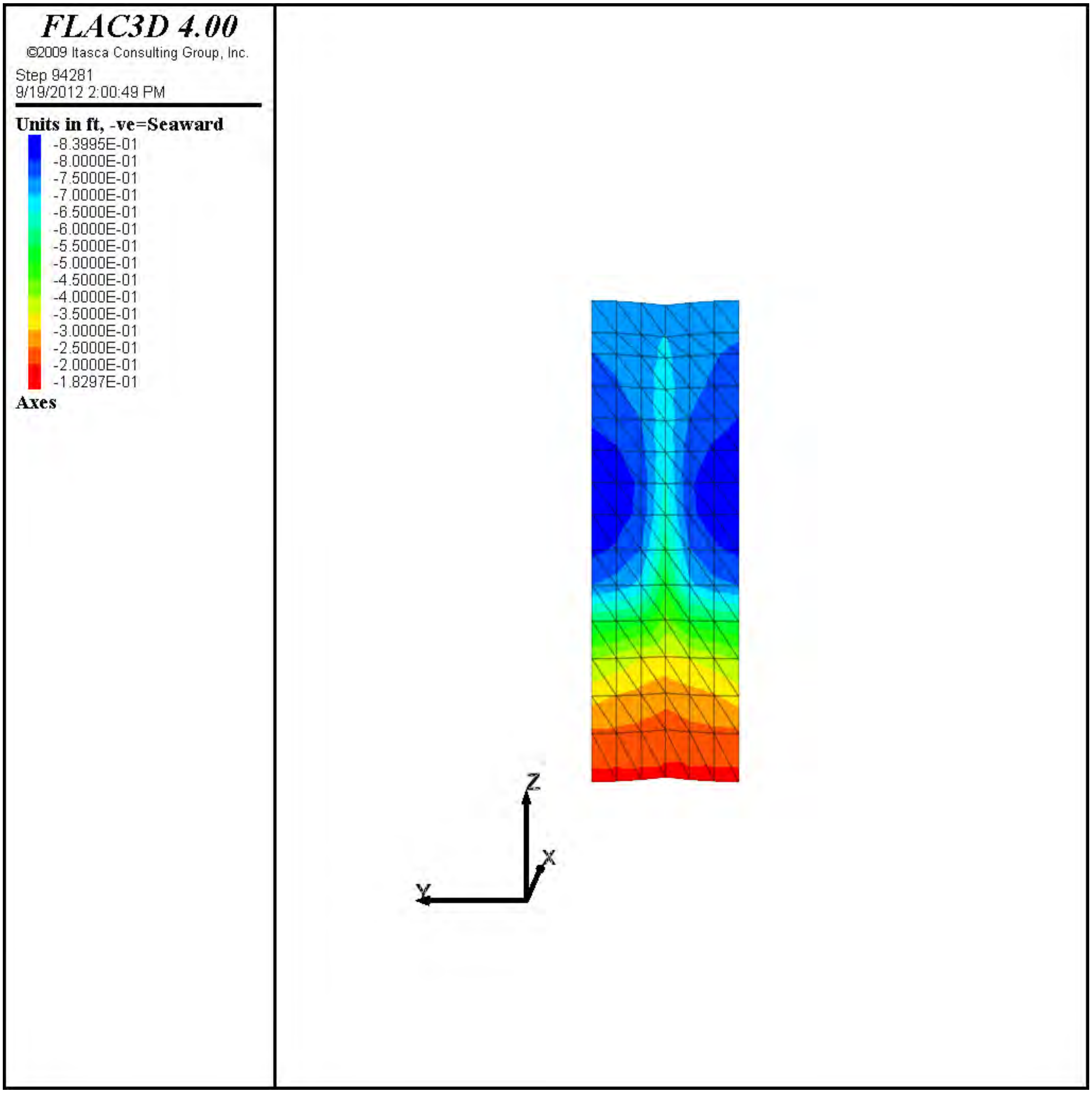


Figure 7.3-13. Facewall X-Displacement Contours – As-Designed Static Long-Term Undrained

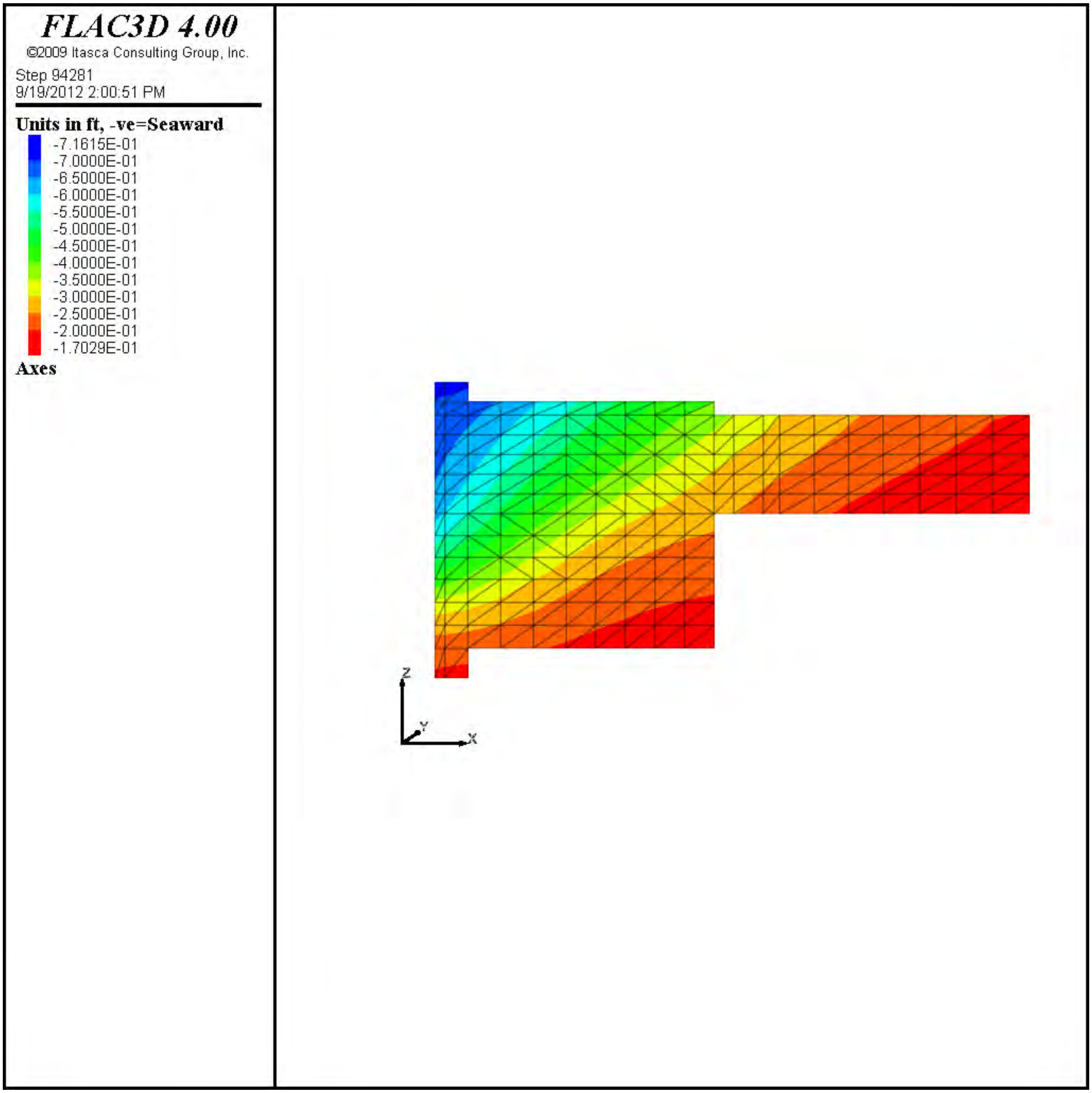


Figure 7.3-14. Tailwall X-Displacement Contours – As-Designed Static Long-Term Undrained

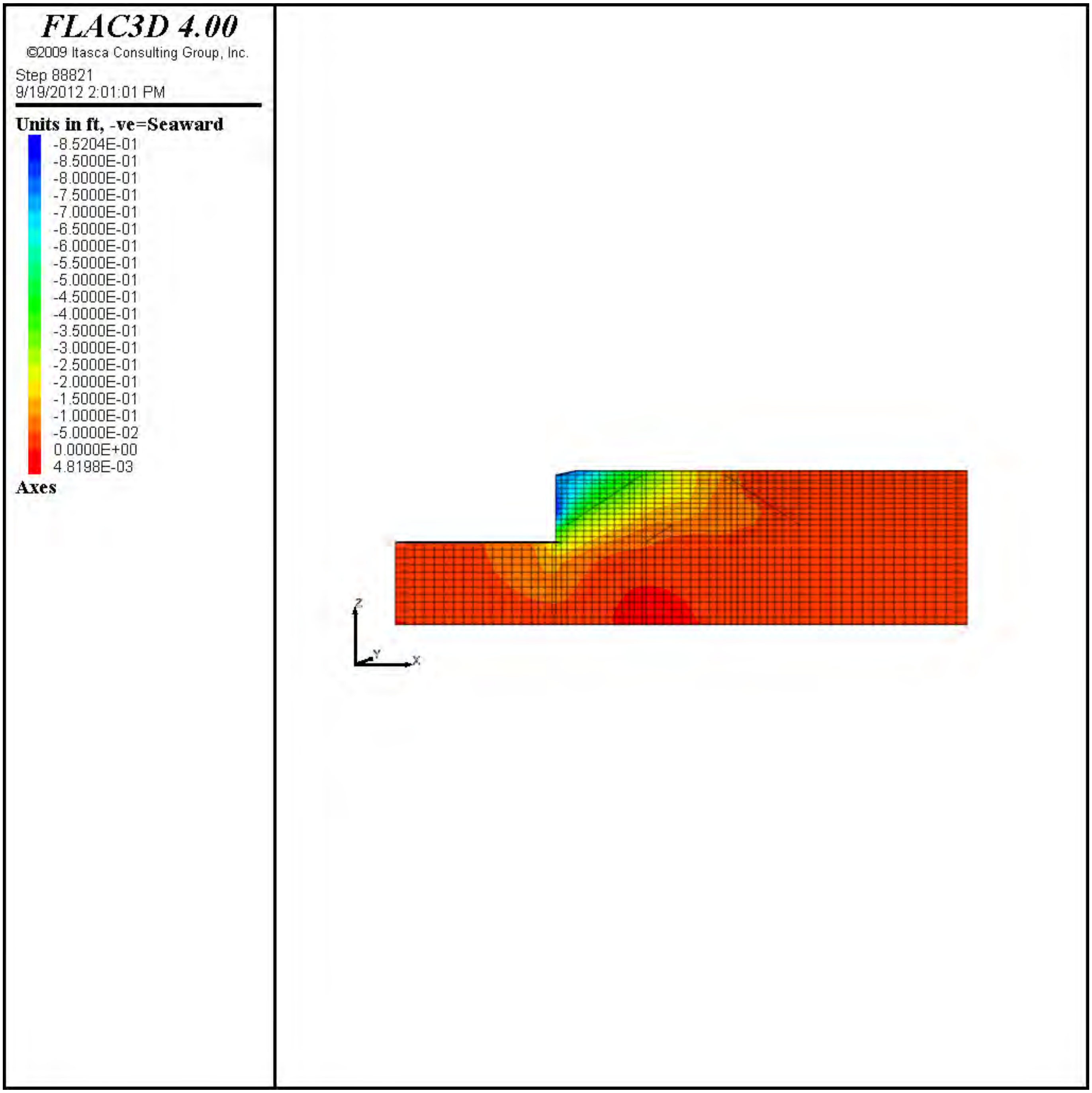


Figure 7.3-15. Soil X-Displacement Contours – As-Designed Static Short-Term

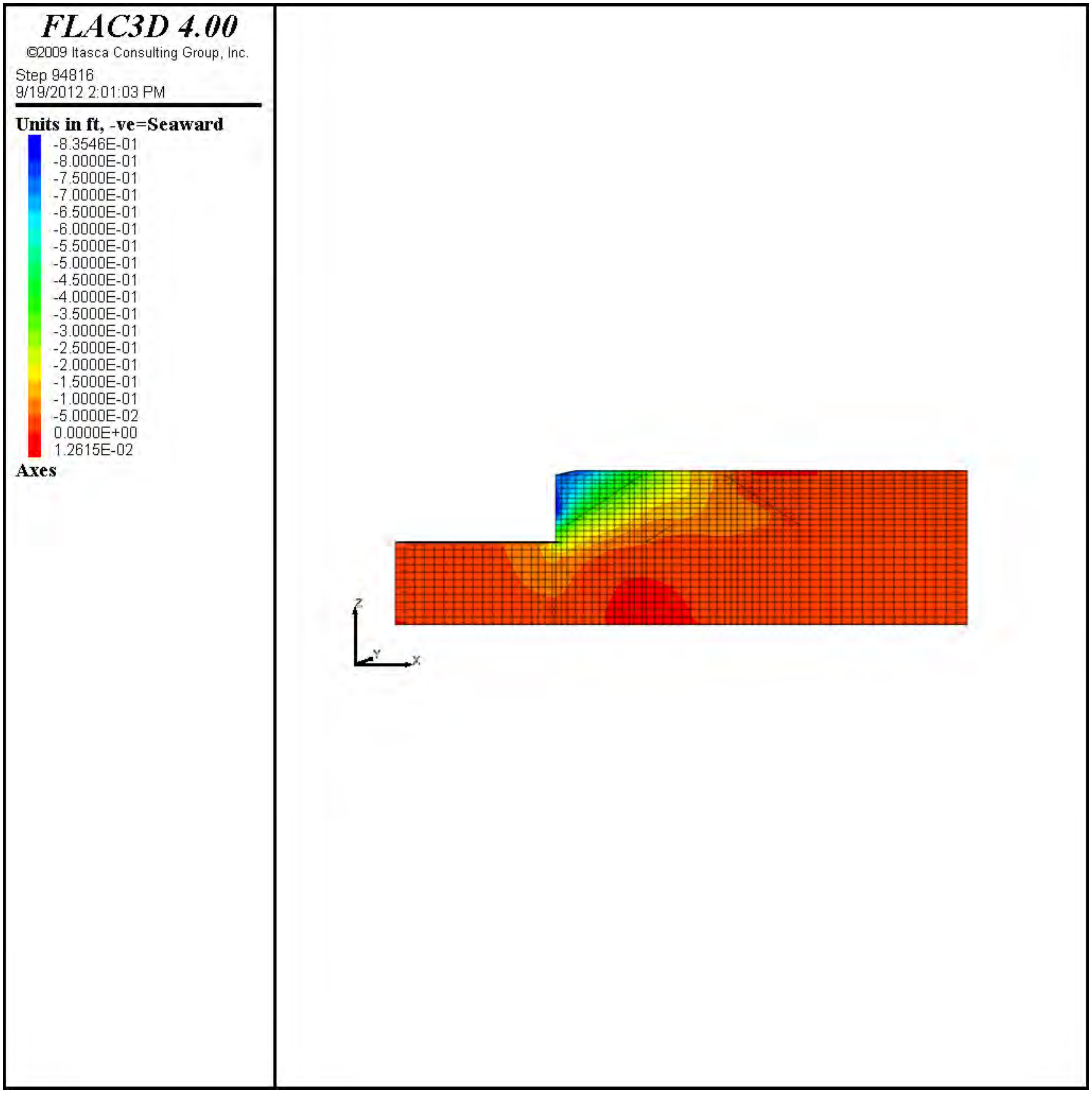


Figure 7.3-16. Soil X-Displacement Contours – As-Designed Static Long-Term Drained

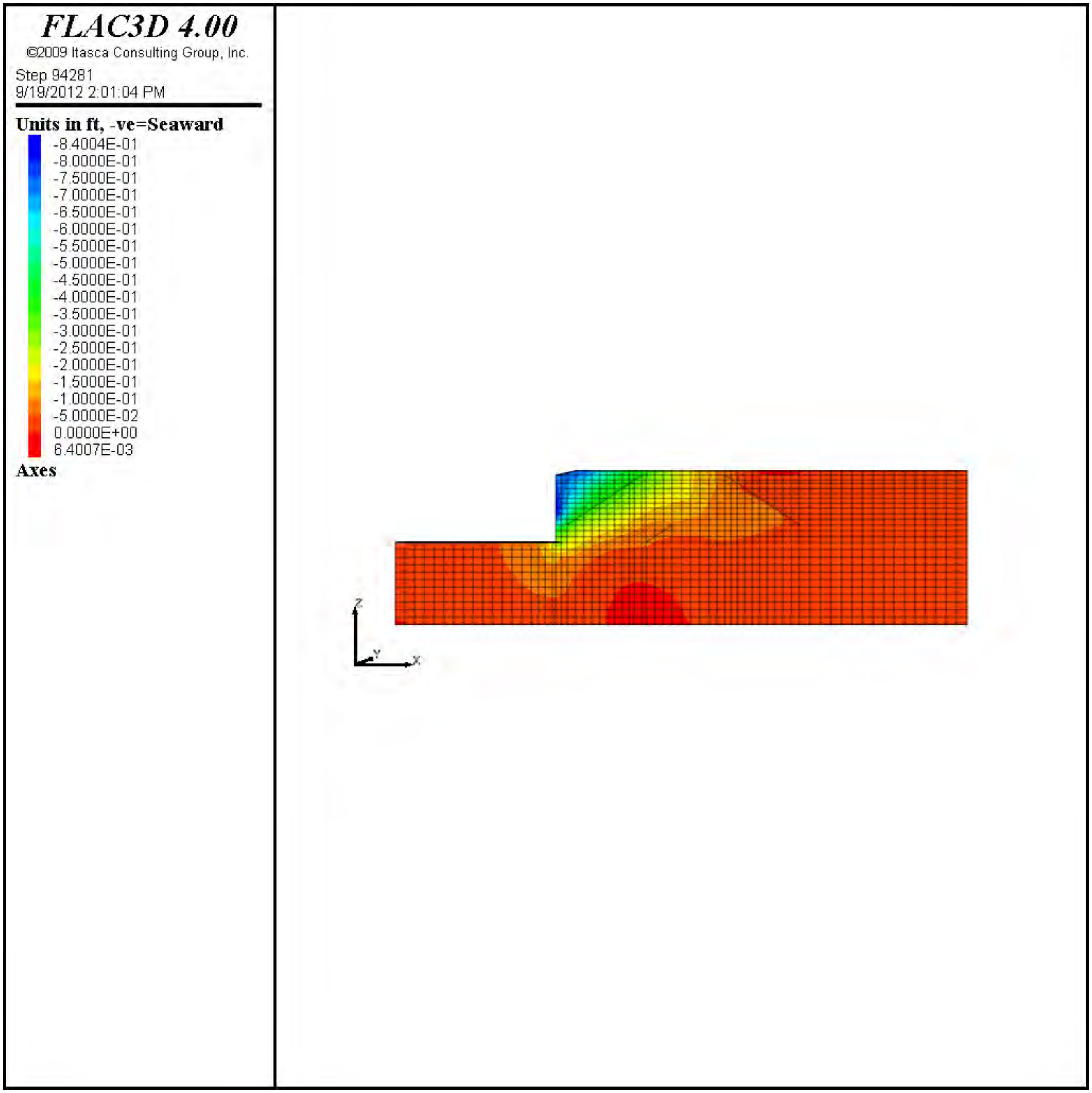


Figure 7.3-17. Soil X-Displacement Contours – As-Designed Static Long-Term Undrained

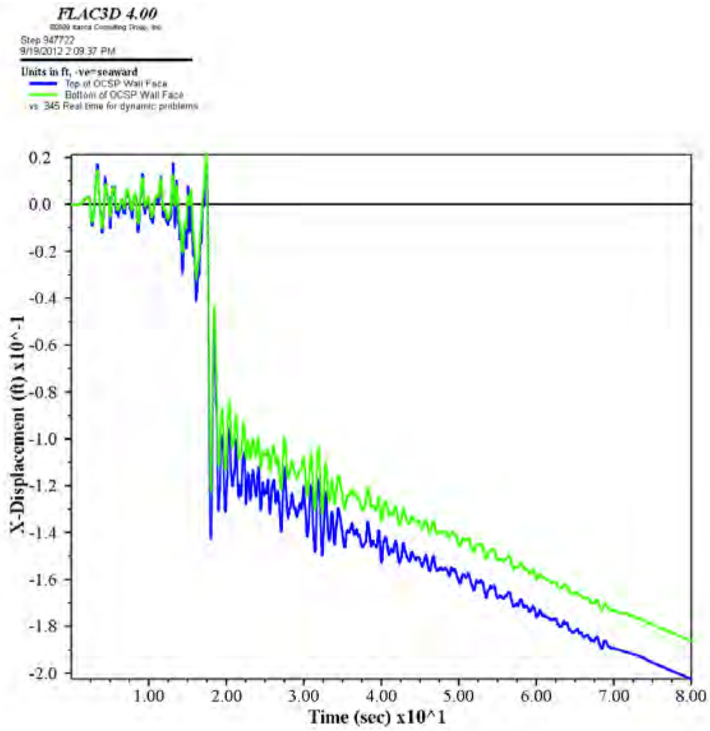


Figure 7.3-18. OLE Facewall X-Displacement-Time History – As-Designed (Michoacan EQ Record)

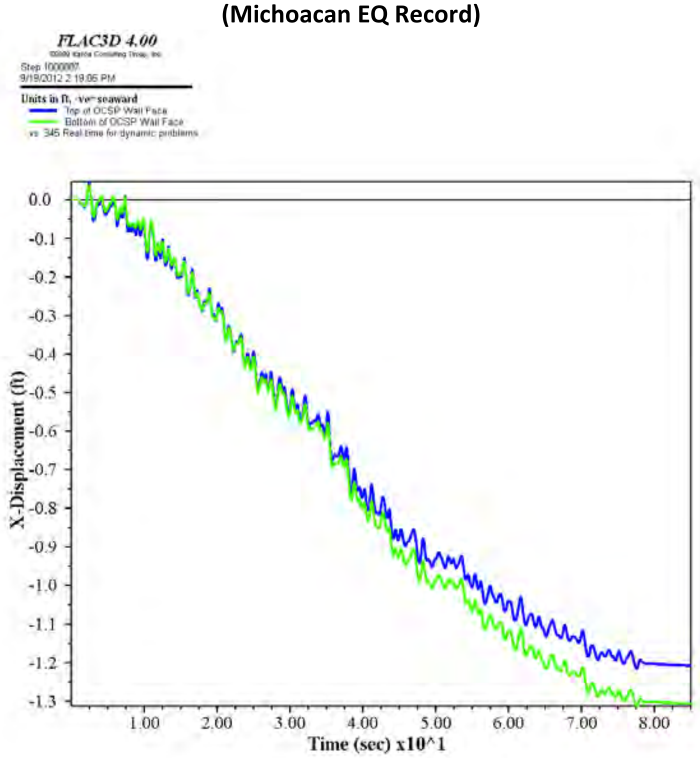


Figure 7.3-19. OLE Facewall X-Displacement-Time History – As-Designed (Puget Sound EQ Record)

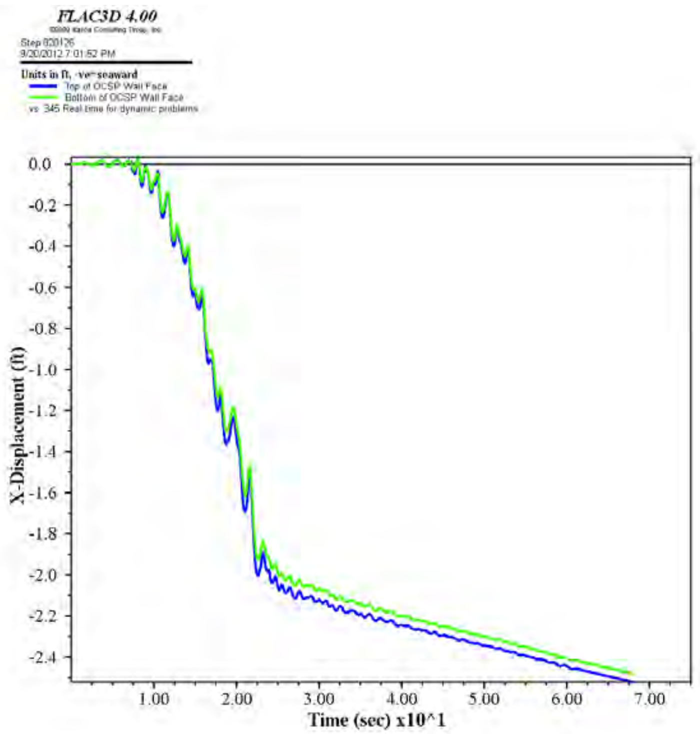


Figure 7.3-20. CLE Facewall X-Displacement-Time History – As-Designed (Michoacan EQ Record)

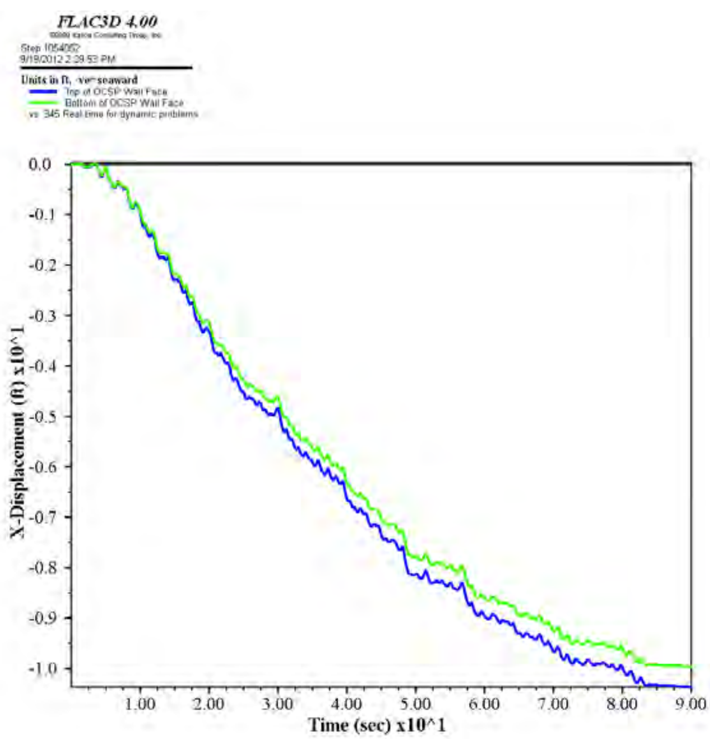


Figure 7.3-21. CLE Facewall X-Displacement-Time History – As-Designed (Western Washington EQ Record)

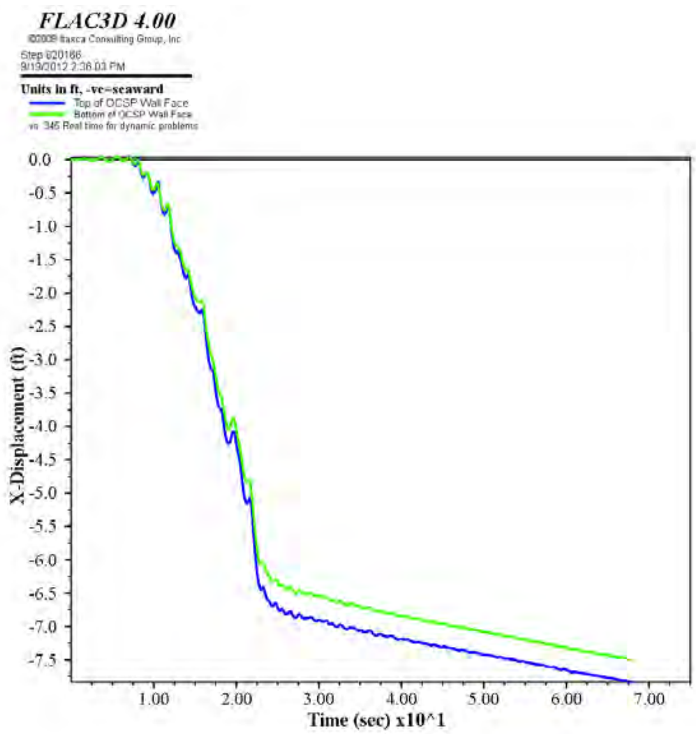


Figure 7.3-22. MCE Facewall X-Displacement-Time History – As-Designed (Michoacan EQ Record)

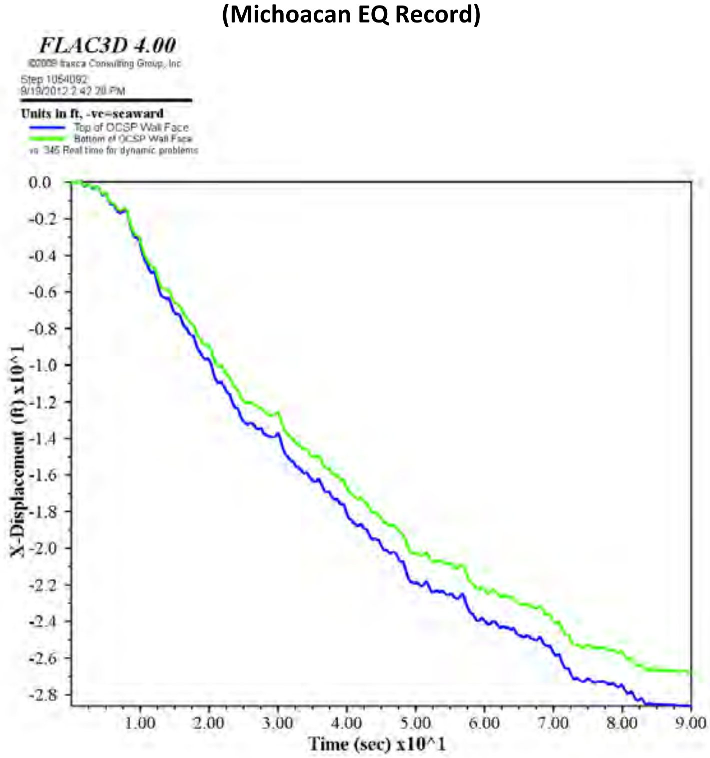


Figure 7.3-23. MCE Facewall X-Displacement-Time History – As-Designed (Western Washington EQ Record)

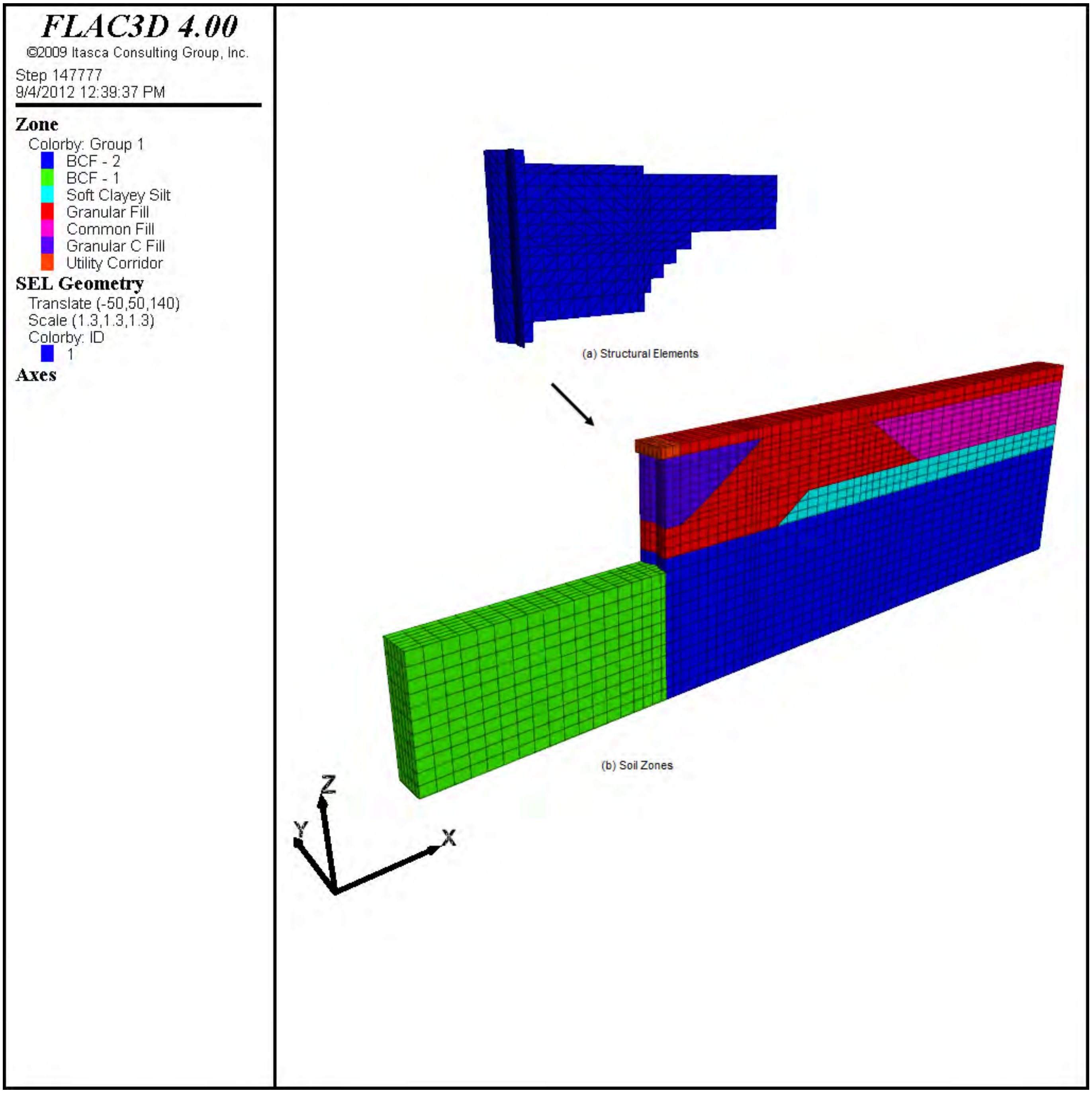


Figure 7.4-1. As-Built FLAC^{3D} Analysis Mesh

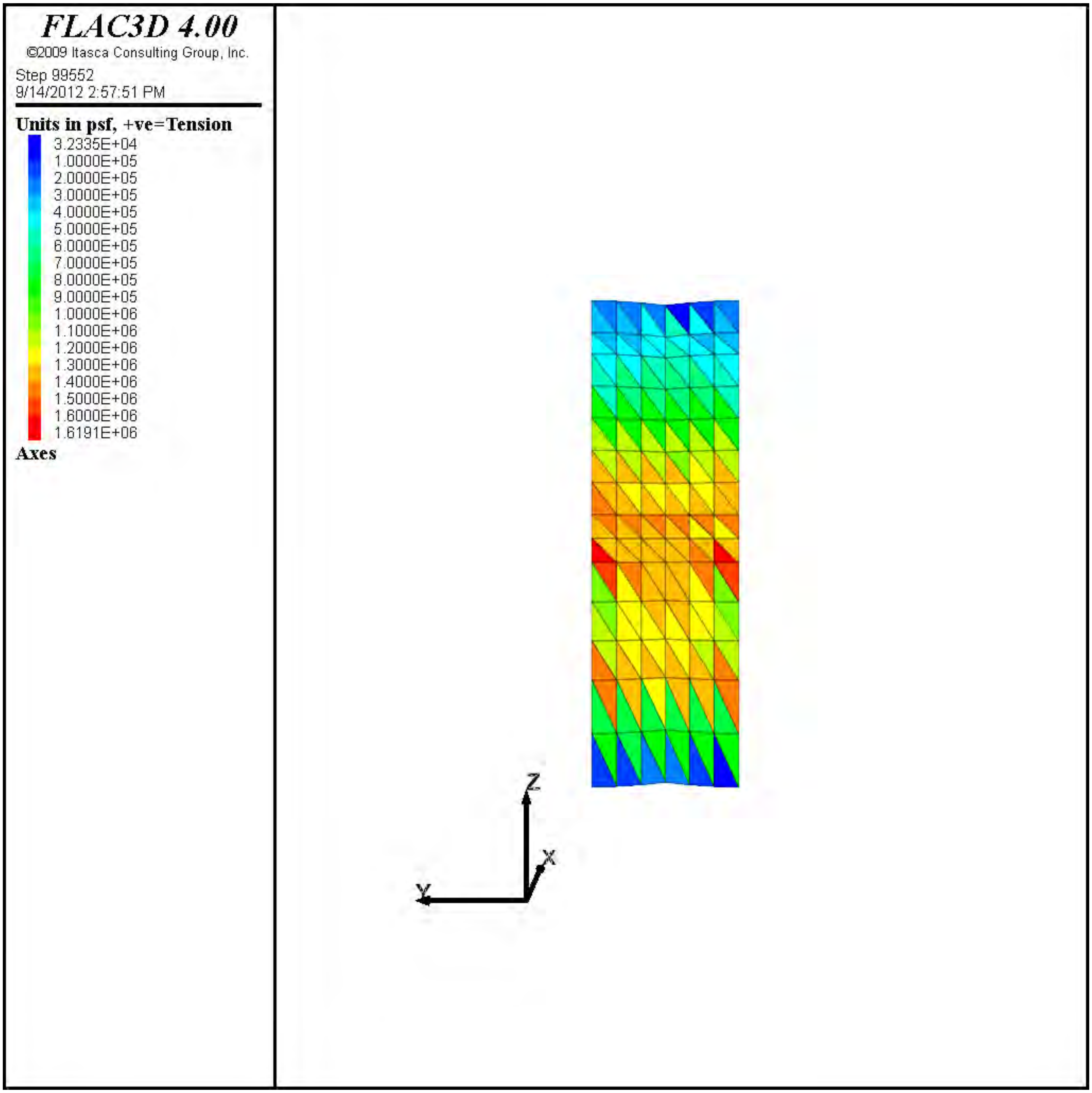


Figure 7.4-2. Facewall Membrane Stresses – As-Built Static Short-Term

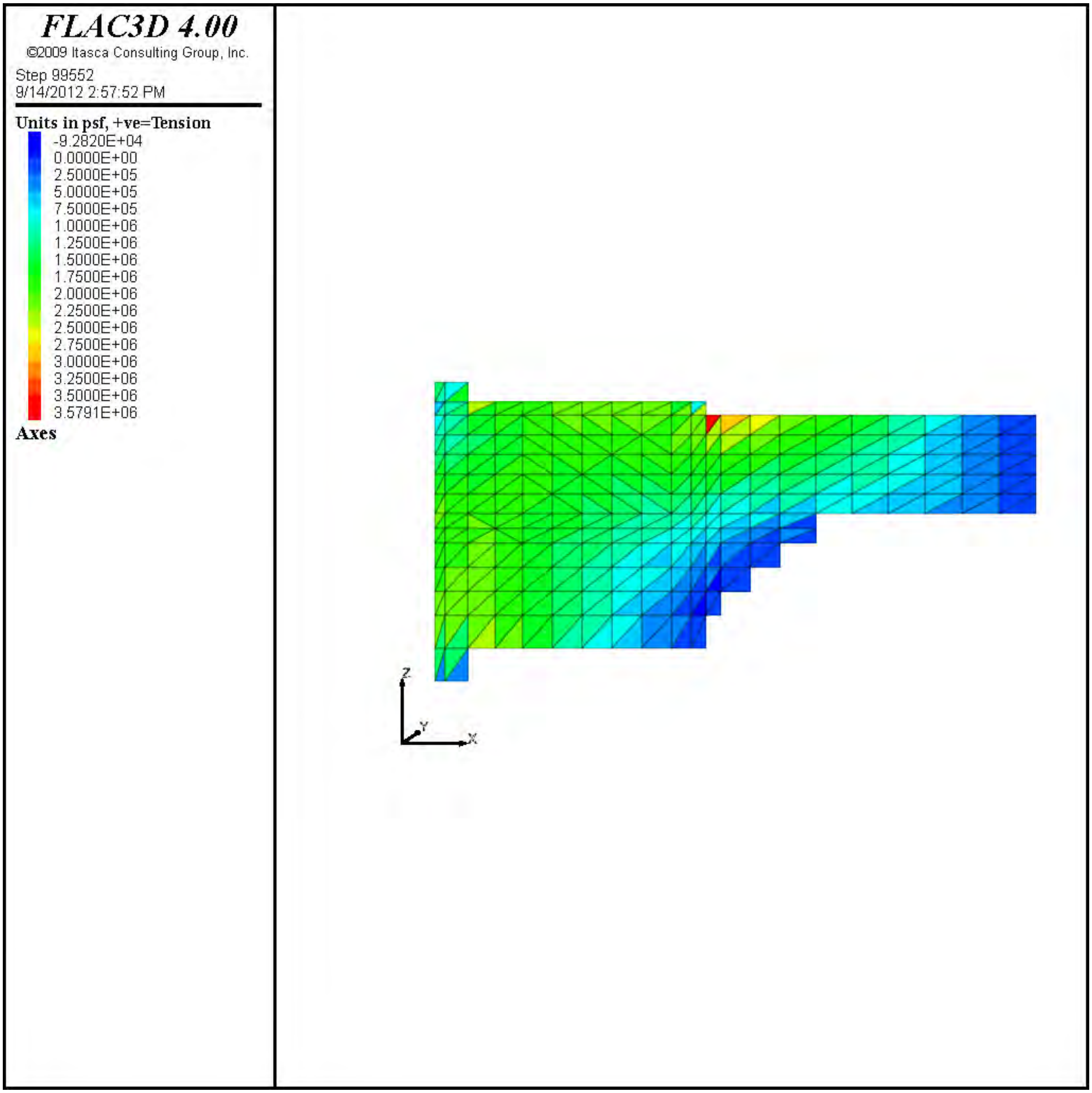


Figure 7.4-3. Tailwall Membrane Stresses – As-Built Static Short-Term

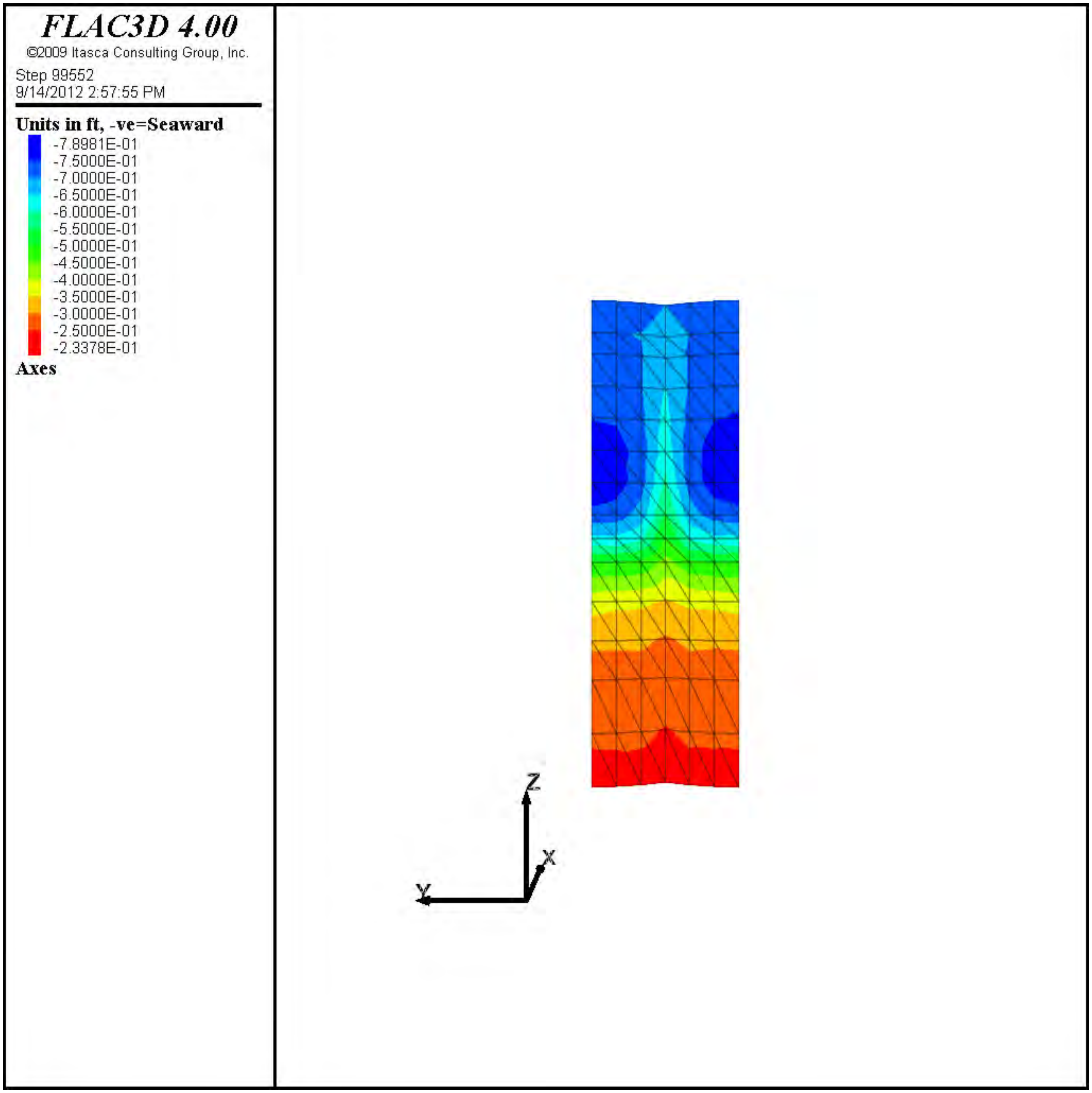


Figure 7.4-4. Facewall X-Displacement Contours – As-Built Static Short-Term

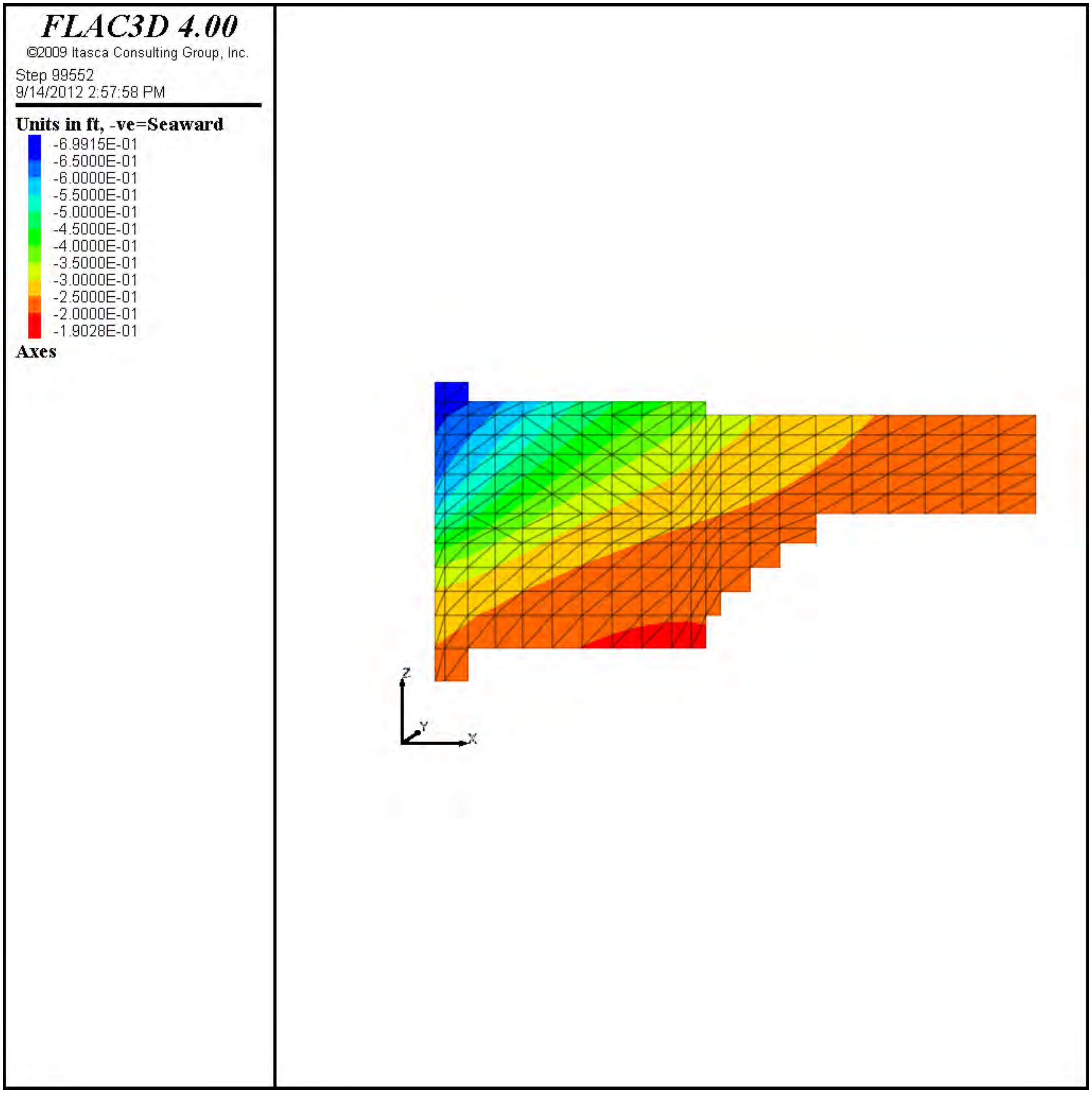


Figure 7.4-5. Tailwall X-Displacement Contours – As-Built Static Short-Term

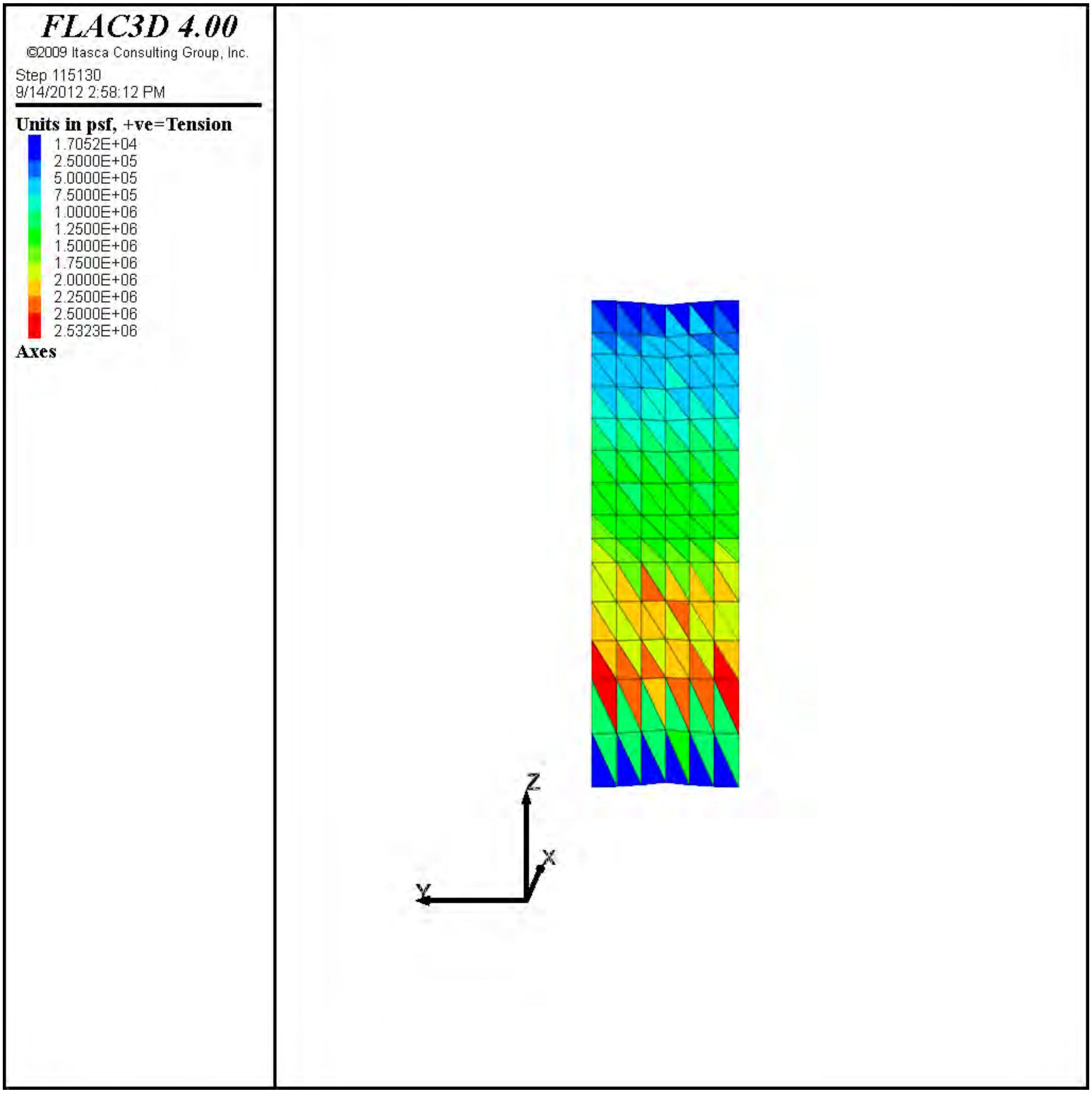


Figure 7.4-6. Facewall Membrane Stress – As-Built Static Long-Term Drained

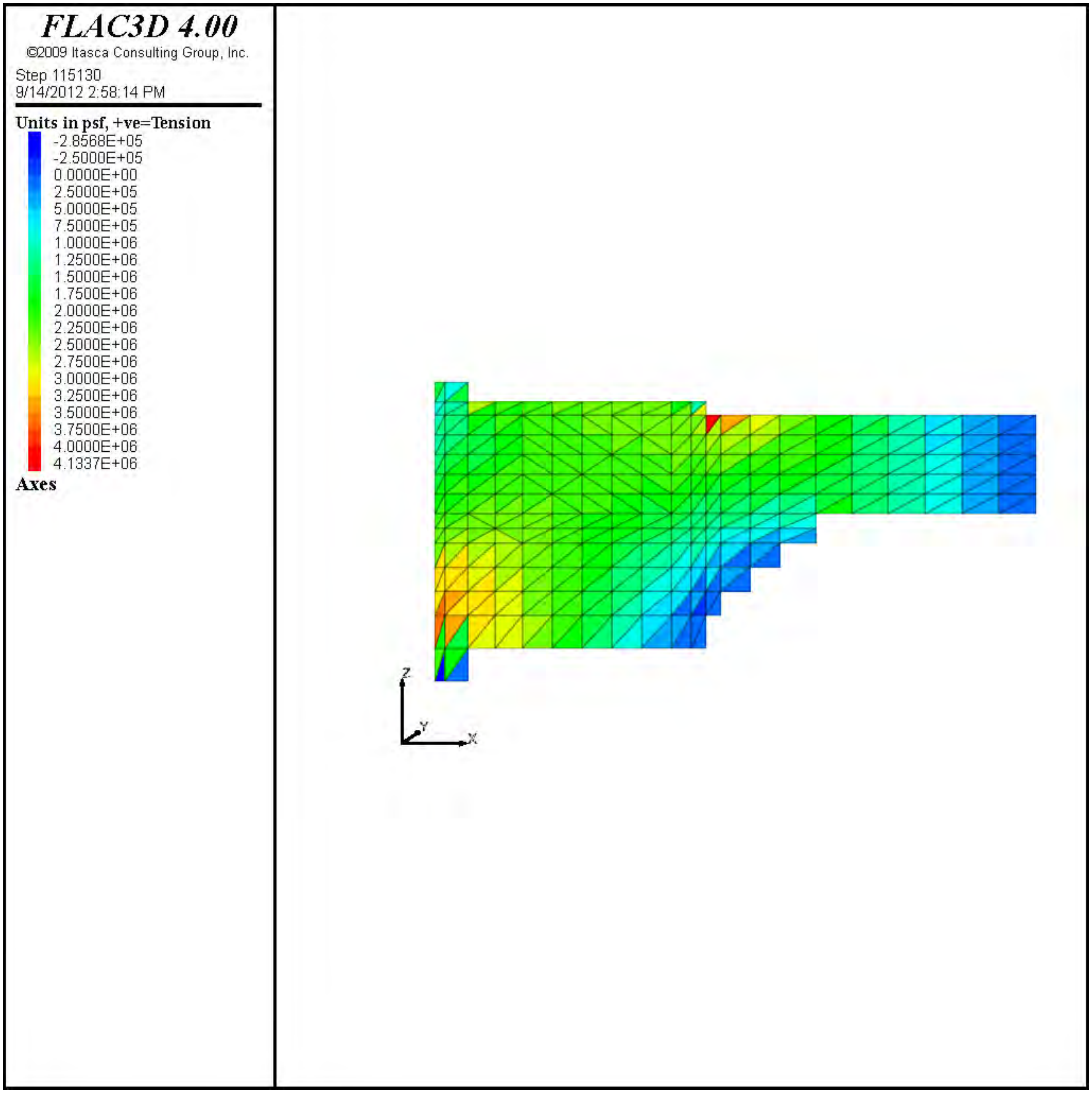


Figure 7.4-7. Tailwall Membrane Stress – As-Built Static Long-Term Drained

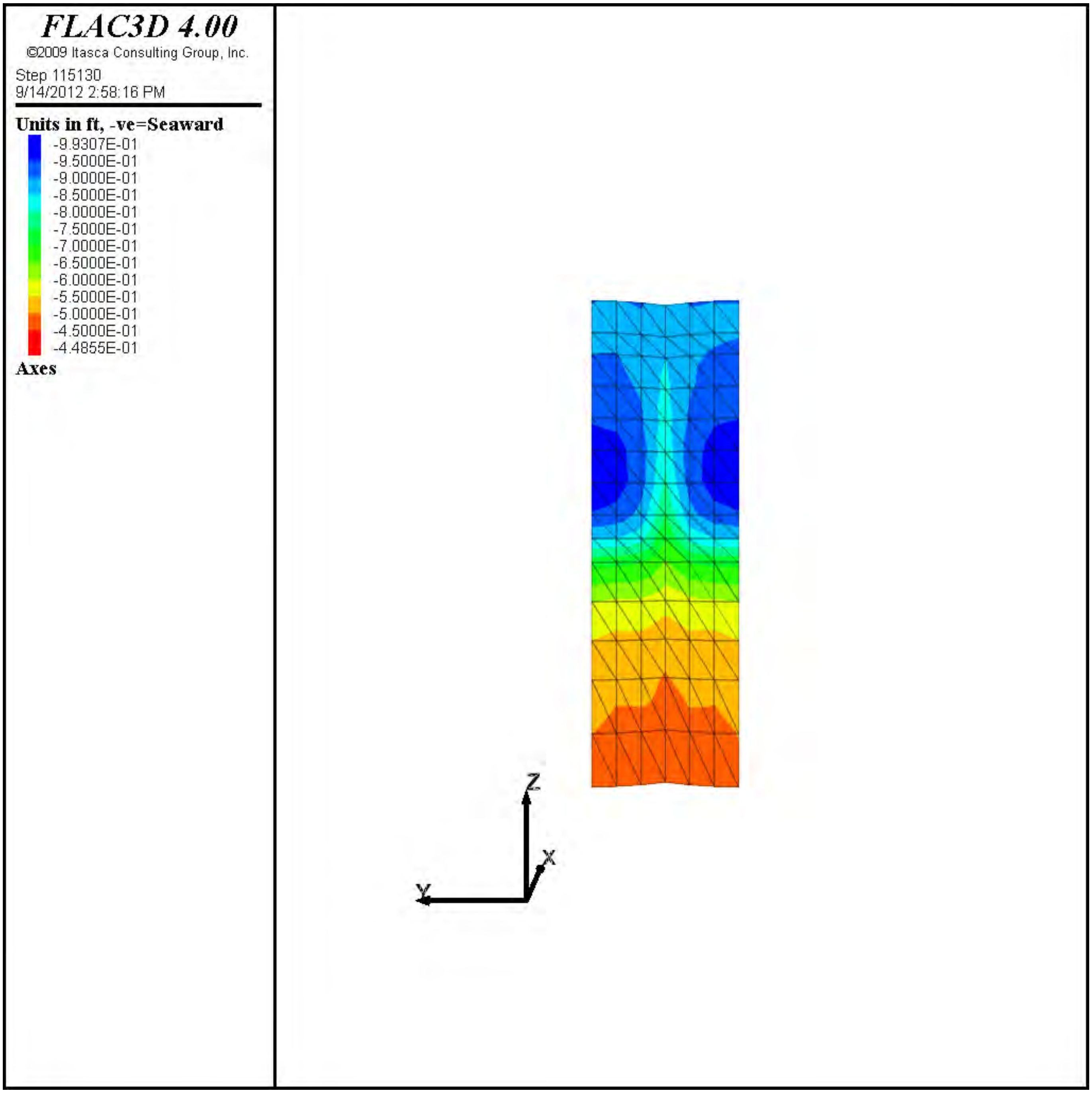


Figure 7.4-8. Facewall X-Displacement Contours – As-Built Static Long-Term Drained

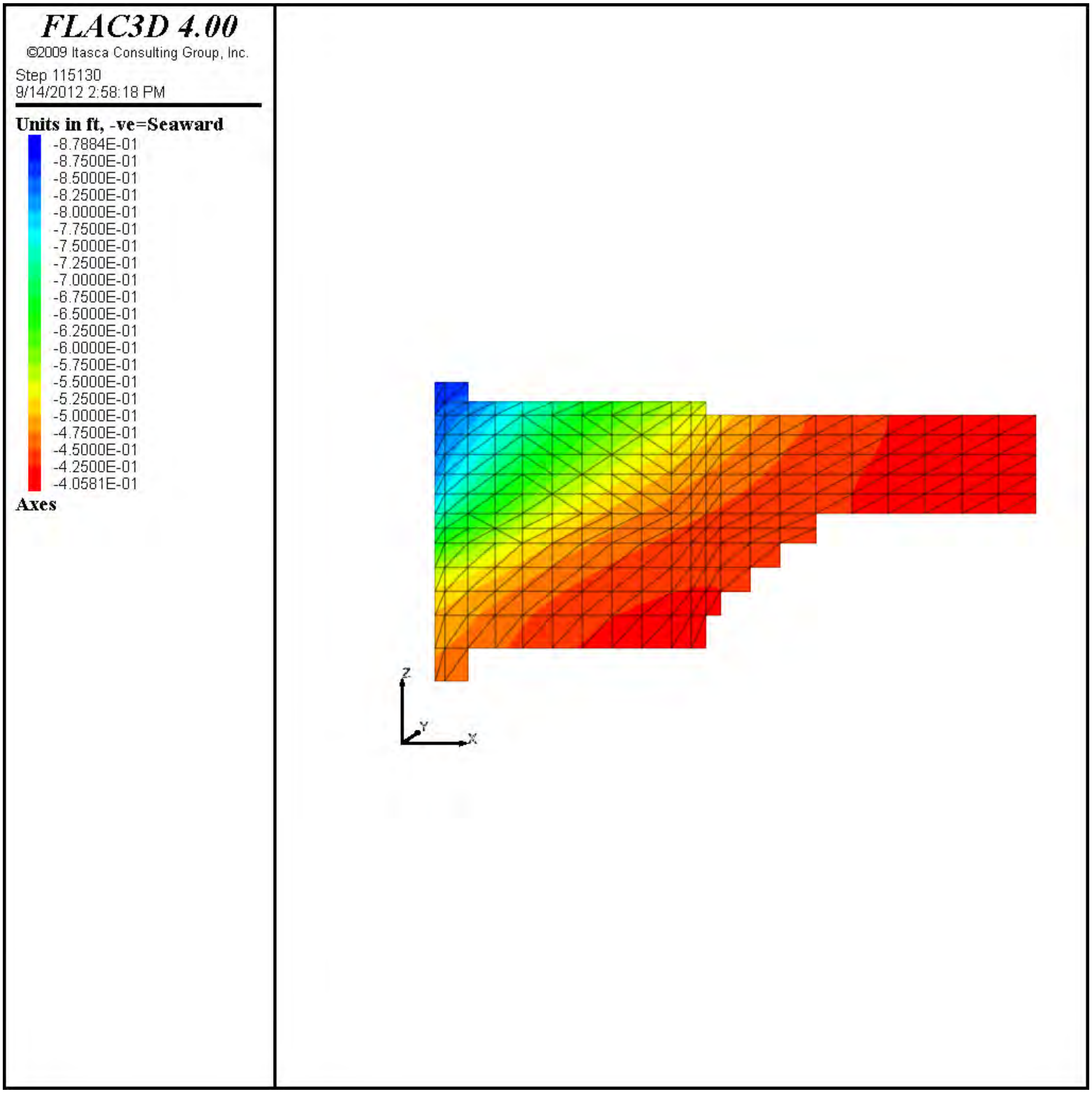


Figure 7.4-9. Tailwall X-Displacement Contours – As-Built Static Long-Term Drained

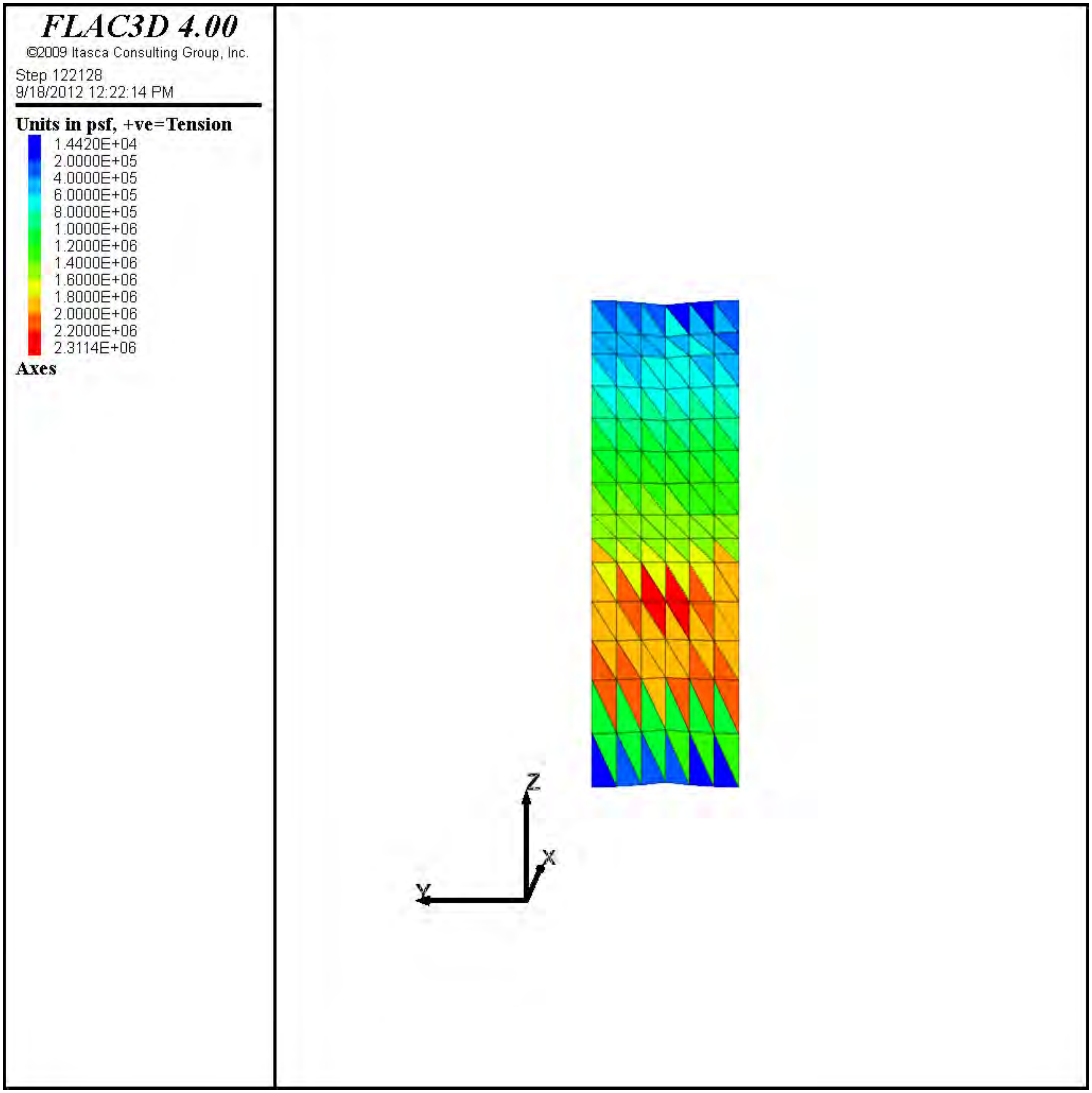


Figure 7.4-10. Facewall Membrane Stress – As-Built Static Long-Term Undrained

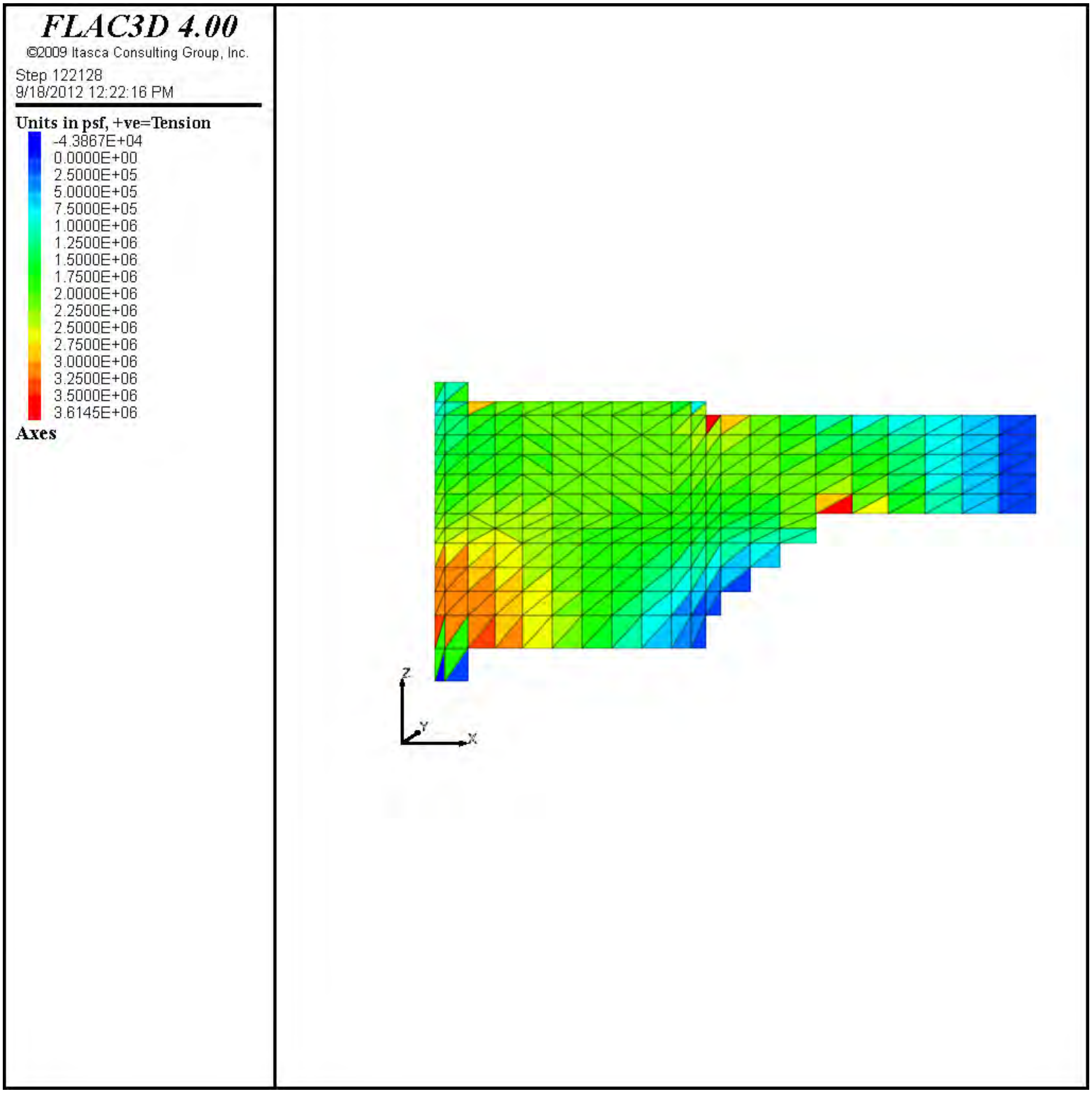


Figure 7.4-11. Tailwall Membrane Stress – As-Built Static Long-Term Undrained

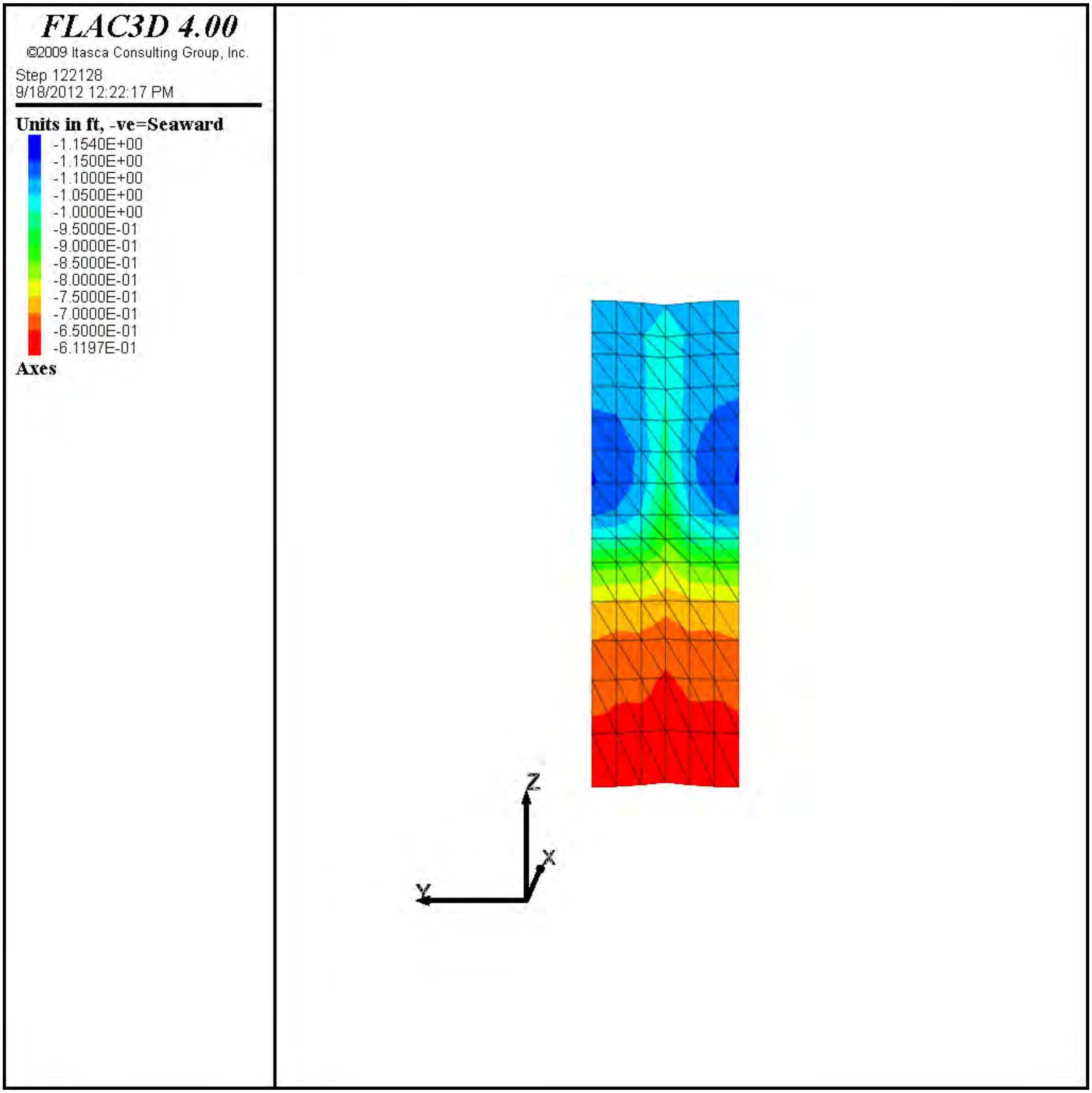


Figure 7.4-12. Facewall X-Displacement Contours – As-Built Static Long-Term Undrained

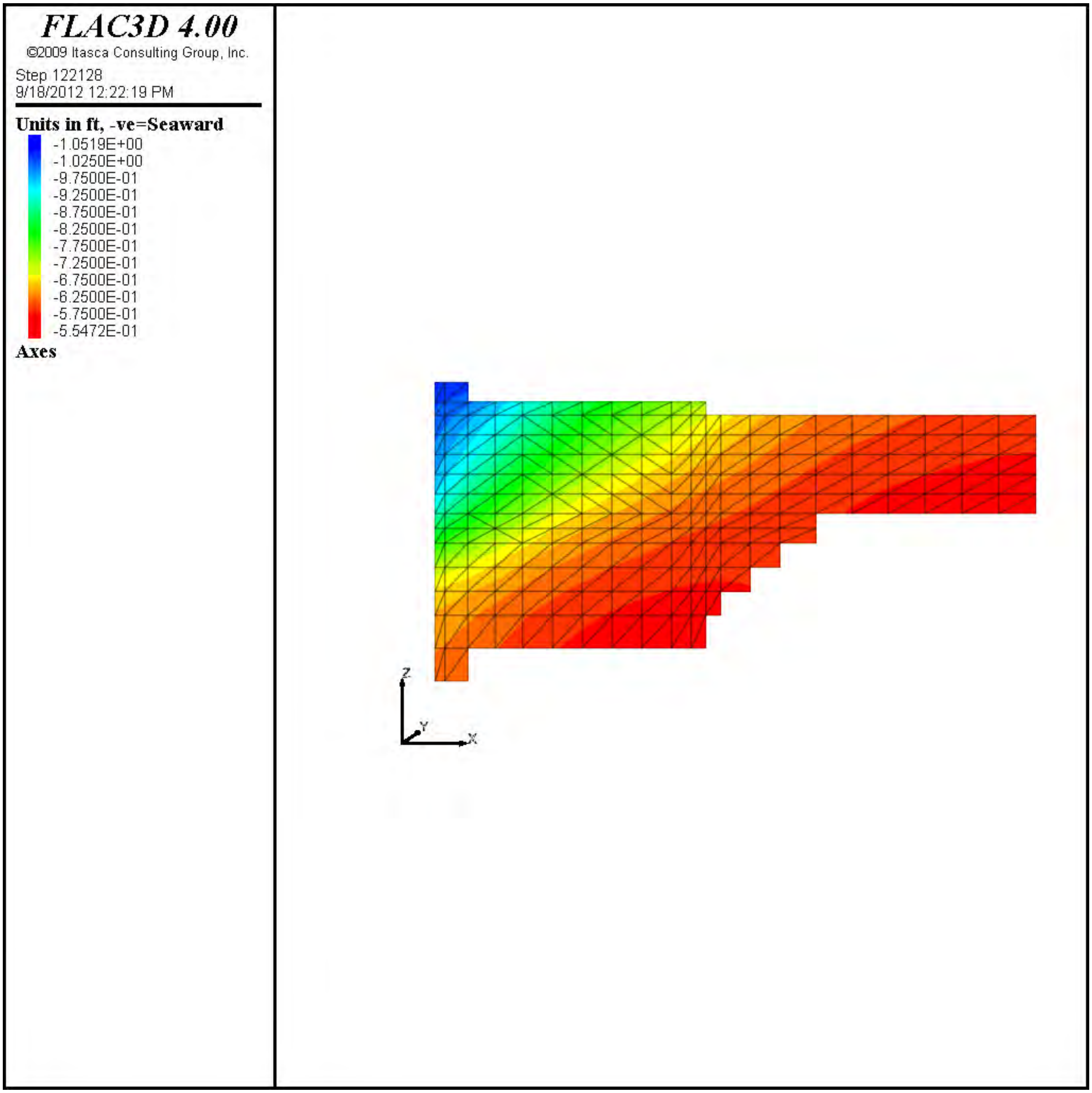


Figure 7.4-13. Tailwall X-Displacement Contours – As-Built Static Long-Term Undrained

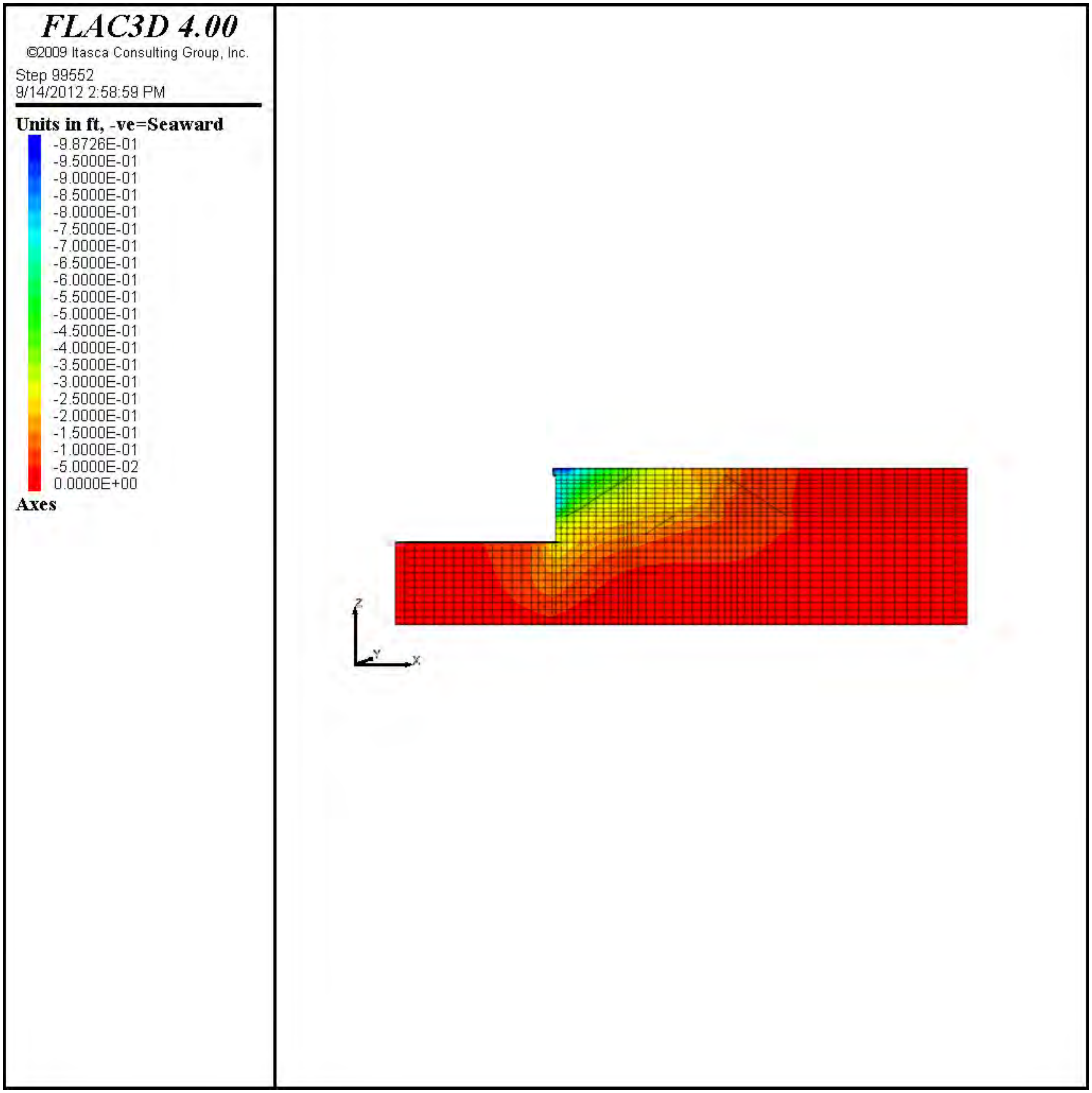


Figure 7.4-14. Soil X-Displacement Contours – As-Built Static Short-Term

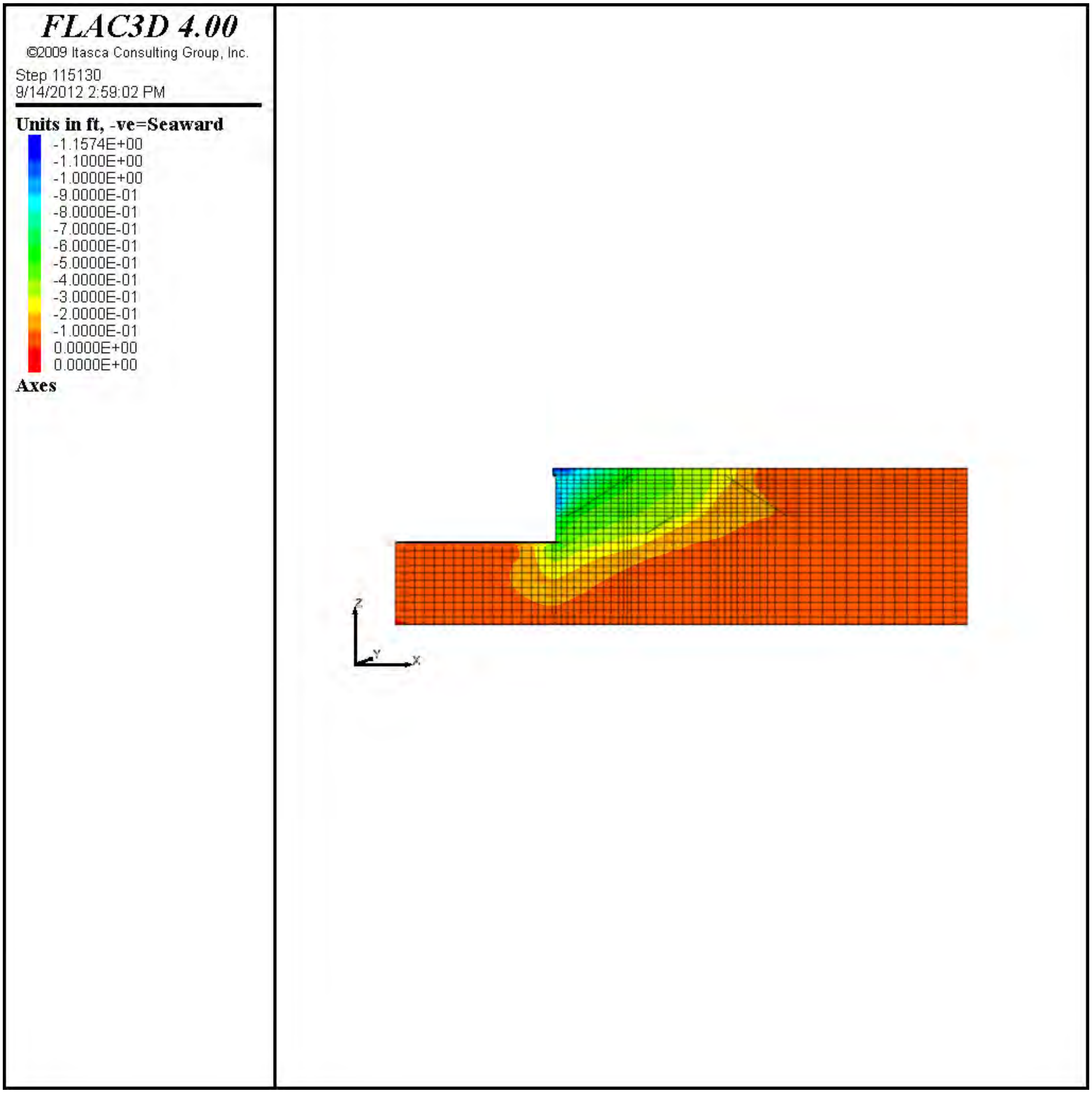


Figure 7.4-15. Soil X-Displacement Contours – As-Built Static Long-Term Drained

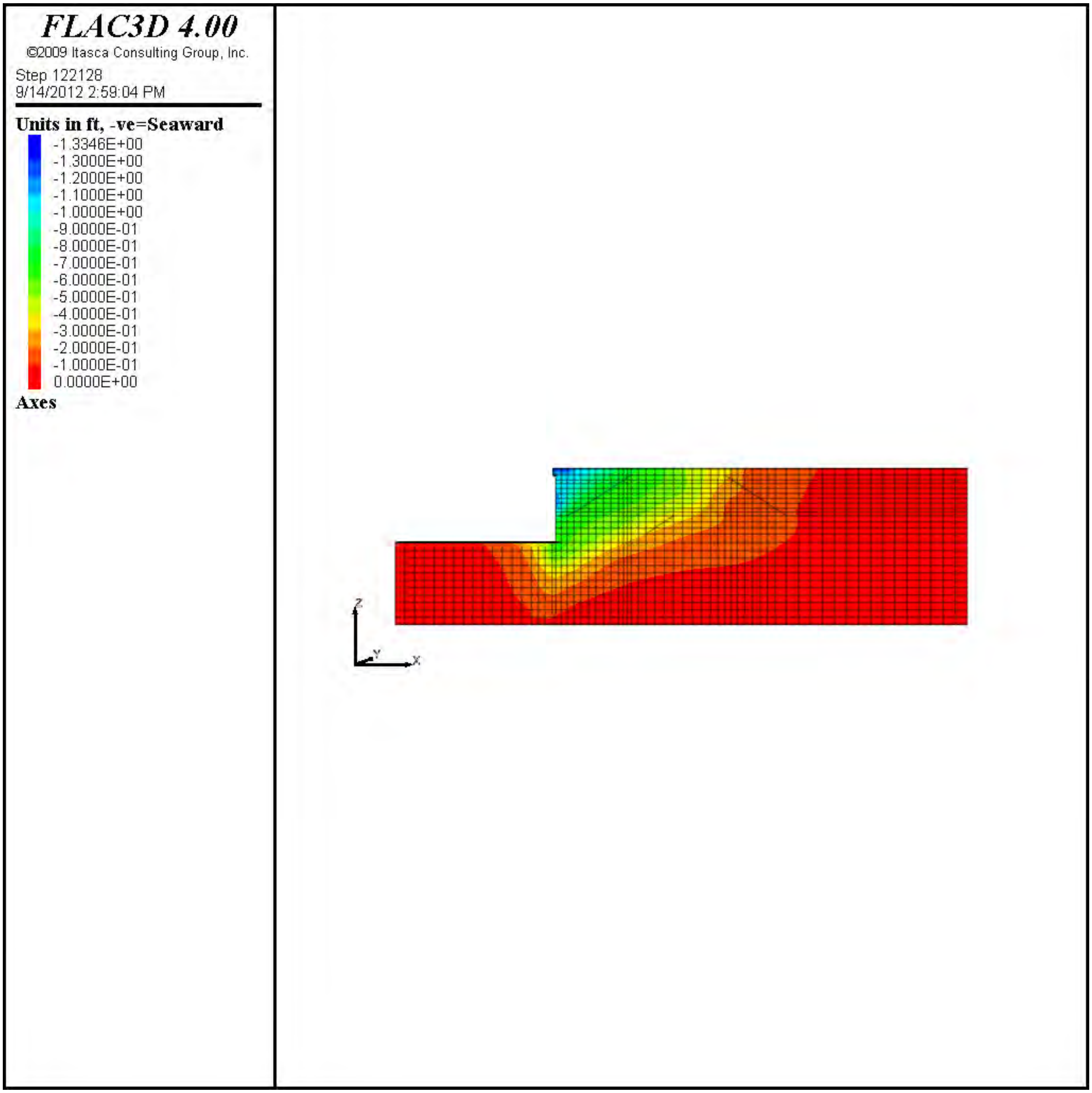


Figure 7.4-16. Soil X-Displacement Contours – As-Built Static Long-Term Undrained

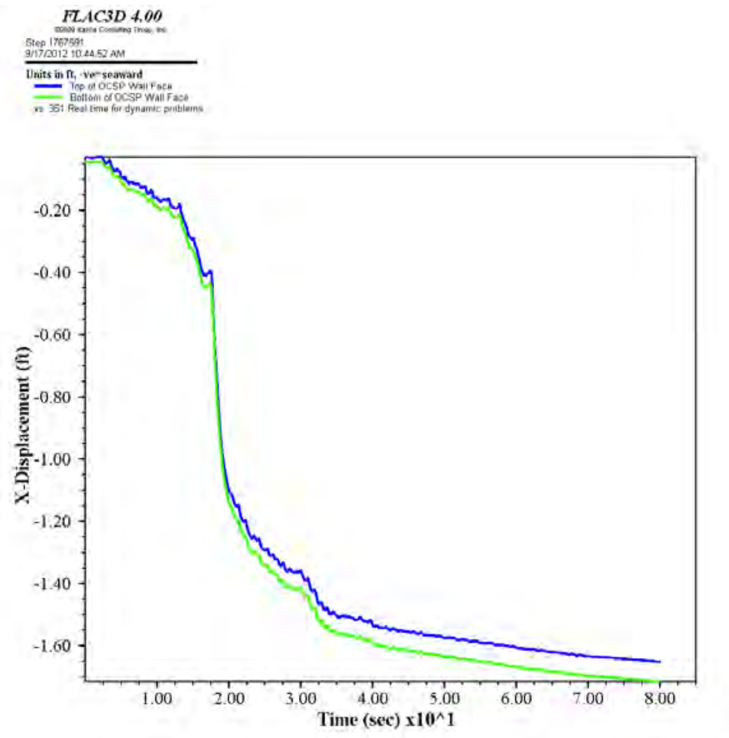


Figure 7.4-17. OLE Facewall X-Displacement-Time History – As-Built (Michoacan EQ Record)

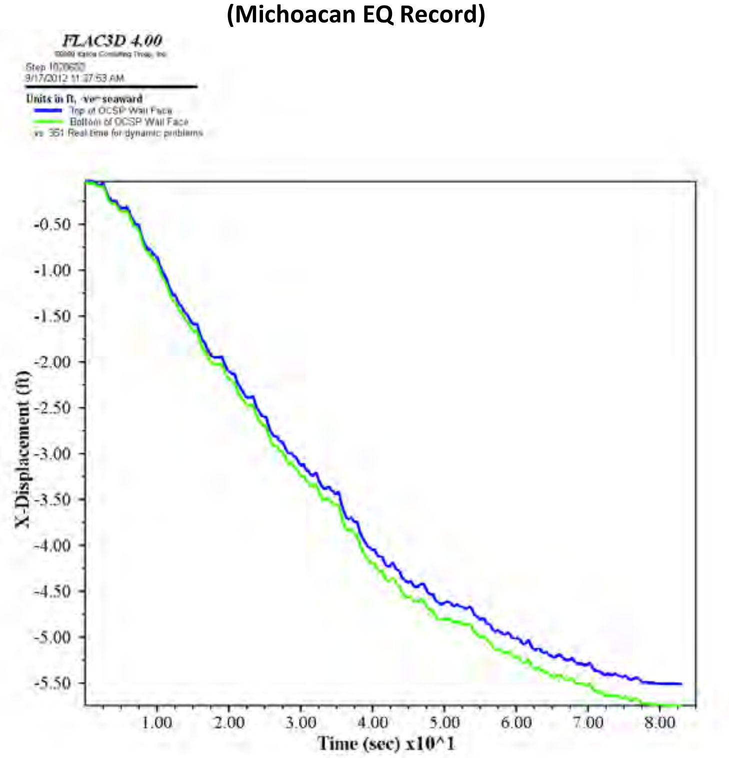


Figure 7.4-18. OLE Facewall X-Displacement-Time History – As-Built (Puget Sound EQ Record)

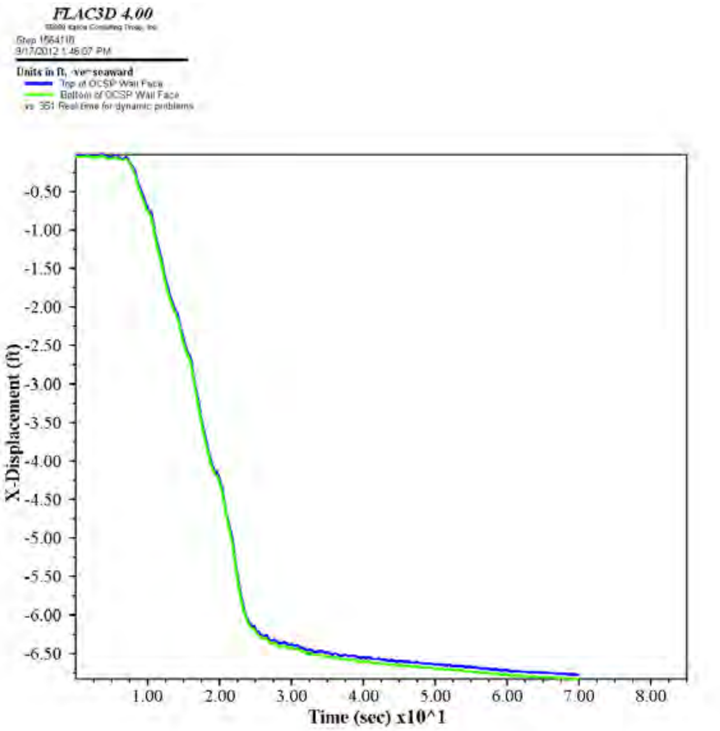


Figure 7.4-19. CLE Facewall X-Displacement-Time History – As-Built (Michoacan EQ Record)

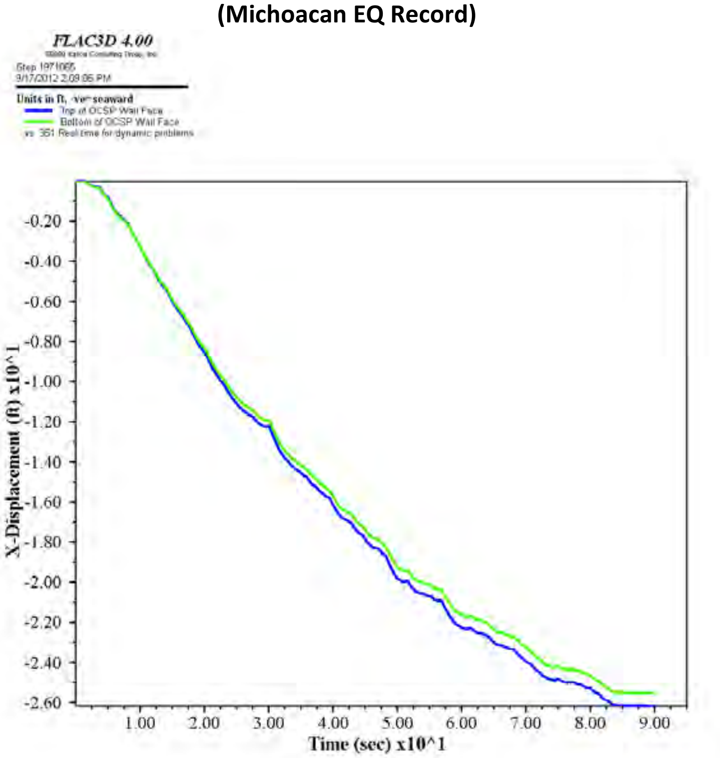


Figure 7.4-20. CLE Facewall X-Displacement-Time History – As-Built (Western Washington EQ Record)

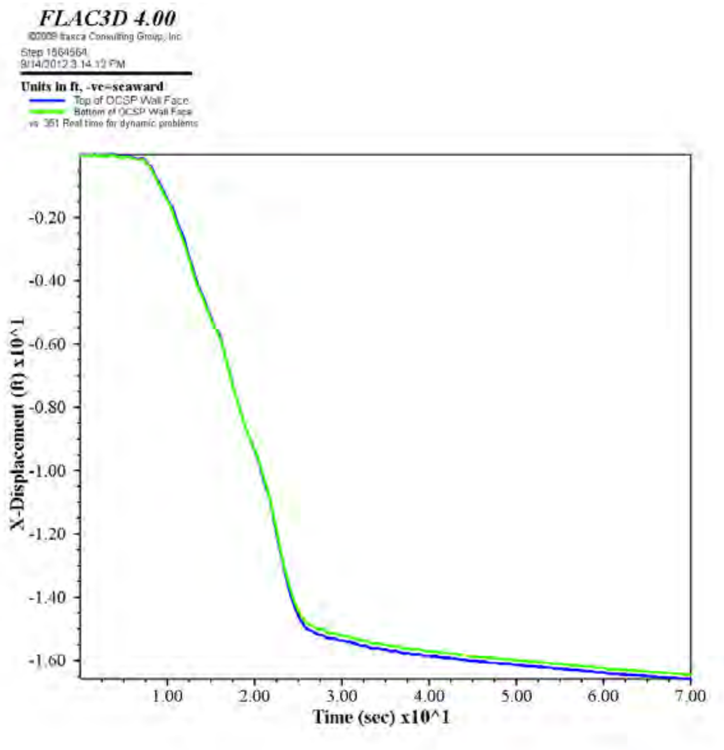


Figure 7.4-21. MCE Facewall X-Displacement-Time History – As-Built (Michoacan EQ Record)

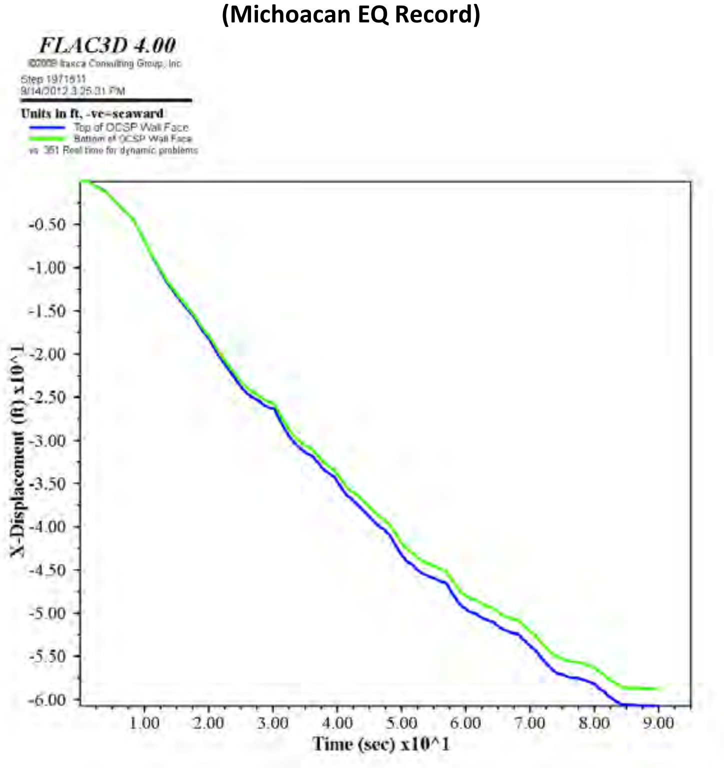


Figure 7.4-22. MCE Facewall X-Displacement-Time History – As-Built (Western Washington EQ Record)

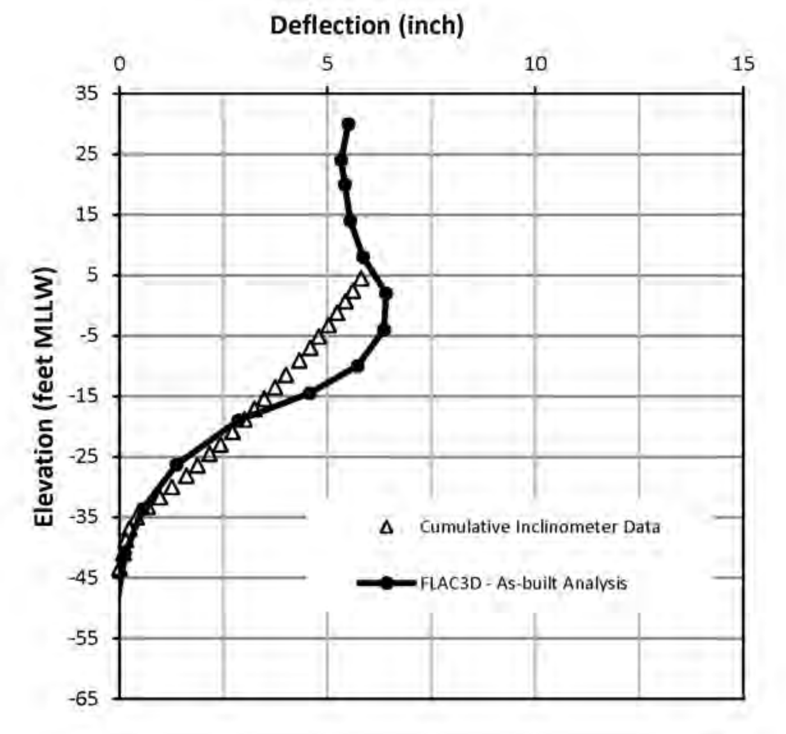


Figure 7.4-23. Deflected Wall Shape (Dredge Elevation -41 Feet)

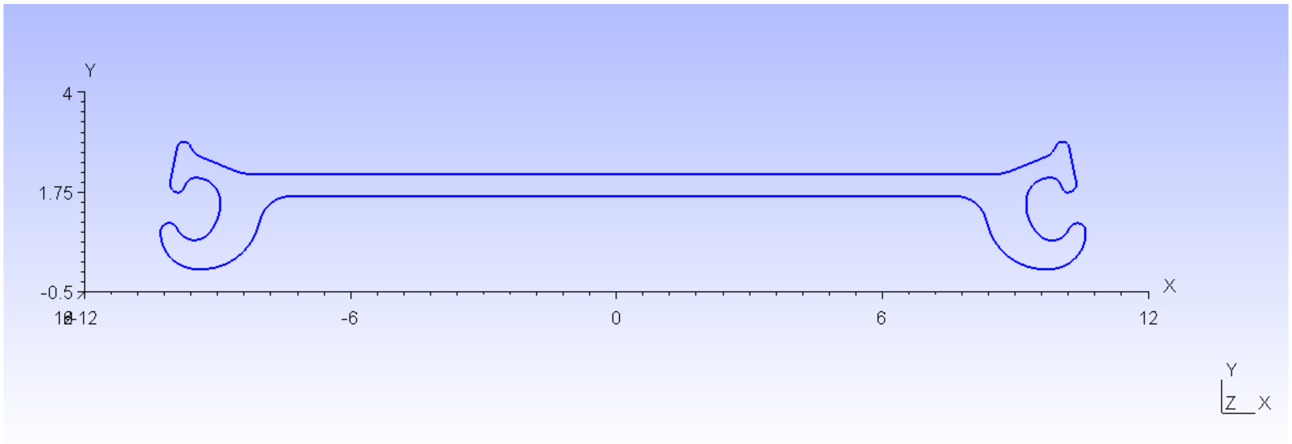


FIGURE 7.5-1. PS31 Sheet Pile Section

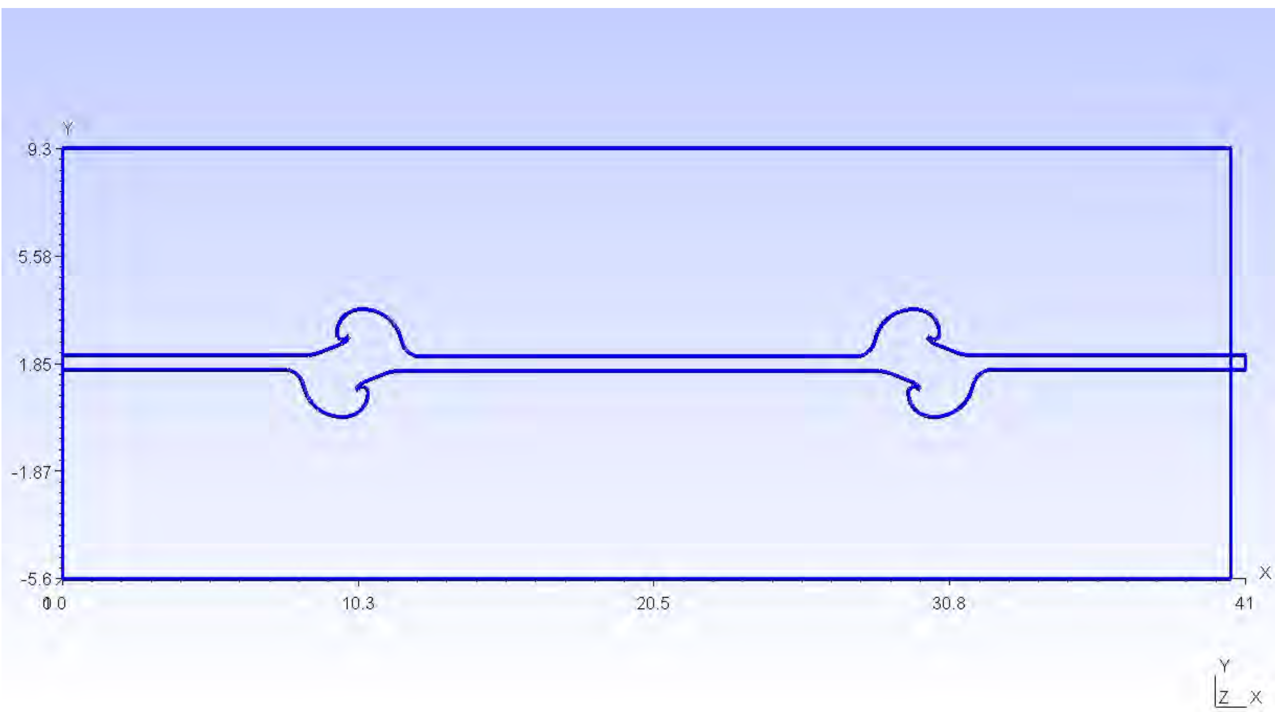


FIGURE 7.5-2. Section View through Conceptual Pullout Test Device with Two Interlock Joints

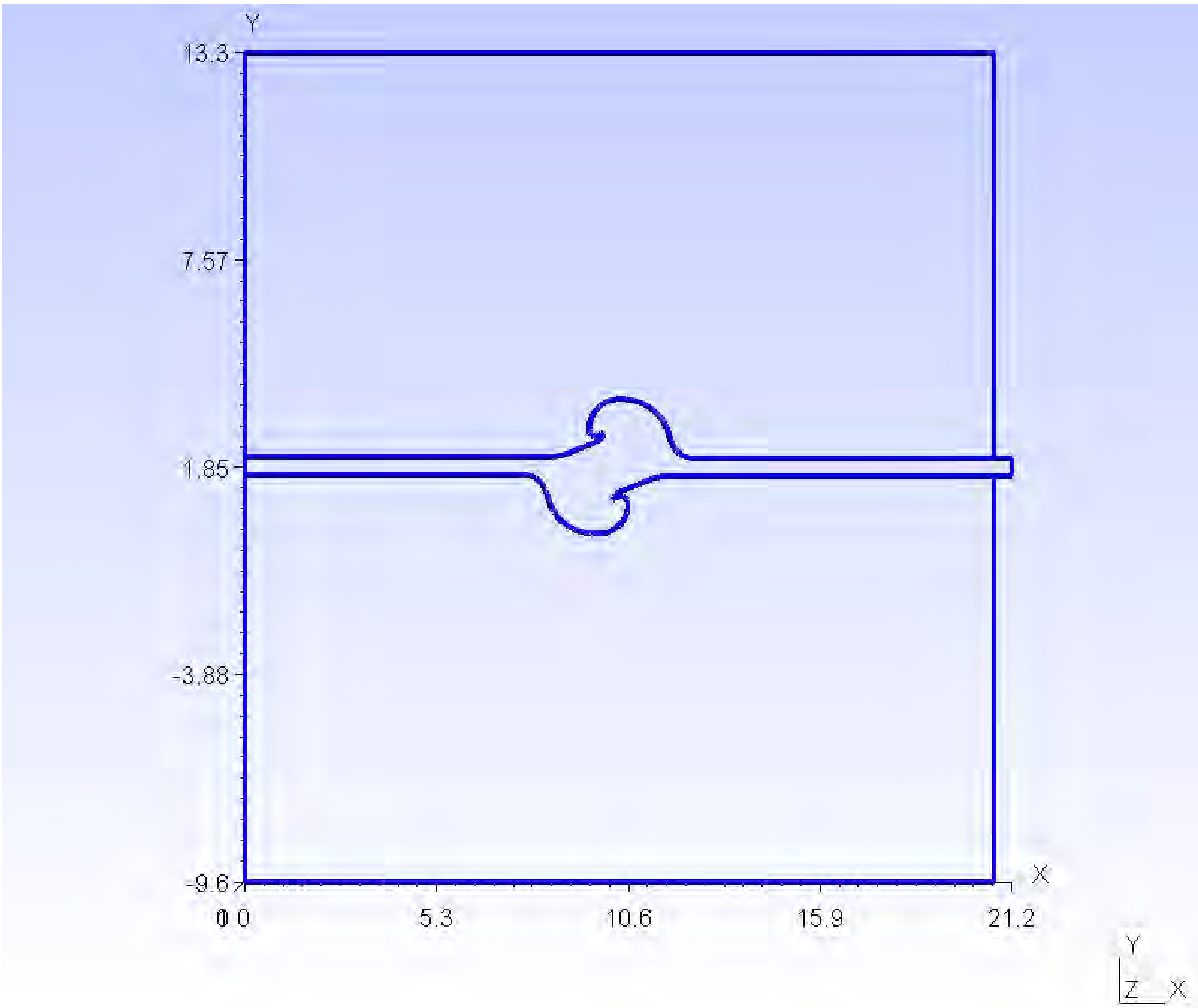


FIGURE 7.5-3. Section View through Pullout Test Model with One Interlock Joint

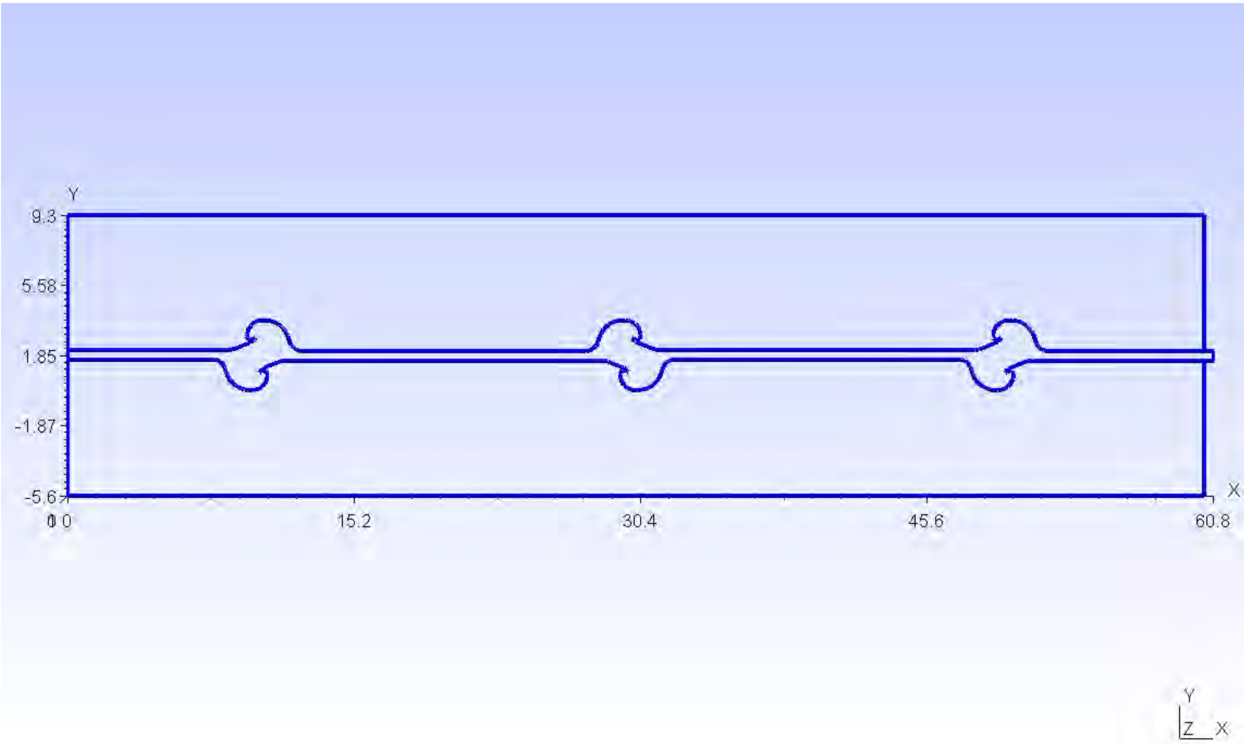


FIGURE 7.5-4. Section View through Pullout Test Model with Three Interlock Joints

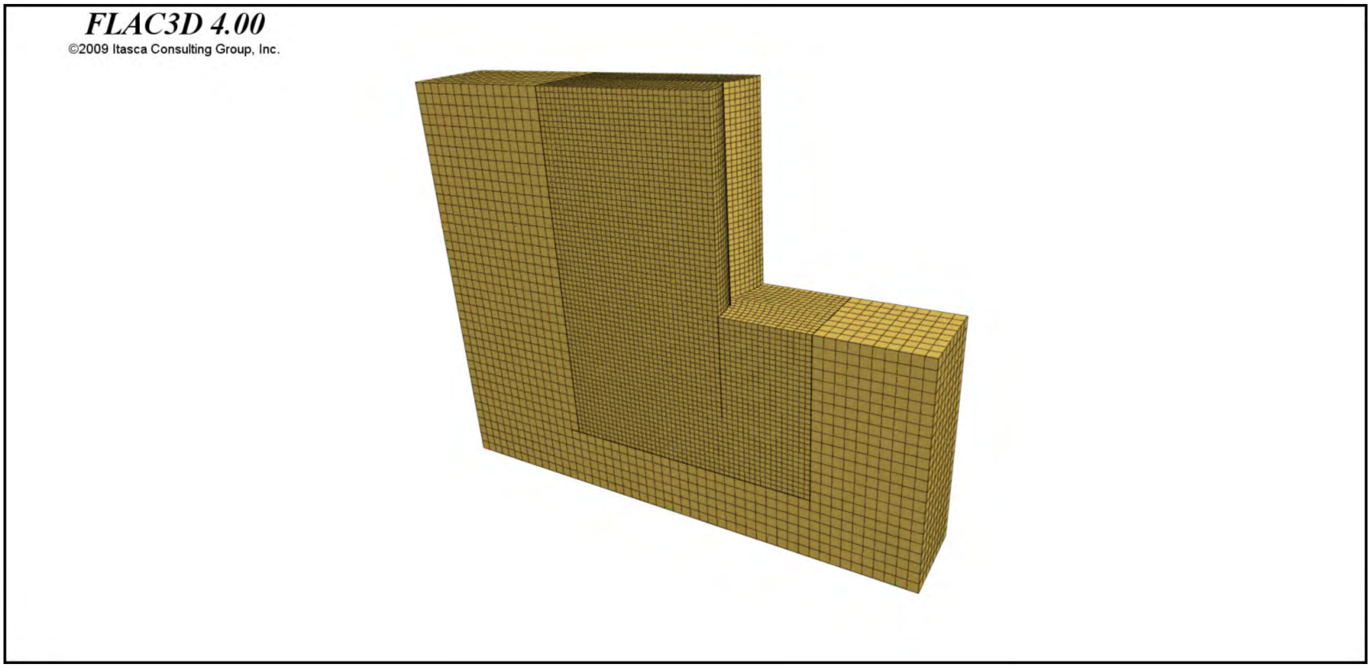


FIGURE 7.5-5. FLAC^{3D} Mesh for 2h (Two Half-Cell) Local Defect Model

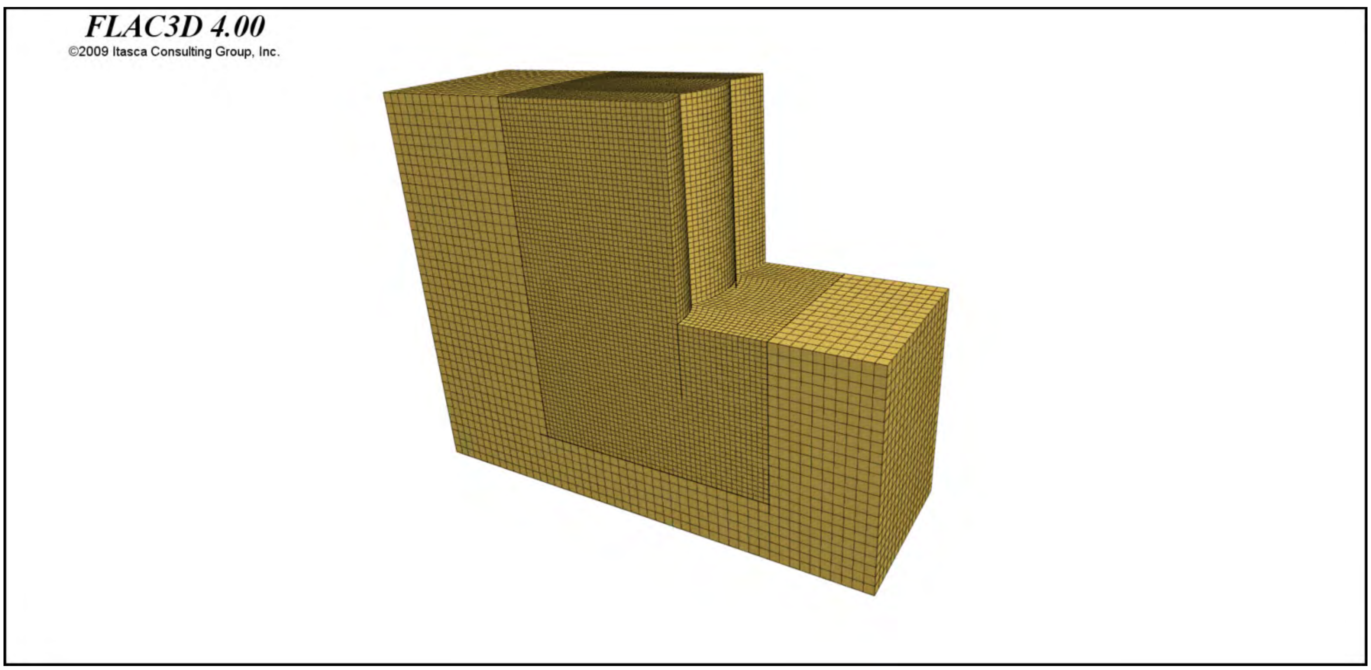
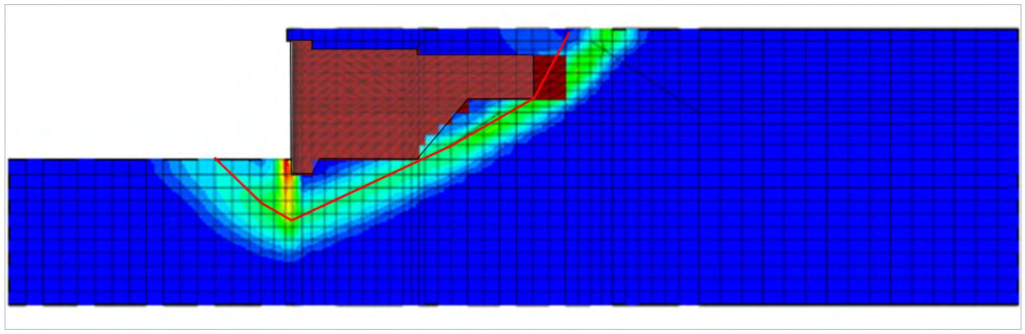


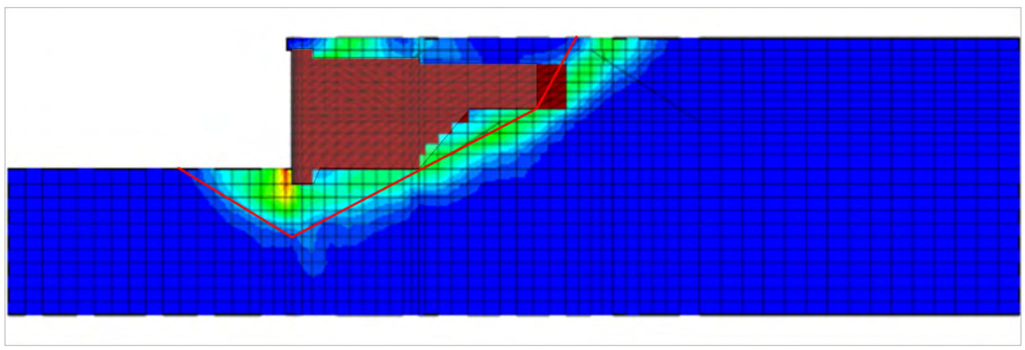
FIGURE 7.5-6. FLAC^{3D} Mesh for 1w+2h (One Whole Cell, Two Half-Cell) Local Defect Model

FLAC^{3D} FOS = 1.13
Slide FOS = 1.14



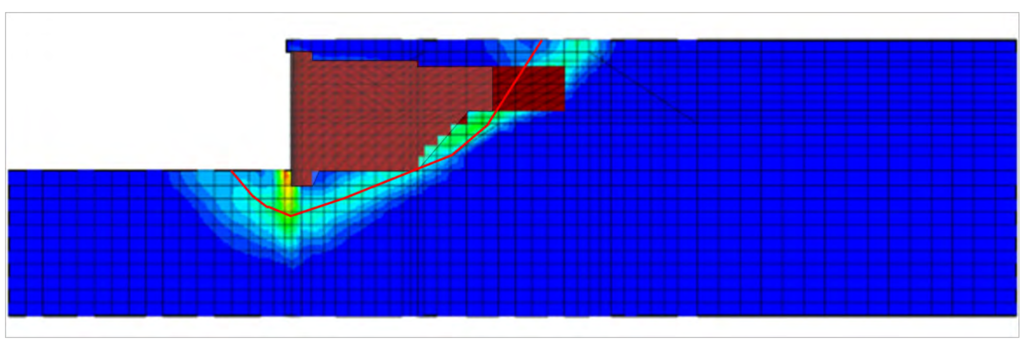
(a) –Short-term Undrained (End of Construction)

FLAC^{3D} FOS = 1.23
Slide FOS = 1.26



(b) - Long-Term Drained

FLAC^{3D} FOS = 1.06
Slide FOS = 1.06



(c) - Long-Term Undrained

FIGURE 7.6-1. FLAC^{3D} Maximum Shear Strain Rate – c-ϕ Reduction Method

SECTION 8

Constructability Assessment

Beginning in 2006 and continuing through 2010, the PIEP awarded four construction projects to create the North Expansion at the POA. As shown in Figure 1.2-3 in Section 1, these projects included the North Backlands in 2006, the Dry Barge Berth in 2007, and the 2008 Marine Terminal Redevelopment (MTR), which included both the Barge Berth Phase 2 and North Extension in 2008-2009. The last contract awarded was the North Extension Bulkhead/Winter Closure in 2010. The projects' essentials and timelines are as follows:

- **North Backlands project, awarded June 2006 and completed December 2006:** consisted of constructing a riprap dike, placing earthen fill material, constructing ditches and erosion and sediment control measures, and seeding (Figure 8-1).
- **Dry Barge Berth project, awarded August 2007 and completed May 2008:** consisted of constructing an earthen dike faced with riprap, placing granular fill material, removing and salvaging riprap from the North Backlands project, and installing piezometers (Figure 8-2).
- **2008 MTR project, awarded June 2008 and ceased August 2010**: consisted of offshore sampling; dredging; construction of the OCSP® structures for the Dry Barge Berth, Wet Barge Berth, and North Extension; mining and placing of granular fill; deep compaction of fill materials; and installation of drainage structures (Figure 8-3).
- **North Extension Winter Closure project, awarded in 2010:** provided forensic analysis and repair to project work already completed by the Wet Barge Berth and North Extension projects and provided a temporary Z-pile wall to seal off areas of the Wet Barge Berth and North Extension that were incomplete. All associated work was performed during the 2010 construction season. No physical work was performed in 2011.

The typical construction season in the Anchorage area, especially the Knik Arm in-water work construction season, occurs from approximately mid-March through November because of the hazardous ice flows, extreme weather, and short daylight conditions in Knik Arm during the winter months of December, January, and February, which prevent construction activities. Some construction operations may be possible during winter months if there is unseasonably warm weather or low-ice conditions (Figure 8-4).

8.1 Introduction

This section documents the examination of the plans, contract documents, and project records of the above described projects. CH2M HILL also interviewed ICRC, QAP, PND, and West. The review of the documents and the interviews were used to complete the following tasks:

- Assess the constructability of the work completed to date as described above, with respect to the phased approach currently shown in Figures 1.2-2, 1.2-3, and 6.1-11.
- Document the risks associated with construction of the as-designed structure given the configuration and environmental conditions.
- Analyze the potential for subcontractor experience-related construction issues relating to the installation of OCSP® systems affecting the project performance outcome, and provide recommendations for the technical criteria appropriate for construction subcontractor selection for future work toward the goal of limiting the potential for subcontractor experience-related construction issues.
- Evaluate construction methods and records, including the construction methods employed on previous phases of the OPEN CELL® bulkhead, and report on constructability issues associated with constructing the bulkhead, highlighting those activities found to be deviations from the intended design or that are found to be problematic for the subcontractor.

- Evaluate the construction equipment used and methods for installation of the sheet piling. Also evaluate the inspection methods, survey control, and QA/QC procedures.
- Recommend appropriate construction methods for future work.

Many of the documents reviewed for the constructability assessment are presented in Appendix H. These are referenced throughout Section 8 and are listed in Table 8.1-1. The reader will gain perspective and greater clarity by reviewing the appendix materials while reading this section.

TABLE 8.1-1
Source Documents for Constructability Assessment *(see Appendix H for copies of these materials)*

Item No.	Stage	Date and Type	Title	Prepared By
H1	Permit	8/10/2007 to 8/15/2011 404 Permits	Conformed 404-10 Permit	USACE
H2	Construction	12/10/2010 Inspection Report	PIEP North Extension/ Wet Barge Berth Sheet Pile Inspections	ICRC
H3	Administration	2/26/2008 to 7/16/2008	Synopsis No. DTMA1R08002 and Contract DTMA1D08012	MARAD
H4	Preconstruction	2/5/2008	Addendum 8, Invitation to Bid #4414-1-S100, 2008 Marine Terminal Development	ICRC
H5	Construction of North Backlands (Permit)	8/24/2005 404 and 403 Permits	POA-2003-502-2 404 and 403 Authorization	USACE
H6	Construction	7/01/2010 As-built plan	North Extension Bulkhead Project Open Cell Layout As-built Plan	PND
H7	Construction	2009 Inspection Reports	Sampling of Cell By Cell Inspection reports	ICRC, PND, DOWL
H8	Construction	10/14/2008	RFI #51 – Driving Conditions	MKB
H9	Construction	6/1/2009	QAP Letter 087 Seaward Wall Movement	QAP
H10	Construction	6/14/2009	RFI #97 – Vibracompaction Refusal	MKB
H11	Construction	6/19/2009	QAP Letter 094 Request for Suspension of Work	QAP
H12	Construction	6/21/2009 Letter	ICRC Letter 047 Non conformance of OCSP® Installation	ICRC
H13	Construction	7/14/2009	QAP Letter 101 Response to ICRC #46	QAP
H14	Construction	8/5/2009	QAP Letter re: RFI 97 and 110	QAP
H15	Construction	6/26/2009	QAP Letter 120, Notice of Intent to Claim	QAP
H16	Construction	12/9/2008 Plan drawing	North Extension Cross Section Conformed Drawings	PND
H17	Construction	1/11/2011 Draft Report	Port of Anchorage Expansion Project Draft Report on Sheet Pile Driving Problems for MKB Constructors	Lachel & Associates
H18	Construction	10/4/2011 Table	Summary Report by ICRC Citing 34% Damaged Piles	ICRC

TABLE 8.1-1
Source Documents for Constructability Assessment *(see Appendix H for copies of these materials)*

Item No.	Stage	Date and Type	Title	Prepared By
H19	Construction	2/21/2008 Table	Side-by-Side Comparison of QAP, AIC, and Kiewit Bids	Not stated
H20	Permit /Construction	3/27/2009 E-mail	No Take Ruling	Wayne Leong, MARAD
H21	Construction	4/9/2009 E-mail	Diana Carlson, ICRC, 19-Point Letter	Diana Carlson, ICRC
H22	Construction	4/11/2011 Final rule	Final Rules IHA April 2009 Federal Regulation, 50 CFR Part 226 [Docket No. 090224232-0457-04] RIN 0648-AX50 Endangered and Threatened Species: Designation of Critical Habitat for Cook Inlet Beluga Whale	National Marine Fisheries Service, National Oceanic and Atmospheric Administration, Commerce
H23	Permit / Construction	1/13/2011 Memorandum	Memorandum from ICRC: In-Water Pile Driving Work Restriction Windows for Construction Activities and Marine Mammal Protection	ICRC
H24	Construction	7/16/2008 Letter	2008 Dredging Overruns PND#061028 Project North Extension/Wet Barge Berth Sheet Pile Inspections	PND
H25	Construction	2/15/2008 Bid addendum	Addendum 9, Invitation to Bid #4414-1-S100, 2008 Marine Terminal Development	ICRC
H26	Construction	2008-2009	Sampling of Coating Inspection Reports	QA Services, IICS
H27	Construction	2009-2011	Corrosion Correspondence	USACE, POA, ICRC, Coffman, PND
H28	Construction	10/17/2011	Corrosion at Pile Splices	ICRC
H29	Construction	7/16/2008, 6/2/2008	1) Dredge quantity letter; 2) Pre and Post dredging surveys	PND, Northwest Hydro
H30	Construction	2008-2009	Requests for Information (RFIs)	MKB
H31	Construction	9/23/2011 Letter	POA Expansion Bulkhead Condition Analysis and Recommendations	PND

Note: Table 8.1-1 lists the documents cited in this section that are included in Appendix H. CH2M HILL also reviewed additional project documents provided by USACE, POA, MARAD, ICRC, QAP, PND, and West.

8.1.1 Limitations

In the course of this constructability review, opinions were expressed by various entities including contractors, designers, and program managers. These opinions were included in the review to provide the reader a better project understanding, but inclusion of these opinions and statements does not imply agreement by CH2MHILL.

8.2 Constructability

8.2.1 Program Delivery Organization

The organizations involved in the project and their relationships are shown in Figure 1.1-1. The ultimate owner of the POA is the MOA. In 2003, Congress required that all federal and non-federal funds provided for the project be transferred to and administered by the Maritime Administrator. Thereafter, MARAD and the MOA entered into a Memorandum of Understanding delineating the responsibilities of the parties.

In 2003, MARAD awarded a contract to Koniag Services, Inc., an Alaska-Native firm, to provide services at the port. That contract was novated in 2004 to another Alaska-Native firm, Integrated Concepts and Research Corporation. In 2008, MARAD stated that they intended to award a contract for continued project management services of the PIEP to ICRC on a sole-source basis. The rationale was that ICRC had been the project manager since 2003. However, ICRC had been purchased by VSE, a large business, and under federal regulations this meant that, absent a waiver of the Small Business Act, ICRC's contract would need to be terminated. A February 2008 Synopsis (Item H3 in Table 8.1-1, Appendix H) goes on to identify in detail the value in keeping ICRC as the project management component. Thus, ICRC was sole-sourced to perform project management services for the construction phase. This contract award placed ICRC in control of phasing and plan development, as well as administration of the construction projects. The contract was effective July 16, 2008, with an initial term through May 31, 2011, with the option to add four 1-year extensions. The scope of work and contract are included in Item H3 in Appendix H.

The following is a list of the organizations and companies, including ICRC, involved in the PIEP:

- **ICRC** provided program management and purchasing activities to design and construct the PIEP. This includes land use planning, design and design management, construction and construction management, environmental management and permitting, project controls, public outreach, stakeholder relations, health and safety compliance, information technology, QA/QC oversight, and commissioning.
- **PND Engineers (PND)** provided design services for the OCSP® system used on this project. PND developed OPEN CELL® technology and holds the patents on the system. They are the primary source of information related to the design of OCSP® systems. They provided construction oversight services on OCSP® construction. During construction, PND performed the typical duties of the Engineer of Record including RFI submittal, shop drawings, vibracompaction plan reviews, etc. In addition, they monitored and documented the installation of the OCSP® each day during the 2008, 2009, and 2010 installations. PND was a subcontractor to ICRC.
- **Terracon** was the geotechnical consultant to the program manager (ICRC). Terracon’s scope of work for the geotechnical exploration phase included designing, performing, and inspecting the offshore geotechnical investigation and testing program. This included geophysical investigations for borehole logging to determine physical parameters of subsurface soils, including cone penetration test (CPT) operations, in situ vane shear, and shear-wave velocity measurement.
- **Coffman Engineers (Coffman)** provided the design of the cathodic protection system.
- **DOWL HKM (DOWL)** provided many services during the design and construction of this project, including QA testing during vibracompaction and daily inspection reports during Quality Asphalt Paving's work, including pile-driving inspection reports.
- **Alaska Interstate Construction, LLC (AIC)** was a subcontractor to ICRC hired to perform the work for the North Backlands and the Dry Barge Berth.
- **Quality Asphalt Paving (QAP)** was the subcontractor to ICRC hired to perform the work for the Barge Berth Phase 2 and North Extension projects, otherwise called the 2008 Marine Terminal Redevelopment Project.
- **MKB Constructors** was a subcontractor to QAP. They performed the pile-driving for the OCSP® on the 2008 Marine Terminal Redevelopment Project.

- **West Construction Company Inc. (West)** was the subcontractor hired by ICRC originally to complete the unfinished sections OCSP® system remaining at the end of 2009. Their work was later changed to forensic work, repair work, and the North Extension Winter Closure of 2010 and 2011. The forensic work was accomplished using West, PND, and Global Offshore Divers.

8.2.2 Construction Phasing

Each of the four construction projects is discussed briefly in order of construction below, followed by a summary of the key construction interrelationships for phasing. The four construction projects are shown on Figure 1.2-3.

8.2.2.1 North Backlands Project (2006-2007, Awarded to AIC)

In August 2005, the USACE issued the 404 and 403 permits for the first phase of the POA North Expansion, the North Backlands (Item H5 in Table 8.1-1 and Appendix H). The North Backlands project, as designed, consisted of constructing an underwater geotextile container dike, placing earthen fill material, ditch construction, erosion and sediment control measures, and seeding. During construction, it was determined the geotextile container dike could not be manufactured in time to complete the project. As an alternative, the subcontractor (AIC) proposed using riprap rock as a substitute for the geotextile container dike. This was accepted and ultimately this phase of the project created 27 acres of new land. The original permit document called for this area to be dredged prior to the placement of fill (Item H5 in Table 8.1-1 and Appendix H). This dredge work did not occur. The lack of dredging in the area of the North Backlands Project did not affect the constructability of this portion of the work and it is unlikely that it affected future phases of the work.

8.2.2.2 Dry Barge Berth Project (Fall 2007, Awarded to AIC)

This phase of project construction originally included placement of earthen fill material similar to the work performed during the North Backlands project and placement of the first sections of OCSP®. Issuance of the USACE discharge permit for this next phase of work was expected to be issued in October 2006. The permit was not issued and therefore project construction was delayed. The permit was rescheduled to be issued by April 2007, but again it was not issued. The USACE discharge permit for this phase was issued on August 10, 2007 (Item H1 in Table 8.1-1 and Appendix H). The permit was not issued due to ongoing environmental discussions related to protecting the Cook Inlet beluga whale. Specifically, the time was used to come to agreement with regulatory agencies regarding mitigation measures and construction requirements regarding in-water pile-driving work. Because of the lack of a permit and unknown restrictions, ICRC rewrote the construction contract, removing the portion that installed the OCSP®. This revised project was advertised and awarded as the Dry Barge Berth project. The Dry Barge Berth project consisted of constructing an earthen dike faced with riprap, placing granular fill material, removing and salvaging riprap from the North Backlands project, and installing piezometers.

The subcontractor (AIC) worked from August 2007 through December 2007 building this phase. Lack of the timely issuance of the 404 and 403 permits, and the desire to move the project forward, were the driving factors that caused the placement of rock revetment in the footprint of OCSP® installed during the 2008 MTR project. In addition, no dredge work was performed prior to placement of the dike fill. The lack of predredging during this phase of the work, specifically along the face of the OCSP® Cells 27- 38, resulted in some of the deepest pile penetrations and most difficult pile-driving on the project, which was subsequently performed by QAP on the next phase of the work. Figure 1.2-3 shows the overlap between the Dry Barge Berth Project and the subsequent phases of work. The intention of the predredging was to remove the soft soil layer for structural stability and to minimize the depth of penetration through the underlying hard clay layer. The PIEP North Extension/Wet Barge Berth Sheet Pile Inspections (Item H2 in Table 8.1-1 and Appendix H) identifies the location where many large rocks were found. The pictures in the report show that most of the rocks had sharp, nonrounded edges indicative of quarried rock such as riprap.

8.2.2.3 2008 Marine Terminal Redevelopment Project (2008 - Fall 2009, Awarded to QAP)

This project, which was awarded to QAP, included the OCSP® elements removed from the Dry Barge Berth contract in combination with the Wet Barge Berth OCSP® and the entire North Extension (see Figure 1.2-3). The combined project was called the 2008 Marine Terminal Redevelopment Project. This project continued to create new property by reclaiming tidal and subtidal areas to provide the basic structure for new waterfront facilities, including the installation of an OCSP® bulkhead. Work associated with this project included:

- Offshore sampling. The subcontractor was to use barge-mounted equipment to drill test holes, conduct SPTs, and recover subsurface samples. Testing was to be conducted for the full length of future construction areas along the OCSP® bulkhead control line.
- Dredging of seafloor materials from within and adjacent to the project footprint and subtrench dredging and backfill along the OCSP® bulkhead footprint (Figure 8-5).
- Construction of an OCSP® bulkhead and tailwalls (Figure 8-6).
- Mining, processing, hauling, and placing granular and common fill materials and construction of rock-protected dikes.
- Salvage and stockpile of rock materials placed during previous contracts.
- Deep compaction of fill materials using vibracompaction methods and layer-compaction of materials placed above elevation +24 feet MLLW.
- Installation of drainage structures.
- Operation, maintenance, and reclamation of borrow sources.
- Operation and maintenance of haul roads.
- Installation, maintenance, and monitoring of pollution prevention best management practices and development and implementation of a stormwater pollution prevention plan and other plans as may be required by current codes and/or regulations.
- Site grading, surveying, and other activities necessary to complete the work as detailed in the construction documents.

The 2008 Marine Terminal Redevelopment Project included plan sheets showing the previously completed work. The construction contract showed the rock revetment which was placed during the Dry Barge Berth construction by AIC. The construction contract identified the removal and salvage of slope fascia rock. Addendum 8 IT #4414-1-S100 Item 14 (Item H4 in Table 8.1-1 and Appendix H) shows the as-built drawing of the slope fascia rock. It did not identify the possibility of rocks deposited by ice plucking,[1] or the potential of encountering previously placed armor rock which had washed out from the face of the Dry Barge Berth dike due to tidal currents and wave action.

In October 2009 and July 2010, dredge work was performed at the face of the newly installed sheet piling along the North Extension OCSP® bulkhead. Within 24 hours of the dredge work that was performed in July 2010, a subsidence hole opened up on the land side of the piling (see photos in Item H2 in Appendix H), indicating that a loss of material had occurred and that portions of the system had failed. Shortly thereafter, additional subsidence holes appeared behind the face of the piling, revealing additional damaged piling or damaged interlocks. The program manager ordered a systematic examination of the face sheets of the OCSP® installed as part of the Wet Barge Berth and North Extension projects to determine the extent of the damage.

8.2.2.4 North Extension Bulkhead (2010, Awarded to West Construction)

This project, awarded to West Construction (West), consisted of installation of a new OCSP® bulkhead, completion of cells partially constructed in 2009, removal and replacement of cells not installed to design specifications, and installation of storm drain outfalls. More specifically, work was located at the North Extension (NE) and Wet Barge Berth (WBB) and included the following (see Figure 8-7 for cell reference numbers):

[1] Plucking is a type of erosion that results from the relative buoyancy of ice, combined with the material being "plucked." This can occur anywhere ice is in contact with hard armor erosion control rocks that have the potential to be moved by flowing water, gravity, or wave action.

- Install 11 new cells (NE 1-8, NE 34-36) and 9 tailwalls. Remove the existing gravel and riprap to facilitate the Work. Upon completion of pile-driving, place and compact engineered fill according to the drawings and specifications.
- Complete the installation of two partially completed cells (NE 10-11). Remove the fill previously placed behind the partially driven cells to the extent necessary to facilitate driving the sheet pile. Sheet pile shall be driven and engineered fill shall be placed and compacted in accordance with the drawings and specifications. Partial pulling or other manipulation of the sheets to achieve final tip elevation shall be considered ancillary to this item.
- Remove and reinstall six cells (NE 9, NE 32, and NE 37-40) and four tailwalls (NE AH-AK); excavate and remove non-conforming sheet pile, appurtenances, and temporary Z-pile adjacent to NE 37 and replace with new or salvaged and undamaged pile materials. Remove fill in the referenced cells, along with fill in adjacent cells, as shown on the project drawings to facilitate removal and replacement of the steel structures. Reconstruction of the cells shall include placement and driving of sheet pile and placement and compaction of granular fill material in accordance with the drawings and specifications.
- Remove and reinstall three cells (WBB 36-38) and two (2) tailwalls; excavate and remove non-conforming sheet pile (see reference document "Cells WBB 36-38 Dive Inspection Report"). Install the 80-foot PS31 face sheet pile in WBB Cells 36-38 to replace the original 70-foot PS31 sheets.
- Install two outfalls (Outfalls A and B) and associated storm drain pipe, and one manhole, and place shot rock and riprap according to the drawings and specifications, including closure Cells WBB 39 and NE 33. Excavate and dredge at the outfalls as necessary.
- Perform vibracompaction for the cells constructed under this project and in other areas as directed by ICRC for up to 3,900 locations.
- Dredge in three locations along the existing face of the bulkhead for dive inspection (NE 41-43, NE 48-50, NE 59-62) as shown in the drawings, and at the outfalls in accordance with the drawings and specifications. Dive inspection shall be performed by others (Figures 8-7, 8-8, 8-9, and 8-10).

The scope of work for the North Extension Bulkhead Project was significantly altered as additional pile inspections were performed and problems were uncovered. The following work was removed or modified in the North Extension Bulkhead contract (see Figure 8-7 for reference to cells):

- Wet Barge Berth Cells 36, 37, and 38 reinstallation removed from contract.
- North Extension Cells 9, 10, and 11 removed from contract. Cell 12 was added and installed.
- North Extension Cells 37, 38, 39, and 40 removed, none reinstalled.
- Wet Barge Berth Cells 36, 37, 38, and 39 and North Extension Cells 1 to 11 removed from contract.
- North Extension Cells 33, 34, 35, 36, 37, 38, and 39 removed from contract.
- Temporary Z-pile walls to be installed at two locations to preserve the project.

8.2.3 Construction Phasing Observations

The original three projects, North Backlands, Dry Barge Berth, and 2008 Marine Terminal Redevelopment, were defined by the issuance of USACE 404 discharge permits. The Dry Barge Berth project was modified during the design phase to remove the previously included OCSP® work and was ultimately issued with riprap rock face. This modification to the project was implemented as a way to make progress on the project while the 404 permit process was underway. The riprap rock fascia placement performed during the Dry Barge Berth project was in the footprint of the future installation of Cells 1-38 of the Wet Barge Berth OCSP® performed as part of the 2008 Marine Terminal Redevelopment Project. By comparison, no riprap was placed under the footprint of the future Cells 1-66 of the North Expansion portion of the work.

The conflicts between the work performed under the Barge Berths Project and the 2008 Marine Terminal Redevelopment Project were identified and described in the 2008 Marine Terminal Redevelopment Project general notes as follows:

- "Rock shall be salvaged from the Dry Barge Berth Rock Dike where there is an estimated 48,000 CY of pit run riprap and armor rock available for recovery. Contractor should recover all available armor rock and at least ¾ of the pit run rock. Contractor shall remove all rock under areas where sheet piles will be driven." (Barge Berths Phase 2, General Notes Page 1 of 5, Materials, B, Rock Material)
- "The Contractor shall submit a report including; material-handling equipment and the Contractors means and methods." Conflicts also identified on the demolition plan sheet. (Barge Berths Phase 2, General Notes Page 3 of 5, Submittals, E, Rock Revetment Removal)

8.2.4 Phasing Recommendations

Projects should not use riprap dikes in the same areas where future pile-driving will occur. There is no issue using riprap on dike faces outside the footprint of OCSP® structures. Where winterization of unprotected dike faces needs to occur in areas of future sheet pile driving, the construction contracts should call for other methods of slope stabilization such as the installation of temporary sheet piling and backfill, or potentially block mats.

In retrospect, lack of permits should not override the need to provide a practical construction sequence.

8.3 Construction Risks and Environmental Conditions

This section of the report documents risks associated with construction of the as-designed structure given the configuration and environmental conditions. All construction projects have risks, and it is typically not feasible or cost-effective to completely eliminate these risks. The responsibility of the design engineer in developing design solutions is to identify the primary project risks and develop design solutions that are reasonably constructable and that also balance the project risks with estimated construction costs.

8.3.1 Risk in Program Delivery

It is important that project risks be adequately communicated to the project owner and that sufficient contingencies are considered that enable the project to be constructed within the overall project budget. The project owner should thoroughly understand and be willing to accept the risk levels that are compatible with the level of project design. It is also important to note a distinction between the project low bid price and the estimated construction cost, with the former typically being the lowest project cost incurred if all goes as planned, and the latter being what the owner should budget to deliver the completed project, assuming some level of cost increase due to changed conditions or unforeseen site issues. The relative magnitude of bid price and ultimate costs should reflect the degree to which project risks have been mitigated in the design.

It is important that project risks be adequately communicated to the contractor/subcontractor through the construction contracts. When substantial risk is placed on the contractor/subcontractor, the plans and specifications should provide for rigorous prequalification of subcontractors to improve the probability that the work is within the contractor's/subcontractor's capacity and experience to complete. This prequalification improves the probability that the low bid represents the bid price of a qualified contractor/subcontractor familiar with similar work.

The 2008 Marine Terminal Redevelopment Project identifies significant project risks within the construction notes on the plan sheets. These risks and their mitigation are assigned to the subcontractor's means and methods to resolve. The plans and specifications do not provide for the rigorous prequalification of the contractor or its subcontractors. The project was awarded on the basis of the low bid. There is limited contract language discussing the need for the pile-driving lead to have direct experience in OCSP® or closed-cell sheet piling. The construction contract language does provide that ICRC could reject personnel. The specification requiring previous pile-driving experience does not extend to the crews and provides little to mitigate the most difficult and risky element of work on the 2008 Marine Terminal Redevelopment Project.

8.3.2 Risks in Plans and Specifications

8.3.2.1 Site Risk

The construction documents identify and define the elements of risk during construction. The general notes of the 2008 Marine Terminal Redevelopment Project contain the following:

> GENERAL NOTE 4.A-1: This project is located in an area of 40-foot tides with periodic fast direction changing currents, high phreatic water table, ice and wave exposure. The Contractor must be aware of these conditions and their effects on earth fills, sheet piles and fill construction. Tides and phreatic water can cause fill instability during construction. The Contractor's work plan shall address these conditions.

This general note provides a description of the extreme environment in which the work will occur. The construction documents state that the tides and phreatic water can cause fill instability during construction. The construction documents place the mitigation of this risk on the subcontractor to develop a work plan which addresses this issue. A review of the subcontractor's submitted work plan shows the plan was developed with the intent of providing a stable working platform for their cranes; however, their initial approved work plan did not address slope stability as it relates to the note above. Once sloughing began, the issue was revisited and a great deal of time and energy was spent trying to stabilize the face of the slope. These issues are discussed in Items H7 through H16 and in the Lachel & Associates report (Item H17) in Appendix H.

The 2007 Dry Barge Berth project prescribed the materials and three typical cross sections of the berms to be built under this contract. The plans described fill materials in the form of granular fill and common fill, and rock materials in the form of filter and armor rock and riprap. Each type of material had assigned specifications regarding gradation, wear, mass, etc. The cross sections show height, width, length, and thickness of each prescribed material.

The 2008 MTR project (Wet Barge Berth and North Extension) demolished the previously placed armor and constructed an OCSP® system on the berm constructed under the 2007 MTR project. The North Extension part of the 2008 MTR project constructed a new berm upon which an OCSP® system was built. The plans prescribed the materials and two typical cross sections for the berms to be constructed under this contract. The plans described fill materials in the form of granular fill and common fill, and vibracompaction of the fill/course fill, and described rock materials in the form of filter, armor rock, and riprap. Each type of material had associated specifications as to gradation, wear, mass, etc. The cross sections show height, width, length, and thickness of each prescribed material.

The Contractor was required to provide submittals for the Dike Construction Plan and Dike Source Material and Placement Plan. The Material and Placement Plan was subject to approval by ICRC. The Contractor was required to provide a Rock Revetment (Temporary Slope Protection) Plan describing material laboratory tests, material-handling equipment, and the Contractor's means and methods including method for removing rock.

The plan information has the caveat in the General Note 4 as noted above. The subcontractor's initial stability plan was based simply on the angle of repose. No special materials or revetment were included in the plan. The subcontractor's initial plan was reviewed and approved by ICRC. As described in the items referenced above, the instability of the dike slope placed lateral forces pushing seaward on the sheet piling. These forces essentially bowed the OSCP outward, binding the interlocks and adding to the difficulty in maintaining alignment and in driving the piles.

It would be typical in construction for the Engineer of Record (PND) to be provided the work plan and details of how the Contractor intended to construct the project and therefore would have been able to assess the constructability of the slopes. Records of this approval would typically be in the files between ICRC and PND, however, CH2M HILL cannot confirm how the approval process was conducted.

8.3.2.2 Work Methods Risk

The 2008 Marine Terminal Redevelopment Project required the subcontractor to build a dike in the water to create a work platform. The construction contract states:

> GENERAL NOTE 4.A-2: This project anticipates the creation of a containment dike by placing fill in the water to create a work platform. Granular fill to be placed within 200 feet of the bulkhead control line at centerline of WYEs from the top of the embankment to the mudline sloping shoreward forming a dike. The Contractor to select their own means and methods. Common fill will be placed on the shore side of the dike. Upon completion of the OPEN CELL® sheet pile bulkhead and filling, the fill will be deep-compacted by Vibracompaction. Granular fill will be placed and layer-compacted above +30 MLLW. Layer compact common fill above +24 MLLW.
>
> GENERAL NOTE 3.A-2: Granular fill shall be clean well-graded with not more than 10% by weight passing the #200 sieve, 20 to 70% by weight passing the No. 4 sieve, not larger than 12 inches, and not larger than 3 inches within 1 foot of finished grade.

Except in design-build type contracts, the difficult aspects of engineering and specifications are usually determined during the design of a project. In this case, it appears the design of the tidally influenced side of the dike was left to the subcontractor's means and methods to provide a stable work platform through which the OCSP® could be driven. The construction contract graphically shows the tidal slope of the dike to be about 1.5H:1V (Item H16 in Table 8.1-1 and Appendix H). This slope proved unstable given the water table elevation and the tidal influence of saturation and draining and the use of vibratory driving equipment.

During the meeting with ICRC on February 8, 2012, Diana Carlson and Brett Flint both stated that work performed on the 2008 Marine Terminal Redevelopment Project was limited to working from the land side to the OCSP® face. Work from the sea side was not possible due to overlapping work schedules between this project and transitional dredging work at the POA. CH2M HILL did not find where the Contract documents expressly state this.

8.3.2.3 Dredge Risk

The subcontractor was instructed to dredge material from the footprint of the OCSP® of the North Extension. This work was to be performed no earlier than 7 days prior to the placement of the dike fill discussed above. The dredge excavation was to be constructed 30 feet deep on a slope of 0.5H:1V.

The specified slope of the dredge cut of 0.5H:1V is a slope normally associated with a cut made in rock or stiff dry clay. The dredging quantity overran significantly. The reasons for this overrun were discussed in a letter sent to ICRC from PND (Item H24 in Table 8.1-1 and Appendix H). PND, responding to a letter from ICRC regarding 2008 dredge overruns, speculated the overruns were in part due to lack of expected inspection/direction of the dredging operation and possible differences in the predredge topography. The baseline topography used was a bathymetric survey done by Terrasond in 2005. In discussions with ICRC on February 8, 2012, Diana Carlson stated that ICRC promoted a much larger footprint of dredging which would have been performed prior to any OCSP® work. Ms. Carlson also stated that dredging work was a difficult issue throughout the projects as to who would perform the work and when the work would be accomplished. She recommended better coordination of dredging and that associated funding should be formalized in a cooperative agreement between the interested parties.

The pre-dredge and post-dredge surveys are included as Item H29 in Appendix H.

8.3.2.4 Materials Risk

The 2008 Marine Terminal Redevelopment Project plans state:

> GENERAL NOTE 4.C – DIKE AND FILL CONSTRUCTION – Paragraph 3: Dike fill material specifications for this project are liberal, providing suitability for the bulkheads' long-term, full "built-out" conditions. Some dike fill materials chosen by the contractor may be prone to dike instability caused by: erosion, tidal current, storms, ice, phreatic water, or loads from equipment or storage and shall be addressed in the contractor's work plan.

The construction contract sheets show the cross section of the dike (Item H16 in Table 8.1-1 and Appendix H). The dike is shown to be made of granular fill material. The general note above identifies the issues related to the construction of the dike/work platform and transfers the risk of dike stability during construction to the subcontractor. As further documented in the cell-by-cell reports (Item H7 in Table 8.1-1 and Appendix H), the subcontractor had noted "deficiencies" during construction regarding "anchors and wyes out of specified location, dike slopes and interlock fit issues at wye welds and splices."

8.3.2.5 Subsurface Risk

The 2008 Marine Terminal Redevelopment Project plans state:

> GENERAL NOTE 4.D – SHEET PILE-DRIVING-Paragraph 2: Contractor shall remove rock or other obstructions under the footprint prior to driving sheets. Contractor shall not place anything in the sheet pile footprint that sheet pile cannot be driven through. Contractor's fill in the sheet pile footprint shall be acceptable to ICRC prior to driving sheet piles.

According to the project record, the subcontractor for the 2008 Marine Terminal Redevelopment Project, QAP, removed the revetment rocks that were on the surface; however, given the large extent of sheet pile tip damage as shown in Chapter 6 and Items H2 and H18 in Table 8.1-1 and Appendix H, it appears that rocks were still present within the sheet pile footprint.

Additionally, the liberal specification of the granular fill as noted in Section 8.3.2.2 above (12" minus) introduces additional sizeable rock that could be difficult to drive the sheet piling through.

8.3.2.6 Pile-Drivability Risk

The 2008 Marine Terminal Redevelopment Project plans state:

> GENERAL NOTE 4.D - Sheet Pile-Driving – Paragraph 3: Sheet Piles Wyes and anchors shall be driven in three-point contact full length to tip elevation with a vibratory hammer and/or impact hammer with a suitable driving head, at locations shown on the plans, by methods that will achieve penetration without damage. Methods such as pre boring at interlock locations or trenching and back filling may be required if driving becomes difficult. Piles shall be driven such that the tip of no pile advances more than 5 feet beyond the tip of the adjoining piles, except in instances of difficult driving where the contractor shall be required to reduce this lead distance to 2 feet. Soil conditions at the site are hard and dense. Driving is expected to be difficult.

This construction contract general note is modified by Addendum #9 (Item H25 in Table 8.1-1 and Appendix H), which is the prescriptive requirement that the vibratory hammer be employed first until refusal, prior to using the impact hammer. This construction contract general note describes anticipated difficult driving conditions and that alternative methods may be needed to successfully install the piles. The subcontractor building the 2008 Marine Terminal Redevelopment Project, QAP and its second tier subcontractor MKB Constructors, had successfully driven the piles located in the Dry Barge Berth area during the 2008 construction season. The piling in the Dry Barge Berth area is approximately half the length of the piling driven in the Wet Barge Berth and North Extension areas. Once refusal was obtained using the vibratory hammer, MKB employed an impact hammer to complete the installation. No other installation methods beyond the impact hammer were required for the Dry Barge Berth.

In the Wet Barge Berth and North Extension areas, MKB had difficulty installing the deeper/longer piles. MKB used an H pile probe to try and locate rocks alongside OCSP® installations. This was deemed ineffective. MKB made repeated attempts at using jetting to install piles. The jetting efforts were not successful. Some piles were not installed to the proper depths, some were out of three-point interlock, and some were damaged during installation. The problems encountered during pile-driving are well documented in the project correspondence file. Several pictures of the damaged piles are shown in Section 6. Issues related to slope movement, hard driving conditions, and prescribed pile-driving order were all discussed as possible contributing factors to the difficult driving conditions. The issues are summarized in QAP's letter #120 Intent to Claim dated June 26, 2009 (Item H15 in Table 8.1-1 and Appendix H). The inclusion of the intent to claim letter in Appendix H of this report is not an endorsement of the claim by CH2M HILL and is included only to frame the issues surrounding the construction issues of the project.

In summary, MKB was able to install the piles in the Dry Barge Berth portion of the project Cells 1 through 24. These piles were 45 feet long or roughly half the length of those installed in the North Extension.

MKB had difficulty installing the piles in the North Extension, including Cells 27 through 38 of the Wet Barge Berth. MKB also had difficulty with Cells 12-32 and 40-66 of the North Extension. These cells had the benefit of predredge work, and no previous placement of dike fill with revetment. Nevertheless, MKB still had difficulty maintaining alignment and driving these piles. Pile-driving issues are described in Items H7 through H17 in Table 8.1-1 and Appendix H.

In October 2009, dredge work was performed in front of Cells 27-38 and dive inspections were performed after the dredging was completed. The dive reports confirmed that many of the previously installed piles had damage to both the bottoms of the piles and the interlocks. On July 28, 2010, QAP was notified by ICRC that obstructions and anomalies were uncovered at Cell 60 and Cell 49 during dredging operations on July 21, 2010. In addition, they were notified that a sinkhole developed on the land side of Cell 60 within 24 hours of dredging activity. This started the systematic process of determining the extent and cause of the damaged sheet piling. Ultimately it was determined that a high percentage of Wet Barge Berth piles 27-38 and North Extension piles 12-32 and 40-66 were installed out of interlock.

According to a summary report written by ICRC (Item H18 in Table 8.1-1 and Appendix H), 627 of 1,858 sheet piles pulled for inspection (34 percent) had damage ranging from out of interlock to buckling and twisted piles.

The 2007 Marine Terminal Development Invitation to Bid (ITB), 2008-2009 Marine Terminal Development ITB, and the 2010-2011 Marine Terminal Development RFP were all reviewed for refusal criteria. None of the plan sets or specifications discuss or set a refusal criterion. The only refusal criterion discovered was that associated with the prescriptive use of the vibratory hammers prior to engaging the impact hammer (Item H25 in Table 8.1-1 and Appendix H). This was in deference to the environmental restrictions regarding marine mammals. The construction record indicates that efforts were made by the Contractor and expected by the designers to advance the piles to the design tip elevation. In a few instances, the Contractor wrote RFIs requesting relief from achieving pile tip elevation, and relief was granted in some instances.

In the case of sheet piling where axial capacity is not required, it is typical to specify the tip elevations as the only criteria for approving the piling. However, sheet piles on this project were routinely driven using blow counts as high as 300 blows per foot to achieve the final tip elevations, some with even higher blow counts up to 1,200 blows per foot near the wyes. When driving is this difficult, it is not unexpected that damaged sheet piling would occur. Another indicator of the difficult driving and potential pile damage was the amount of fresh heading that occurred (fresh heading refers to the practice if cutting away bent metal at the top of the piling and is indicative of hard driving). ICRC's response to RFI#88 (Item H30 in Appendix H) of the 2008-2009 Marine Terminal Development ITB states “Reaching planned tip elevation on Barge Berth cells 36 through 38 is essential to the future function of the Wet Barge Berth. Please proceed with whatever means and methods you see fit to achieve plan tip elevation without damaging the sheet piles.”

Blow counts of 300 blows per foot (25 blows/inch) observed on the project are well beyond practical refusal even for circular steel piling, which are considerably stronger than flat sheets. Typically, an example limiting driving criterion is 10 blows per inch for a hammer that is of sufficient size to meet sheet pile tip elevation requirements. A suggested approach on future projects would be to include a bid item for pile-driving by alternative methods such as spudding, drilling, jetting, etc. This approach shares the risk and allows the engineer more flexibility and control to ensure that the piles are installed correctly with interlocks intact.

8.3.2.7 Other Pile Installation Risks

The 2008 Marine Terminal Redevelopment Project plans called out the following prescriptive pile-driving requirement:

> GENERAL NOTE 4.D - Sheet Pile-Driving – Paragraph 3: …Piles shall be driven such that the tip of no pile advances more than five feet beyond the tip of adjoining piles except in instances of difficult driving where the Contractor shall be required to reduce this lead distance to 2 feet. Soil conditions at the site are hard and dense. Driving is expected to be difficult….

General compliance with this specification was achieved. However, the photo log shows some areas where MKB significantly deviated from this requirement. During a meeting with PND, Garth Howlett confirmed the general compliance achieved by MKB. He stated that adherence to this specification is important as it provides the lateral stability to guide proper alignment of the piles. Mr. Howlett also said that the method by which the piles are installed requires finesse. The piles must be worked into the ground. Often they must be backed out a bit and reworked to maintain pile integrity and interlock. As part of this constructability review, CH2M HILL viewed many hundreds of photos and videos of the installation of the OCSP® system. With some noted exceptions and within the limitations of an after-the-fact photo review, CH2M HILL agrees that there was general compliance with the 5 foot leading edge rule.

The 2008 Marine Terminal Redevelopment Project plans also discussed the need for a detailed pile-driving plan. This language refers to spudding, augering, chisels, threaders, and jetting:

> GENERAL NOTE 4.H - Pile-Driving – Paragraph 8: Pile driving plan shall specify and discuss all limitations of all equipment to be used including: Pile hammers, extractors, spudding, auguring, chisels, threaders, jetting equipment, etc. Information shall include manufacturer's name, model numbers, capacity, rated energy, hammer details, cushion materials, helmet, and templates, etc.

The 2008 Marine Terminal Redevelopment Project plans discussed the need for a template as noted below:

> GENERAL NOTE 4.H - Pile-Driving – Paragraph 9: Driving templates shall provide for adequate stability of sheets during placement, driving and until adequate soil is placed for cell stability. Template design shall incorporate a system of structural framing sufficiently rigid to resist lateral forces such as water current, driving forces and support of sheet pile until driving tip elevations of the entire cell is stabilized. Provide outer template straps or other restraints as necessary to prevent the sheets from buckling, warping or wandering from alignment.

During the 2008 and 2009 construction season, MKB used vibratory hammers to refusal, then impact hammers to tip elevation or refusal. MKB used a standard simple system of round piles and two curved metal templates with marks to guide the sheet pile alignment. MKB did not employ extensive use of spudding, augering, jetting, or any other alternative method of pile installation. As stated earlier, MKB did attempt jetting, but it was found to be ineffective as applied.

8.3.2.8 Pile Inspection Risk

The subcontractor was required to provide weekly pile-driving reports. The report was to include pile penetration rate data, pile top and bottom elevations, modifications to piles, and damage or repairs made to piles. A final pile-driving report documenting all variations from project requirements was also required. The subcontractor performed the work to document the pile-driving. This was supplemented by records provided by PND (samples included in Item H7 in Appendix H). There were times when pile-driving records were not provided in a timely manner due to a breakdown in working relationship.

According to the construction contract, the subcontractor was required to inspect the interlock joints of driven piles. This task was discussed during the meeting with PND on February 8, 2012. PND was asked what the standard practice was for a contractor to inspect the interlock joints, or to ensure interlock had been achieved. Charles Kenley, P.E., stated that prior to this issue there was no standard protocol for systematic verification. During a meeting with West, the same response was given; no standard protocol for systematic verification of pile interlock has been used in the past. CH2M HILL did not identify an industry standard protocol for the systematic verification of interlock. In the fourth contract, while installing new piles or reinstalling piles, the protocol of pulling of every tenth pile was established (or as often as hard driving conditions warranted) for examination of pile integrity and interlock, then reinstalling it or repairing it as needed. This resulted in the systematic inspection of the work. Using this method, the contractor identified some damaged piles that were repaired and reinstalled. The damage to piles driven by West was attributed to subsurface rocks or hard driving conditions. These conditions were overcome through the use of spudding and excavation. The verification of every tenth pile is an indicator that installation procedures, equipment, and personnel are achieving piles that are being installed per plan and specification. It is, however, only an indication. This check alone does not provide positive proof that all piles have been installed straight and true, maintaining interlock. Pulling every piling and reinstalling would give the needed assurance, although with considerable cost and schedule impacts.

According to the records in previous contracts, a few piles were pulled and reinstalled by QAP; however this task was not completed by QAP in a systematic manner. The 2008 Marine Terminal Redevelopment Project provided no specificity in this inspection specification as to how the interlock joints were to be inspected. In addition, there was no pay item for the verification of the interlocks.

ICRC, PND, and DOWL inspectors were all onsite performing routine inspection (examples are included in Table 8.1-1, Item H7) . Daily logs were maintained by all parties. The individual daily reports were detailed in their review and discussion of ongoing issues. The overall project documentation was comprehensive. Materials were verified. Compaction tests were taken and verified. Splicing work was performed by certified welders. Coating repairs were reviewed and verified. Daily photos were taken of ongoing work. There were no apparent errors or omissions found in the daily documentation of the project work by inspectors.

8.3.2.9 Sheet Pile Alignment Risk

The subcontractor was to monitor the location of all wye joints on a weekly basis using x, y, z coordinates. The construction contract calls for the wyes to be driven not more than 2 inches from plan location. Once set, the wyes were to be braced to maintain alignment during fill operations. Beginning with Cell 25 of the Wet Barge Berth, the subcontractor had difficulty in maintaining the tolerances called out in the construction contract regarding the location of the wyes. The pile-driving records consistently note that this specification was not met (samples included in Item H7 in Appendix H). The subcontractor asserted that soil movement impacting the face sheets of the OCSP® was the main issue affecting their ability to maintain the tolerances called out in the construction contract. The subcontractor struggled to maintain correct installation tolerance of the wyes. As a potential solution, PND recommended the installation of tailwalls to be used as anchors prior to the installation of the face sheets. However, this eliminated working to the free end of the tailwall piling and was deemed unsuccessful by PND.

The use of a template (Figure 8-11) was required to achieve the following vertical and horizontal tolerances for the face sheets:

- Three inches from plan location at cutoff elevation and no more than ¼ inch per foot length out of plumb (max is 90 feet × ¼ inch = 22.5 inches).
- Tailwall driven within 2 feet of plan location in straight or curved line as shown. No more than ½ inch per foot length out of plumb (max is 80 feet × ½ inch = 40 inches).
- Tip elevations were to be achieved +/- 1 foot.
- Cutoff elevations were to be left 1 foot high.

During a meeting with PND on February 8, 2012, Garth Howlett discussed the difficulty the subcontractor had maintaining plumb tolerances both on the face and on the side of the piles. Maintaining this tolerance required the removal and reinstallation of numerous piles. Additionally, the NEPA requirements for work windows and whale sightings required stopping and restarting the installation of some piles. The subcontractor stated this stopping and restarting contributed to the leaning of the piles. The Contractor's difficulty in maintaining the plumb tolerance is well documented in the contract pile-driving records. After partial installation, many piles had to be extracted and redriven. The contract record also documents the frequent starting and stopping of work due to the whale sightings. The impact of the slope movement on individual piles driven on the tidal side of the berm is a function of time. CH2M HILL agrees with the comment that stopping and restarting contributed to the leaning of the piles.

The construction contract called for fill to be placed in even lifts across the entire cell area. Fill was to be placed around tailwalls and anchor walls first, then against face sheets. The elevation of fill between adjacent sheets could not differ by more than 3 feet. The construction contract stated that uneven filling of cells or failure to maintain plan distance between the wyes will result in the wall face distorting seaward during filling and compaction or tailwall distortion. The subcontractor was in general compliance with this specification.

8.3.2.10 Tidal Variation Risk

High tidal variations were called out as a force that would affect sheet pile construction. These forces were to be minimized and accommodated by cutting 12-inch-diameter holes in the tailwall section. The subcontractor complied with this requirement.

8.3.2.11 Sheet Pile Coating Risk

There were three main issues concerning the galvanizing of the sheet piling:

1. Thickness of the initial galvanizing specification was reduced to increase availability of the sheets from more suppliers and to decrease the galvanization build-up in the interlocks. As described in a letter from ICRC on 3/15/2011 (Item H27 in Table 8.1-1 and Appendix H), the mil thickness range of 6-12 mils was established. This range was originally submitted by PND by letter on July 29, 2010 (Item H27 in Table 8.1-1 and Appendix H) and concurred to by Coffman on March 20, 2011 (Item H27 in Table 8.1-1 and Appendix H).
2. Protection and repair of the pile coatings was the responsibility of the subcontractor. The project record documents that many pile coating repairs were made in 2008. These observations came from independent review by coating specialists. A sampling of the inspections and repairs are included in Item H26 in Appendix H.
3. Some field splicing of the sheet piling was required because galvanized sheet piling was not available in full 90-foot lengths. A splice detail was provided in the North Extension plans calling for splices to be made above elevation +15 feet MLLW. The field splicing created blistering and delaminating of the galvanizing. A memorandum written by ICRC and backed up with a letter from PND (Item H28 in Table 8.1-1 and Appendix H) documents these coating failures on installed face sheets that had been sand-blasted, welded, and re-coated.

8.3.2.12 Environmental Risk to Schedule

The 2008 Marine Terminal Redevelopment Project experienced a severe impact in October 2008 when the Cook Inlet beluga whale was listed as an endangered species. The listing of the Cook Inlet beluga whale placed a stop work order on the 2008 Marine Terminal Redevelopment Project. The work completed at the time of shutdown included the completion of all of the piles in the Dry Barge Berth Cells 1-24, the transition piles in Cells 25-26, and the piles in Cells 27-38. Although not reflected in the subcontractor's schedule, the anticipated and subsequent listing of the Cook Inlet beluga whale as an endangered species impacted the subcontractor's ability to complete the work in a timely manner.

The subcontractor was required to provide a bid based on Addendum 9 (Item H25 in Table 8.1-1 and Appendix H) issued during the bid period. Addendum 9 identified the final bid requirements:

- No harassment of beluga whales, killer whales, harbor seals, harbor porpoise, or other marine mammals.
- During in-water activities, subcontractor to provide a dedicated marine mammal observer.

- Subcontractor to enforce safety radius of 656 feet for in-water pile-driving work for one mammal present.
- Subcontractor to enforce safety radius of 1,148 feet for impact pile-driving if five or more mammals present.
- Subcontractor to enforce safety radius of 2,624 feet for vibratory pile-driving if five or more mammals present.
- Subcontractor to monitor total impact to beluga whales; once 26 whales are observed within safety radius, radius limits shall be reset; 656-foot radius eliminated.
- ICRC may conduct additional underwater noise monitoring.
- Safety radius may be modified based on additional underwater noise monitoring.
- Subcontractor to enforce safety radius of 165 feet for in-water fill placement.
- Visibility shutdown requirement added during fog or poor light.
- Subcontractor to develop stop and soft start procedures.
- Mammal briefing required for all workers.

The contract was modified by Change Order #4, Revised Marine Mammal Requirements, on July 15, 2008. This change order modified the marine mammal requirements by adding the following:

- Defined and limited the "taking" of 34 beluga whales, 20 harbor porpoises, 20 killer whales, and 20 harbor seals. A mammal is considered "taken" if the mammal enters into the National Marine Fisheries Service (NMFS) determined harassment isopleths while vibratory pile-driving is ongoing. Thus the larger the radius of the harassment isopleths, the more likely a taking will occur.
- Stated that if a listed mammal is taken by injury or death, it may result in suspension of authorization.
- Added reporting requirement of any animals taken under this authorization.
- Required notification of NMFS 2 weeks prior to in-water pile-driving.
- Stated that work must occur during low use period of Cook Inlet beluga whales around the POA.
- Stated that in-water impact driving shall not occur 2 hours either side of low tide.
- Required shutdown for any whale calf sighted within safety zone.
- Added if take is reached or exceeded, then any mammal entering safety zone requires shutdown.
- Added language regarding mammals approaching non-pile-driving equipment. Shut down at 165 feet.
- Further defined soft start requirements: 15-second operation, one-minute delay, repeat a total of three times.
- Added requirement if mammals are approaching the safety zone, then delay start.
- Further defined restricted visibility due to fog, rough sea.
- Increased monitoring to 30 minutes before operations begin.
- Stated that one to two mammal observers will be posted.
- Required Marine Mammal form to be filled out with requested data and submitted.
- Required submittal of Monthly Marine Mammal Monitoring Report including a report of all pile-driving activities broken down by type of driving equipment used and number of hours. Required a final report summarizing all data. The final report shall estimate the frequency that marine mammals were present, characterize habitat use and behavior of marine mammals at and around the port, characterize sound levels around the port related to and in the absence of construction, and address and analyze impacts of construction.

In a memorandum written by ICRC on January 13, 2011 (see Item H23 in Appendix H), it was determined that the NEPA requirements related to marine mammal restrictions effectively reduced the potential available time for in-water impact pile-driving by 56 percent, and by 34 percent for vibratory pile-driving. This calculation does not include other lost time potentials due to fog, elevated sea state, passing waterborne vessels, sea glare, and the presence of marine mammals in the work zone, nor does it calculate the increased loss of time due to the increased safety radius. There is no question the listing of the Cook Inlet beluga whale and associated permit conditions have made the PIEP extremely difficult to construct.

8.4 Subcontractor Selection Criteria

The PIEP is a challenging project constructed in an extreme environment. The following paragraphs list CH2M HILL's suggested subcontractor experience-related selection criteria for the installation of OCSP® systems.

The subcontractor must have demonstrated experience successfully installing OCSP® in a marine environment to specific tight tolerances. When experienced subcontractors work on projects similar to projects they have previously built, they bring this knowledge base to the new project. This experience allows contractors to see the potential for problems and take steps to avoid or mitigate the problems before they arise. When problems are encountered, they can go to their knowledge base to offer up possible solutions. These solutions start in the simple, less expensive realm and progress toward the complex and costly. The more experience a subcontractor has, the more solutions they can offer when issues arise. When a using a subcontractor with little to no experience, there is no past experience to draw upon when problems arise. The relationship of project managers, design engineers, and subcontractors is important when seeking solutions to problems. Each brings something to the table the others do not have. On a large complex project such as the PIEP, this ability to draw on past experience and knowledge is vital to the success of the project. OCSP® work is particularly complicated. The installation of piles and the subsequent backfilling of cells need finesse, skill, and knowledge to complete successfully.

The subcontractor must have demonstrated experience successfully handling and installing long sheet piles 70 to 90 feet in length. The successful handling of long piles is another difficult aspect of the PIEP. Mishandling of piles contributes to the difficulty in installing the piles. Long sheet piles are significantly more difficult to drive than short sheet piles. Often a long, flat, rigid bed called a tipping table is used to support the piles as they are brought from a horizontal to a vertical position, thus preventing damage during positioning and installation. Damage can occur at any point between manufacturing and installation. Longer sheet piles are significantly more flexible and harder to align and control during driving. An experienced subcontractor will take the necessary precautions to help improve the probability of a successful project.

The subcontractor should have previous experience working in an area of large tidal fluctuation. The large tidal forces on this project were called out in the construction contract. However, the method to deal with the tidal forces was largely left up to the subcontractor. In this case, as was previously discussed, the tides combined with the steep slope of the dike to create large outward forces which significantly affected the subcontractor's ability to drive the piles.

The subcontractor should have demonstrated experience using vibracompaction to compact deep fill sections. One of the major components of work on this project was the vibracompaction of the deep soils. The successful compaction and settlement of these soils is vital to the ultimate success of the project.

The subcontractor must have demonstrated experience in the inspection and reporting of in situ piles and interlock integrity. The success of an OCSP® system relies on pile and interlock integrity. The subcontractor should have demonstrated experience in providing this inspection. On the fourth contract, the method of pulling every tenth pile was developed for quality assurance. This inspection assures that 10 percent of the piles have a solid interlock. In the future, this inspection frequency should be increased to every pile.

8.5 Recommendations for Future Selections

As previously stated, the PIEP has awarded four projects to date. The first three projects (North Backlands, Dry Barge Berth, and 2008 Marine Terminal Redevelopment Project) used the low bid as the main criterion for subcontractor selection. Going forward, a Request for Proposals (RFP) process would allow more weight be given to consideration of factors other than low bid price in the selection of a subcontractor. Other selection factors are determined by the contracting agency and may include work approach, previous work experience on similar projects, personnel, equipment capabilities, financial position, and litigation history. These requirements can be very specific in nature and provide a level of assurance that the subcontractor selected has successfully performed similar work. The fourth project awarded used the RFP process as the method of selection. It is recommended that all future PIEP contracts use the RFP process, or a similar process that ensures subcontractor qualifications.

8.6 Recommendations for Future Work

Many expensive lessons were learned during the construction of the 2008 Marine Terminal Redevelopment Project. The fourth project, called the North Extension Bulkhead/Winter Closure, addressed many of the problems inherent to the design and construction criteria of the 2008 Marine Terminal Redevelopment Project. While the North Extension Bulkhead/Winter Closure project included much remedial work, it also included new construction not completed during the 2008 Marine Terminal Redevelopment Project.

The following is a list of recommendations gleaned from the body of this review:

- The dike shown in the 2008 Marine Terminal Redevelopment Project construction contract proved to be unstable despite the subcontractor's efforts to stabilize it. This resulted in significant effort and expense to install the piles through the sloughing slope. Temporary crane support work platforms should be used in a land-based effort to install OCSP® in order to prohibit differential horizontal pressure on the face piles. Alternatively, sheets could be placed from a floating derrick or jack-up barge. Dredging should be performed in all locations of suspected difficult driving to remove hard clay, large rocks such as rip rap, and other obstructions. The design consultant should be consulted regarding the minimum embedment depth required to install the OCSP® system. Contract dredging work must be coordinated with USACE transitional dredging. Stand-off or set-back limits should be established until the structure is complete.
- The work performed and issues encountered on previous projects need to be fully disclosed and accounted for on subsequent projects. An example is the lack of disclosure of the riprap blowouts encountered during the construction and maintenance of the Dry Barge Berth.
- The contract should provide a bid item for the subcontractor to excavate material in cases of hard driving.
- A suggested approach on future projects would be to include a bid item for pile-driving by alternative methods such as spudding, drilling, jetting, etc. This approach shares the risk and allows the engineer more flexibility and control to ensure that the piles are installed correctly with interlocks intact.
- Future contracts would benefit by prequalifying subcontractors against the major work components of the project. The prequalification criteria may change as the design adjusts to the varied conditions encountered in Knik Arm, including a 40-foot tidal range, 2- to 3-knot currents, and ice conditions. Specifically, prequalifications should address the following on an OCSP® system contract:
  - The subcontractor must have demonstrated experience in successfully installing OCSP® systems in a severe marine environment to specific tight tolerances. This work experience should include placement of fill material in the OCSP® system.
  - The subcontractor must have demonstrated experience successfully handling and installing long sheet piling 70 to 90 feet in length.
  - The subcontractor should have previous experience working in an area of large tidal fluctuation, high currents, and ice.

  - The subcontractor should have demonstrated experience using vibracompaction to compact deep fill sections.
  - The subcontractor should have demonstrated experience in the inspection and reporting of in situ piles and interlock integrity.

- Since the OCSP® design is a patented system, PND should play a focal role in the installation process. At a minimum, PND should be consulted and be the final decision-maker in all issues related to the installation of the OCSP® system. In addition, the EOR should be represented by a full-time inspector to observe the installation process on a daily basis. The installation of sheet piles is the critical path of the project.
- USACE and the project management team should sign an agreement regarding dredging prior to the contract. Work should be allowed to occur on either side of the OCSP® system work line with time restrictions for each party to the agreement. This agreement should include a realistic agreement as to the project boundary. The current boundary at the face of the wall work line was insufficient.
- The subcontractor should be provided a bid item for work related to the inspection of the OCSP® system. Minimum standards for inspection of the OCSP® system should be detailed in the plans and specifications.
- The project management consultant should be well versed in the construction management of heavy civil projects involving OCSP® in a severe marine environment. The inspectors working for the project management team should have prior experience with the installation of OCSP® systems. To the extent possible, limit the turnover of inspection staff.
- The project management team should incorporate a proactive, inclusive, team problem-solving approach to resolving issues. This should include the use of design engineers. This project is a good candidate for the use of a disputes review board. Future work should prohibit the installation of riprap rock in locations where the OCSP® systems will be installed later. The use of temporary sheet piles or some other system should be used.
- Sufficient testing of the soil conditions should be performed prior to designing a project. Testing data were very limited in the northern section of the Wet Barge Berth and Dry Barge Berth. This contributed to insufficient data to order dredging of the area, the lack of which caused extremely difficult pile-driving.
- Major permits should be in place and not subject to change prior to executing contracts.

8.7 Conclusions

The constructability review revealed many factors that contributed to the ultimate unsuccessful completion of the Wet Barge Berth, and the North Extension. The Dry Barge Berth was completed successfully and is fully functional. The key issues are summarized as follows and are not listed in order of importance:

- The selection of the most qualified contractor to perform this type of work is best achieved through the use of a qualifications-based contract rather than a low bid contract. Low bid contracts were used to secure contractors for both the 2007 Dry Barge Berth and 2008-2009 Marine Terminal Redevelopment contracts.
- Some issues encountered during preceding phases were not adequately communicated through the plans and specifications for subsequent projects. This omission of information contributed to the unsuccessful completion of this project.
- Dredging work must be performed based on a current survey to avoid costly overruns in quantity.
- Dredging should be performed in all areas where difficult driving is anticipated and must be coordinated with USACE transitional dredging work so that clear work areas are defined.
- Lack of permits should not override the need to provide a practical construction sequence. The redesign and phasing of the 2007 Dry Barge Berth and the 2008 MTR projects resulted in large rock such as rip rap in the future footprint of the OCSP® system.

- Projects should not use riprap dikes and large fill materials (such as 12” minus) in the same areas where future pile-driving will occur.
- The risk of providing a stable dike face was assigned to the Contractor. A plan to accomplish this was submitted to and approved by ICRC. The failure of this plan to provide a stable slope significantly contributed to the inability of the Contractor to properly place the sheet piles.
- Except in design-build type contracts, the difficult aspects of engineering and specifications are usually determined during the design of a project. In this case, the design of the tidally influenced side of the dike was left to the subcontractor’s means and methods to provide a stable work platform through which the OCSP® system could be driven. The plan's prescriptive use of a land-based platform with a 1.5:1 fore slope contributed to the unsuccessful completion of this project.
- The specification requiring previous pile-driving experience did not extend to the crews and provided little to mitigate the most difficult and risky element of work on the 2008 MTR Project. The lack of similar experience contributed to the unsuccessful completion of this project.
- The prescriptive construction contract general note in Addendum #9 (Item H25 in Table 8.1-1 and Appendix H) requiring that the vibratory hammer be employed first until refusal, prior to using the impact hammer, contributed to slope movement during construction.
- The direction given to the Contractor to continue driving piles after practical refusal had been achieved likely contributed to pile damage during construction.
- In October 2009, inspection dredge work was performed in front of Cells 27-38 and dive inspections were performed after the dredging was completed. Damaged piles found during that work should have flagged issues regarding the installation of the OCSP® system. A rigorous analysis at this time could have been conducted to determine the cause of damage before additional sheet pile driving work was allowed to proceed.
- The lack of enforcement or development of a system for the systematic verification of interlock integrity contributed to the unsuccessful completion of this project.
- Starting and stopping of work due to the whale sightings increased the impact of slope movement on the vertical alignment of individual piles driven on the tidal side of the berm and significantly decreased the amount of time available to drive the sheet piling on this project
- Thickness of the initial galvanizing likely contributed to binding of the interlocks, which contributed to the unsuccessful completion of this project.
- The listing of the Cook Inlet beluga whale as a Threatened and Endangered species and associated permit conditions contributed to the unsuccessful completion of this project. The listing affected the construction phasing and the pile-driving methods, and significantly reduced the available work window in which pile-driving could be accomplished.

The challenges of a severe marine environment with 40-foot tidal swings, 2- to 3-knot currents, and ice conditions combined with 90-foot sheet piles contributed to the unsuccessful completion of this project. While OCSP® has been used successfully on numerous projects since the early 1980’s, the installation at the Port of Anchorage is seriously flawed. Any major construction project involves risks and it is the understanding and management of the risks that determine whether or not the project is ultimately successful. As we have described in this section, the number of risks for the OCSP® system projects in the PIEP are substantial. And while some of the OCSP® installation risks on their own can be managed, the sheer number of risks on this project tends to magnify the chances for construction problems. As a result, and certainly with the benefit of hindsight and time to reflect on the project, PIEP had a low chance of success from the outset. It will be important to learn from what has happened during the construction from 2006 - 2011 and make sure that future risks are better managed. On such a vital and visible project for the MOA and POA it will be critical to chart a path forward that keeps the risks at a minimal level, so that the next phases of this project are successful.

FIGURE 8-1. North Backlands, 2006

FIGURE 8-2. Dry Barge Berth Complete, July 2008

FIGURE 8-3. Wet Barge Berth and North Extension, June 2010

FIGURE 8-4. Ice Conditions, January 8, 2008

FIGURE 8-5. Dredging, June 2008

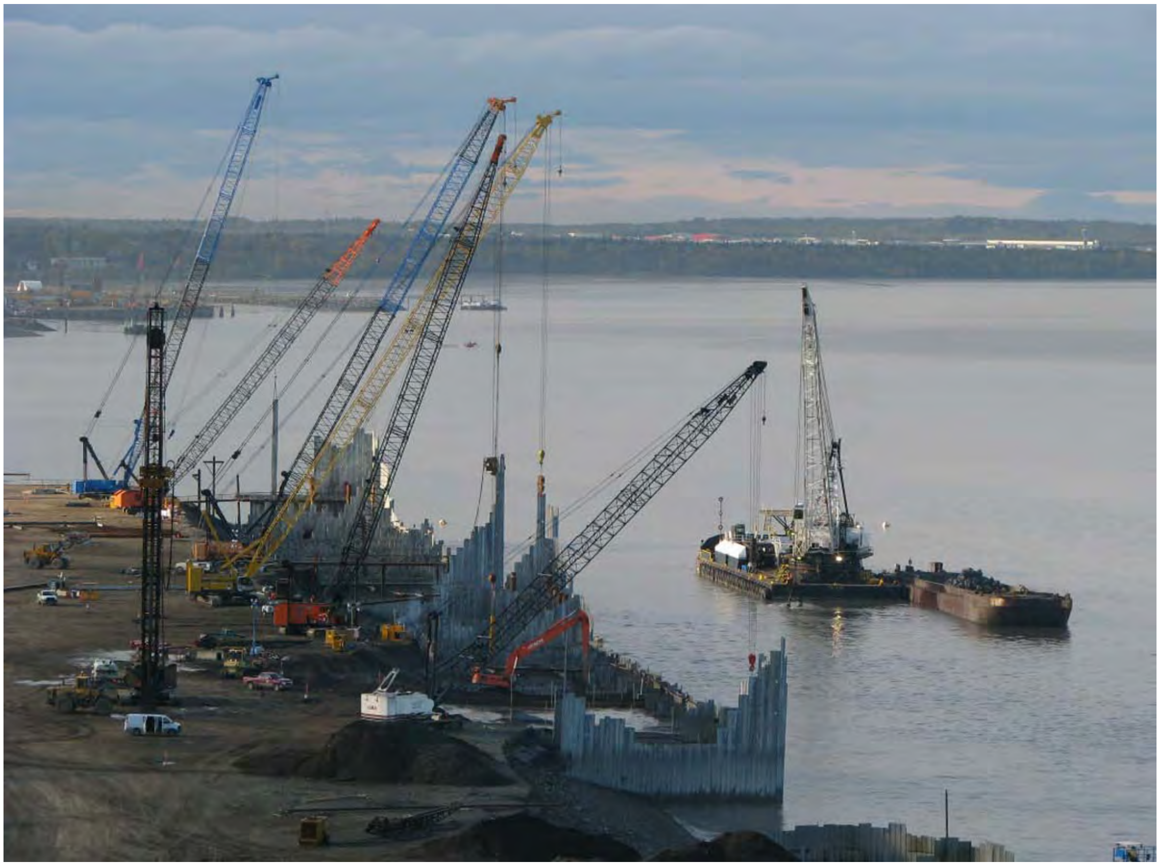


FIGURE 8-6. North Extension, September 2009

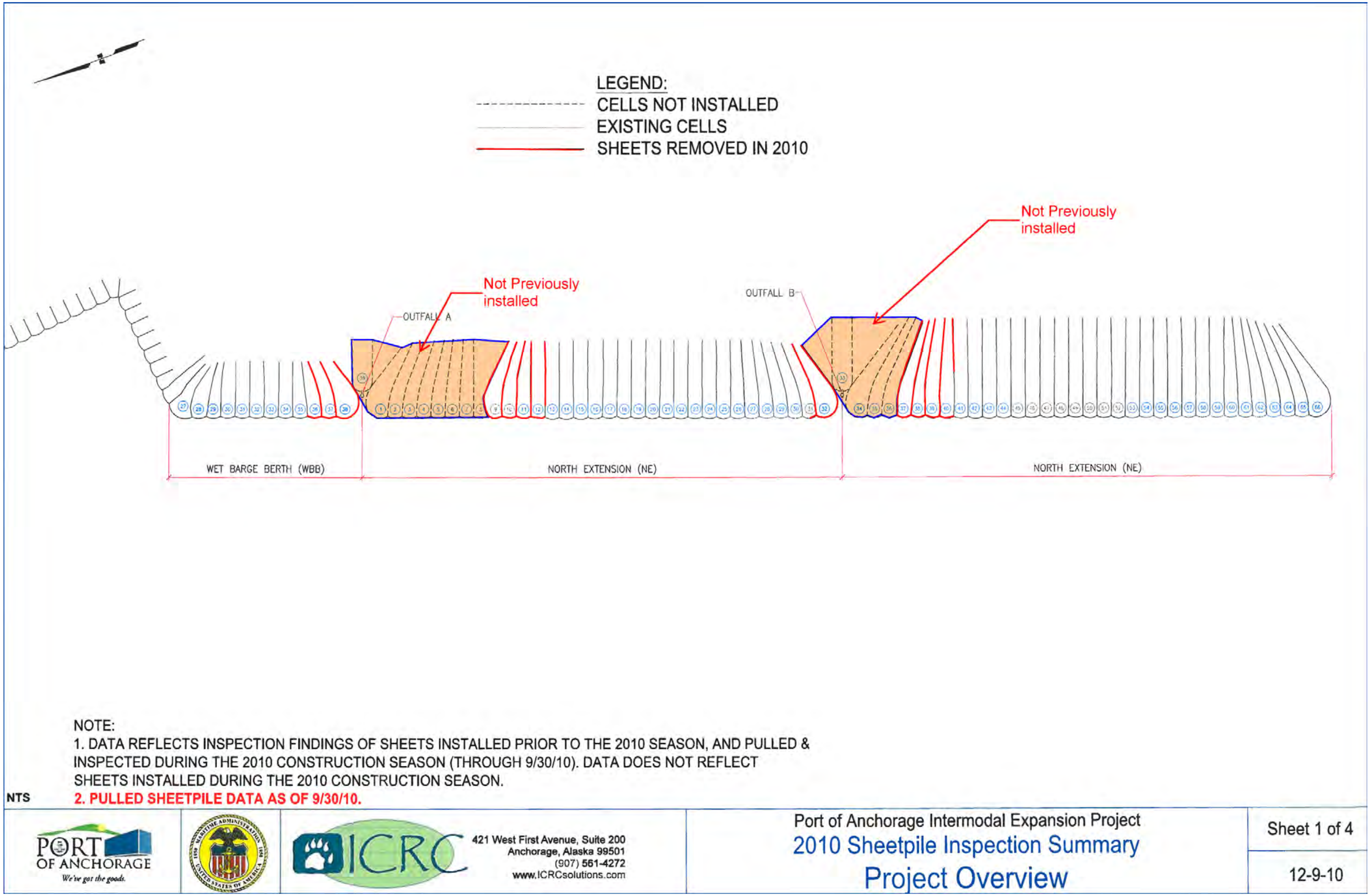


FIGURE 8-7. 2010 Sheet Pile Inspection Summary - Project Overview

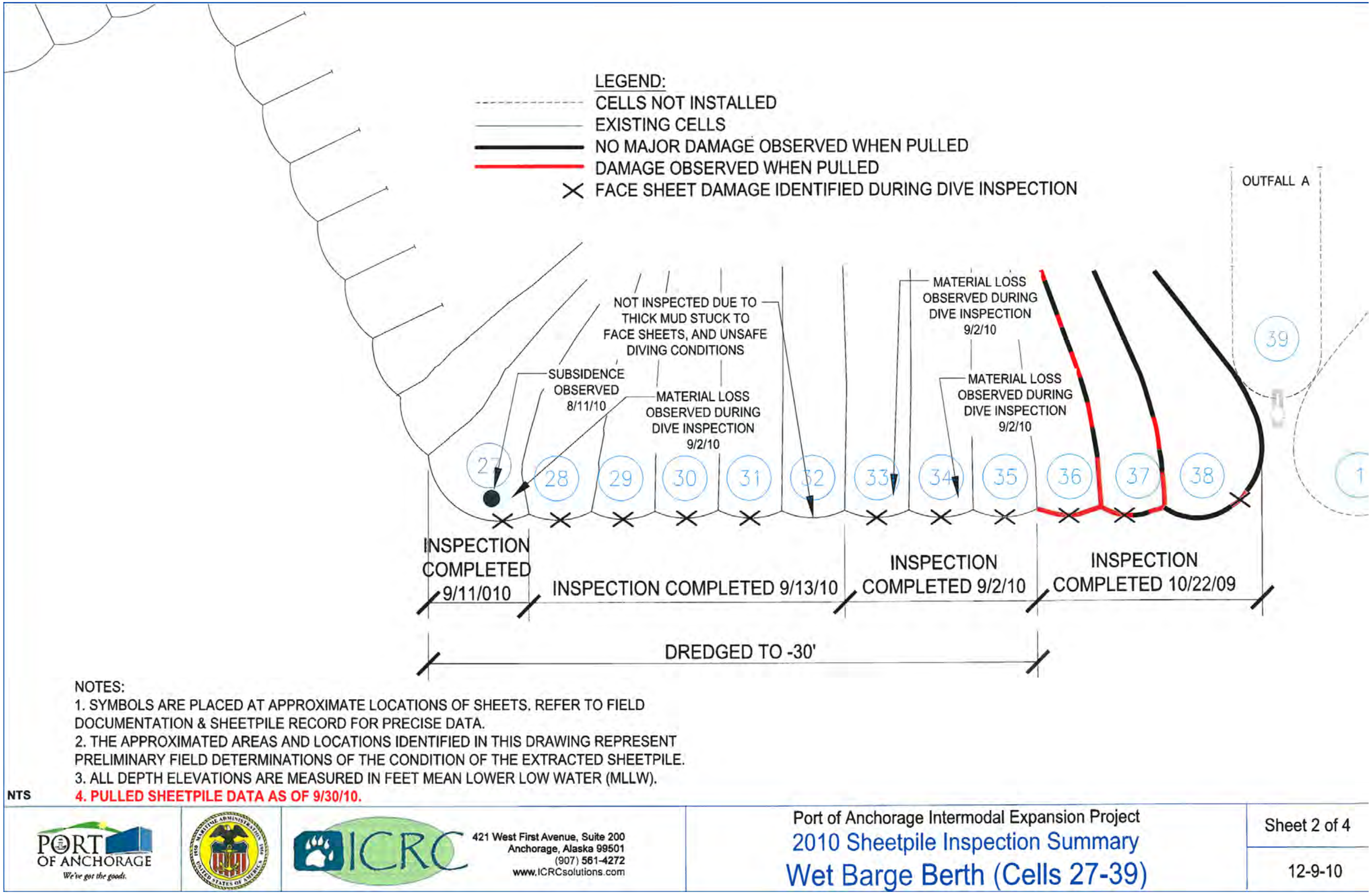
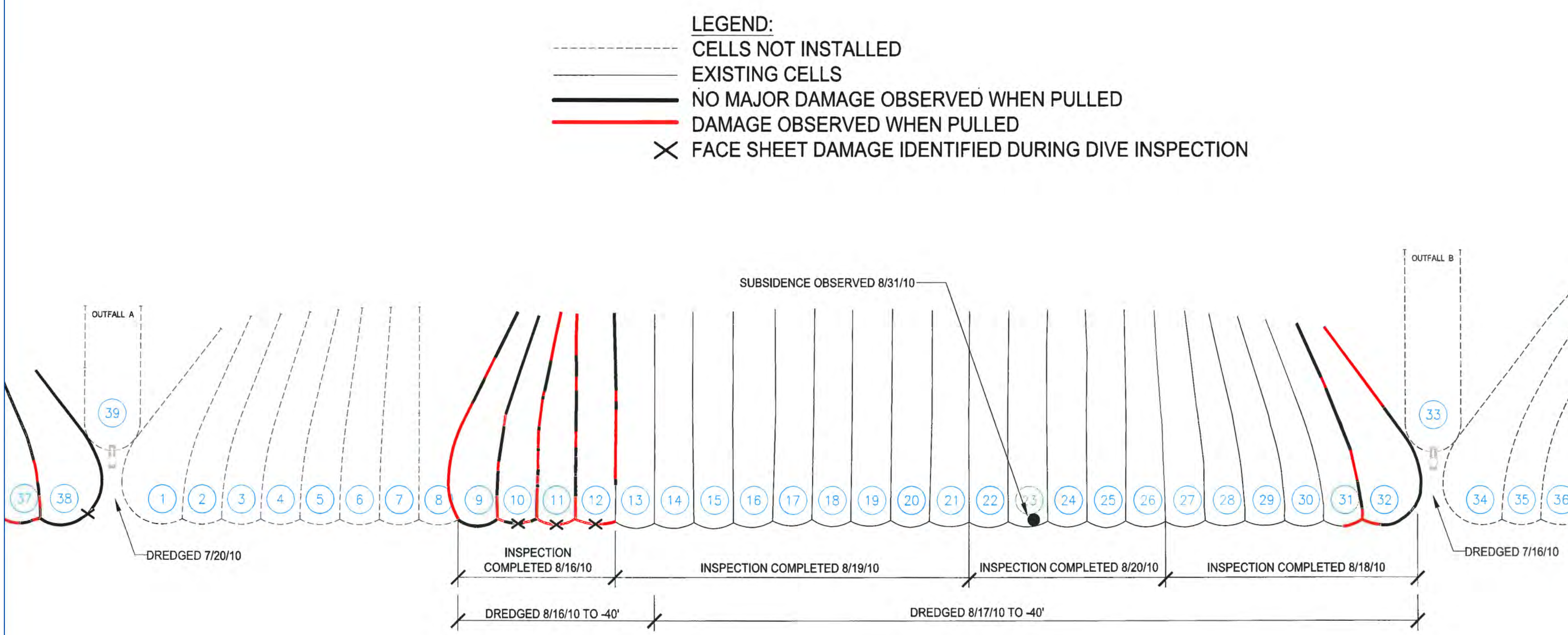


FIGURE 8-8. 2010 Sheet Pile Inspection Summary - Wet Barge Berth (Cells 27-39)



NOTES:

1. SYMBOLS ARE PLACED AT APPROXIMATE LOCATIONS OF SHEETS. REFER TO FIELD DOCUMENTATION & SHEETPILE RECORD FOR PRECISE DATA.
2. THE APPROXIMATED AREAS AND LOCATIONS IDENTIFIED IN THIS DRAWING REPRESENT PRELIMINARY FIELD DETERMINATIONS OF THE CONDITION OF THE EXTRACTED SHEETPILE.
3. ALL DEPTH ELEVATIONS ARE MEASURED IN FEET MEAN LOWER LOW WATER (MLLW).
4. **PULLED SHEETPILE DATA AS OF 9/30/10.**

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North Extension (Cells 1-33)

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FIGURE 8-9. 2010 Sheet Pile Inspection Summary - North Extension (Cells 1-33)

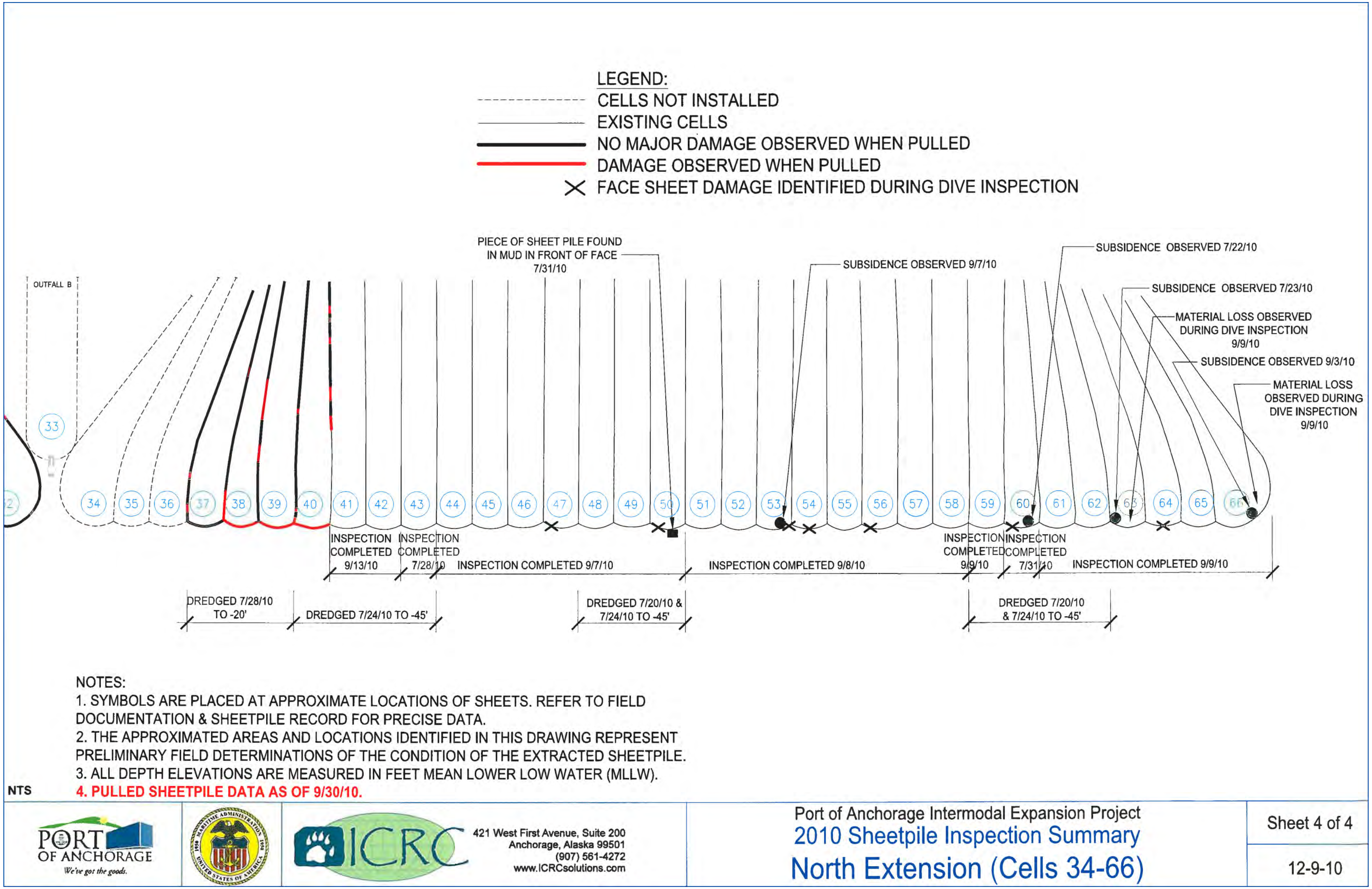

FIGURE 8-10. 2010 Sheet Pile Inspection Summary - North Extension (Cells 34-66)

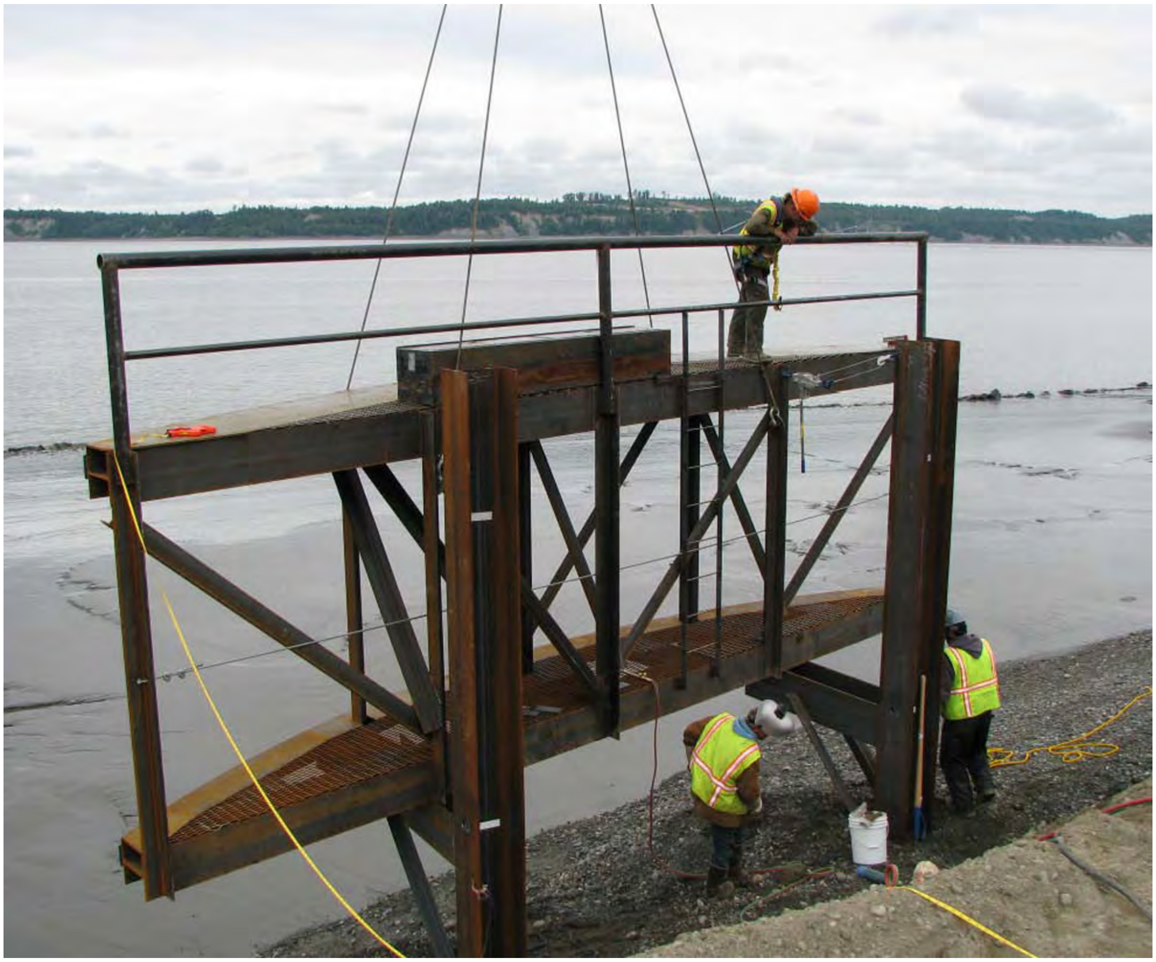

FIGURE 8-11. Pile Alignment System, July 2008