SECTION 9
Independent Design

An independent design concept was also identified and evaluated by CH2M HILL as part of the POA suitability study. The intent of this task was to determine whether or not modifications could have been made to the proposed OCSP® system to either meet or come closer to meeting the PIEP design criteria summarized in Section 2 of this suitability study. The independent design development was performed with the constraints that the basic project layout, loading requirements, dredge depths, and OCSP® geometry would remain the same. Specifically, the radius of the OCSP® facewall and the types of sheet piles would not change. However, the configuration of the tailwall, the characteristics of the backfill material, and the resistance of the underlying BCF clay could be modified such that FS values for global stability and displacements for seismic loading would either meet or be more consistent with the PIEP design criteria. The modifications should ideally be constructable at a reasonable cost, and it was further assumed that none of the current OCSP® project had been built. Results of this independent design are summarized in the following discussion.

9.1 Objectives of Independent Design

This section provides background information for the independent design. First, it discusses the deficiencies observed with the existing OCSP® system, both from a design and construction perspective. This discussion of deficiencies is based on observations and conclusions reached in previous sections of this suitability study report. The deficiencies include FS values that are too low and seismic displacements that are too high. Based on these deficiencies, an attempt was made to identify concepts that could potentially be used to improve the low FS values and reduce the excessive seismic displacements. However, as discussed in the following subsections, the severity of the seismic loading problem could only be partially remediated.

9.1.1 Existing Deficiencies

Results of limit-equilibrium and numerical analyses summarized in Sections 5 and 7, respectively, of this suitability study, suggest that the current design of the OCSP® system does not satisfy the PIEP design criteria in at least two areas. First, FS values for global stability are less than those normally expected for static, short-term (end-of-construction) loading and for long-term, undrained loading (operational loading and extreme low tide).

The low FS values for static loading also have implications regarding the ability of the OCSP® system to meet project design criteria for seismic loading. The design criteria require minimal displacements of the structure during the OLE, accept larger deformations during the CLE, and require the essential facility part of the expansion to be usable within a short period of time after the MCE. In the PIEP design criteria, the acceptable displacements for the OLE, CLE, and MCE are 3 inches, 6 inches, and 18 inches, respectively. These deformations are in addition to those that occur during gravity loading. Results of the limit-equilibrium and numerical analyses summarized in Sections 5 and 7 show that the amount of displacement, if little strength loss occurs during cyclic loading, will likely be closer to 6 inches during the OLE, will potentially range from 18 to 23 inches during the CLE, and could be 28 to 37 inches during the MCE. These estimates are based on simplified Newmark sliding-block analyses. Results also show that the amount of deformation could be 10 feet or more if strength degradation occurs similar to results of constant-volume ring shear tests conducted for the project and observed for similar tests on soils from Fourth Avenue and Port MacKenzie, as well as degradation observed at various locations in the Anchorage area during the 1964 Alaska earthquake. Results of FLAC^{3D} analyses also provide support for the possibility of large, 1964-Alaska-earthquake-type displacements.

Whether very large displacements occur similar to what was observed at Fourth Avenue, L Street, Government Hill, and Turnagain Heights during the 1964 Alaska earthquake, or smaller permanent displacements obtained from the Newmark sliding-block analyses occur, deformations of the OCSP® system under all earthquake shaking levels exceed the PIEP design criteria.

In this assessment of global stability and performance during seismic events, limited consideration was given to the local damage that occurred during construction of the OCSP® system. This damage includes sheet piles that are out of interlock along the face of the OCSP® facility. Although less certain, it is possible that at least some sheet piles along the tailwall are also damaged and may not provide adequate support during either future dredging or a seismic event. These current known deficiencies on the facewalls and potential deficiencies in the tailwalls are expected to result in additional local failures of the OCSP® system, such as facewall failures and possibly failures of the tailwall, during the CLE and MCE seismic events.

The modeling results showing low FS values and construction observations showing damaged sheet piling suggest that a number of factors have to be addressed during an independent design. To address the global stability issues at the POA site using the OCSP® technology, either the forces causing instability must be decreased or the reaction of the soil to loading must be increased. The construction method must be changed to avoid the damage that occurred during sheet pile installation for the existing OCSP® facility at the POA. The next subsection describes an approach intended to address these requirements. Only preliminary stability and displacements checks on the adequacy of this approach have been performed; detailed studies would be required before adoption of any of these concepts to confirm that the concepts are both technically viable and cost-effective.

9.1.2 Proposed Design and Construction Concepts

The proposed concepts were developed assuming that the OCSP® system would be constructed in the same location using the same sheet pile types and same sheet pile geometry. Loading conditions from operational loads and from seismic events would be the same as the original design. Tidal fluctuations and groundwater elevations would also be the same. It was further assumed that issues related to construction, such as sheet piles driven out of interlock, would be successfully addressed by revised construction methods.

The main modifications that could be made were to the forces causing and resisting loads, as follows:

- **Reduced Driving Force.** This option involves reducing the driving force causing instability. The driving force is primarily caused by the free height of the wall and corresponding weight of the granular backfill within the OCSP® walls. This backfill results in a large driving force for static and seismic stability. If the weight can be reduced, performance is improved. Various light-weight materials have been used on other projects to reduce weight, as discussed below. Reduction of the live load from container storage is another possible variable; however, a decision was made to maintain the current level of live loading (that is, 1,000 psf) in order to preserve the intended function of the facility. With most modern ports, one of the primary functions is to store containers. A backlands live load for storage of 1,000 psf is standard for most ports. Reduction in the live load means that containers could not be stored as high, which limits future growth opportunities. Further, the POA wants to service North Slope oil field services, and these equipment and materials also require the capability of handling large live loads.
- **Increased Soil Resistance.** The other main option involves increasing the resistance of the BCF clay. Of particular importance is addressing the potential for large strength reduction during seismic-induced displacement of the BCF clay. Various options for increasing resistance exist, including use of structural elements such as piles or the use of ground improvement; each of these has significant cost implications. These options are also briefly discussed below.

9.1.2.1 Light-Weight Fills

Various types of light-weight materials have been used to reduce the weight of soil fills. These light-weight materials are usually used where soils beneath the fill are compressible. By using light-weight fills, the load on the compressible soil is reduced and settlement decreases. For the PIEP OCSP® system, settlement was not found to be a controlling issue, as discussed in Section 5.2. However, the light-weight fill would reduce the relative load on the OCSP® system, making it equivalent to a lower-height retaining structure. As shown in Section 5, the severity of the global stability and seismic displacement issues diminishes as the height of the wall decreases, and therefore, intuitively, use of these materials would lead to better performance.

A number of products have been used as light-weight fills, including (1) extruded polystyrene (EPS) which is usually referred to as geofoam, (2) low-density cellular concrete (LDCC), (3) expanded shale and clay, and (4) pumice (volcanic stone). Table 9.1-1 summarizes properties of these materials and provides rough order-of-magnitude costs. Other light-weight products also exist and have been used to reduce loads. These include wood fibers, fly ash, boiler slag, and air-cooled slag. These were not considered for use at the PIEP for various reasons, including potential environmental concerns from contact with seawater.

TABLE 9.1-1
Properties of Some Common Light-Weight Fills

Light-Weight Fill Type	Unit Weight (lb/ft^3)	Compressive Strength	Approximate Cost ($/cy)
EPS/Geofoam	1 to 3	5 to 44 psi[a]	100 to 130
LDCC	20 to 50	100 to 1,000 psi[a]	70 to 110
Expanded Shale or Clay	24 to 63	ϕ = 37 to 45 degrees	50 to 60
Pumice (Volcanic Stone)	21 to 40	ϕ = 45 to 50 degrees	unknown

[a] Strength of EPS based on unconfined compressive strength (q_u) at 5 percent strain; strength of LDCC based on q_u at 28 days.

The idea of using light-weight fills for port facilities is not new. A case history was presented by Porbaha and Yamane (2002) where the area behind a caisson seawall was filled with LDCC—referred to by Porbaha and Yamane as foamed concrete—as part of damage repair following the 1995 Kobe earthquake. Figure 9.1-1 shows the concept used at Kobe.

Of the four types of light-weight fills identified in Table 9.1-1, LDCC seems to be the most suitable for application at the POA because it has good resistance to hydrocarbons and can have high strength. Geofoam was not considered, despite a supplier being close to the POA, because of its high cost and vulnerability to degradation from hydrocarbon spills. Expanded shale and clay are not available in Alaska and the cost of importing them from a source such as California would be cost-prohibitive. While ample supplies of pumice are available in Alaska, no commercial source of this material is currently available, and developing such a source would be very expensive. The LDCC requires specialized equipment and products to create the light-weight material; however, the necessary equipment is available in Alaska and the foaming agents can be obtained.

9.1.2.2 Increased Soil Resistance

Performance of the OCSP® system at the POA is controlled by the strength of the BCF clay, particularly when the BCF clay undergoes large displacements. Strength of the BCF clay was found to decrease by up to 70 percent when large shear displacements were imposed during constant volume ring shear tests, similar to what was observed from ring-shear testing of BCF clay from Fourth Avenue and Port MacKenzie and what was back-analyzed at Fourth Avenue following the 1964 Alaska earthquake. To counter this strength loss, the resistance of soil shear through the BCF clay must be increased to limit the potential for large shear displacements. Two approaches were considered for accomplishing this increase. One involves use of a structural system comprised of secant or tangent piles. The structural system could also include a forest of "pinch" piles, such as the ones used at the Port of Seattle for liquefaction mitigation. The other approach involves the use of ground improvement. The ground improvement approach has been used at other ports to improve the ground; for example, at Port of Oakland following the Loma Prieta earthquake.

The structural approach involves extending the secant, tangent, or pinch pile system through the critical slip surface in the BCF clay. Forces above the slip surface are transferred to more competent foundation materials below the surface through the bending stiffness of the structural elements. These systems would be installed after the backfill has been placed and the pile systems are constructed, as follows:

- **Secant or Tangent Piles.** Secant and tangent piles involve a wall created by bored piles. These walls can be 3 to 5 feet thick. The secant piles overlap one another, while the tangent piles are tangent. This approach to stabilization has been widely used for slopes and for bridge approach fills, often where soils are liquefiable. Either tie-back or deadman anchors can be used to provide additional sliding stability when this approach is used. If tie-back anchors are used, the anchor must extend past the critical slip surface to develop reaction to load. Alternatively, inclined pile systems with a deadman wall have been used at port facilities to develop the reaction loads. Cost of this approach is relatively high, particularly where wall heights are similar to those that would be required at the POA.

- **Pinch Piles.** This approach involves driving a large number of piles at spacings that range from 5 to 10 feet. The piles are not tied together with a pile cap. Resistance is developed through bending of the pile. For granular soils, additional soil resistance is developed by soil densification; however, for a clay site, such as the POA, only the bending resistance component occurs. The piles can be of various materials. Timber piles have been used in some locations. This approach is lower in cost per unit area than the secant or tangent pile approach, but it would likely require a larger area of improvement.

Various types of ground improvement can be used, including stone columns, deep soil mixing (DSM) columns, and jet grouting columns.

- **Stone Columns.** The stone column approach is similar to the vibracompaction already used at the POA to densify backfill soil within the OCSP® cell walls. This approach has been used at other ports including the Port of Tacoma and the Port of Los Angeles. The stone columns involve placing densified gravel or crushed stone columns at 5- to 10-foot spacing. Improved resistance is achieved through the high frictional capacity of the stone columns and any densification that occurs in the soil surrounding the column. As discussed above, little improvement would be expected in the BCF clay. Although this approach has been used over water at Roberts Bank near Vancouver, BC, and probably elsewhere, for densification of granular soils, logistics at the POA would be difficult because of the large tidal range that occurs. This condition would likely result in having to place the stone columns once the OCSP® cells were constructed above tidal levels. With special equipment, stone columns can now be placed to nearly 200 feet below the ground surface. This is the depth that would be required at the POA, making this approach very difficult and expensive.
- **Deep Soil Mixing (DSM) Columns.** DSM methods involve mixing in-place soil with cement in columns or in walls forming a grid. The diameter of the column or wall can be 2 to 3 feet or more; the strength of the mixed soil can range from less than 100 psi to 400 or 500 psi, depending on soil type and the amount of cement included in the mix. This approach has been used at the Port of Oakland to stabilize a marginal wharf as part of a liquefaction mitigation program. One of the concerns with this approach is the potential runoff from the mixing process into the seawater. The high pH of the mix would not meet environmental requirements, likely forcing the improvement to be done in an area retained by sheet piles or a similar containment system or after the OCSP® cells were constructed above tidal levels. The disadvantage of placing the DSM columns after the backfill is in place is the large depth of granular fill that would have to be penetrated. It is also questionable whether current equipment could reach the depth within the BCF clay that would be required to control stability and displacements.
- **Jet Grouting Columns.** This approach also involves mixing cement in the soil. Mixing is performed using high-pressure jets. The resulting columns can be 5 to 10 feet in diameter, depending on soil type and equipment features. Strength of the jetted column can range from 100 to 300 psi. This approach creates a significant amount of spoil, often as much as 100 percent of the jetted column volume. The presence of cement in the mix means that the spoil would have to be fully contained. The benefit of the jet grouting relative to the DSM is that it can reach deeper areas below the ground surface. As with the other procedures, the large tidal fluctuations at the POA site make this application more difficult, but still possible. For example, the jet grouting would have to be conducted after the OCSP® cells are constructed above tidal levels, when containment of spoil is easier. This approach is more suited for deep ground improvement than either the stone column or the DSM method.

9.1.3 Independent Design Description

Based on the results of this review, it appears that a combination of ground improvement and light-weight fills could be used at the POA to provide added soil resistance and reduced driving force to improve global stability and reduce seismic displacements. The conceptual approach for construction is challenging and would involve separate construction sequences depending on whether the ground improvement is on the land side or sea side of the bulkhead. The construction sequences below are conceptual:

<u>Land Side</u>

- Dredging existing estuarine deposits within the zone that would be improved
- Placing OCSP® facewalls and tailwalls from the sea side or work trestle from land
- Placing granular fill to elevation +20 feet MLLW
- Filling from elevation +20 to +35 feet MLLW within the tailwall section behind the OCSP® face with LDCC
- Drilling through the fill and improving ground from bottom of fill to the top of the glacial till layer at elevation -150 feet MLLW
- Densifying granular fill to elevation +35 feet MLLW

<u>Sea Side</u>

- Dredging existing estuarine deposits within the zone that would be improved
- Improving ground from sea floor to the top of the glacial till layer at elevation -150 feet MLLW

The cross-section for this general approach is shown in Figure 9.1-2. It has been assumed in the above construction sequences that the contractor can develop construction methods that adequately address environmental and technical constraints associated with the work. The seaside ground improvement would need to be sensitive to the aquatic marine environment and therefore the likely option would be stone columns. Behind the bulkhead, other grout-intensive systems such as jet grouting would be considered.

A series of structural evaluations, limit-equilibrium stability and displacement checks, and numerical analyses were conducted to evaluate this independent conceptual design and construction sequence. These analyses are summarized in Sections 9.2, 9.3, and 9.4.

9.2 Structural and Construction Considerations for Independent Design

Figure 9.1-2 shows the typical cross section of the independent design at Section 2-2 (see Section 5 for discussion of sections). This location has the tallest exposed height of the OCSP® wall throughout the North Expansion project with future harbor dredging at this location expected to reach -45 feet MLLW. Considering a 6-foot allowance for overdredging and storage dredging, the tip elevation of the facewall sheets is set at -60 feet MLLW, or 9 feet below the final mudline. The top elevation of the facewall sheets is set at elevation +30 feet MLLW, resulting in a total pile length of 90 feet. The tailwall is roughly rectangular in shape, with a top at elevation +24 feet MLLW and a bottom at elevation -51 feet MLLW; the tailwall extends 144 feet landward from the bulkhead control line.

The location and geometry of the independent design closely follow the original design relative to sheet pile layout. It is assumed that the same top of wall elevations would need to be accommodated in the independent design. The independent design is composed of the same 27.5-foot radius cell as the original design, as shown in Figure 9.1-3. The facewall consists of 17 PS31 sheet piles, and the tailwall will be composed of PS27.5 sheet piles. Facewall sheets and tailwall sheets connect to each other in 120-degree angles at the wye connection. An end anchor is provided at the end of each tailwall.

The FS values for interlock tension and tailwall pullout for the independent design are presented in Table 9.2-1. Procedures used to estimate interlock tension and tailwall pullout are the same as those discussed in Sections 2 and 5. For all load cases, the FS values exceed the allowable values.

TABLE 9.2-1
Independent Design – 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	3.1	4.0	2.9	2.8	2.1	1.7	3.1
Required FS	1.5	2.0	2.0	1.5	1.3	1.1	2.0
Pullout of Tailwall	5.2	6.6	4.8	2.2	1.6	1.2	2.4
Required FS	1.3	1.5	1.5	1.3	1.1	1.0	1.3

It is assumed that the same corrosion protection system, namely galvanization combined with cathodic protection, would be used for the independent design. Using the same assumptions as described in Section 6.2, the FS values for internal stability at the end of the 50-year service life are calculated and presented in Table 9.2-2. Values in Table 9.2-2 suggest that in all load cases, a sufficient FS can be achieved at the end of service life. The FS values for the independent design are higher than those discussed in Section 6 for two reasons: (1) the earth pressure on sheet piling is less, due to the light-weight fill and (2) the increased strength of the improved soil. These two factors result in less facewall and tailwall stress.

TABLE 9.2-2
Independent Design Horizontal Tension Factor of Safety – Cathodic Protection System with 40-Year Service Life

		Loading Conditions					
Structural Component		**Long-term Static (Drained)**	**Long-term Static (Undrained)**	**OLE**	**CLE**	**MCE**	**Post-Earthquake**
Facewall Sheet (PS31)	Original	4.0	2.9	2.8	2.1	1.7	3.1
	After 50 yrs	3.4	2.4	2.3	1.8	1.4	2.6
Tailwall[a] (PS27.5)	Original	4.0	2.9	2.8	2.1	1.7	3.1
	After 50 yrs	3.2	2.3	2.2	1.7	1.4	2.5
Required FS		2.0	2.0	1.5	1.3	1.1	2.0

[a] Critical section at the wye connection.

9.3 Global Stability Evaluations

Global stability analyses were conducted for the independent design following the procedures described in Section 5.2.6 of this suitability study. Soil strengths within the BCF clay were assumed to be the same as used for the evaluations of Sections 2-2 and 3-3. A zone of ground improvement was defined from just above the top of the BCF clay to elevation -150 feet MLLW, where the glacial till occurs. The width of the ground improvement was determined from the analyses; the strength of the improved zone was specified as 5,500 psf (that is, 38 psi) based on preliminary analyses performed to determine the required strength.

The type of ground improvement was not specified. Rather, it was assumed that contract documents would be written such that the contractor was required to meet a composite strength of 5,500 psf, which would be a weighted average between the strength of the reinforced zone and the existing soil. Typically area replacement ratios of 25 to 30 percent are used to improve soils. With this replacement ratio, the improved ground might have a strength ranging from 100 psi to 200 psi. However, the combination of replacement ratio and improved strength can be varied to achieve an optimum combination of percent cement, replacement ratio, and diameter of improved ground.

Light-weight fill was assumed to be low-density cellular concrete (LDCC) placed in the top 15 feet of the fill from the face of the bulkhead to the end of the tailwall. The density of the light-weight fill should not exceed 25 pcf, and compressive strength should not be lower than 100 psi. The rest of the fill material will be compacted granular fill. It is expected that the vibracompaction technique used in the original design can be used to achieve the desired fill material properties—those assumed for as-built analyses. The 15-foot depth of light-weight fill was selected to limit the amount of under-water work that would be required. It is possible that this depth could be increased by 5 to 10 feet with further construction evaluations.

Stability analyses were conducted for the cross-section shown in Figure 9.1-2 to evaluate FS values during construction, operational, and seismic loading. The required minimum FS values were 1.2 for short-term undrained loading (immediately following construction) and 1.5 for long-term drained and undrained loading. For the critical case of long-term static undrained loading, the FS was 2.0, as shown in Figure 9.3-1. The yield acceleration for seismic loading was determined to be 0.15g, as shown in Figure 9.3-2. As shown in Figures 9.3-3 through 9.3-5, the average seismic-induced deformations for the OLE, CLE, and MCE event are 1, 2, and 4 inches, respectively, based on the simplified chart solutions. The chart method followed procedures recommended by Bray and Travasarou (2007). This approach to estimation of seismic deformations was taken rather than the double integration method described in Section 5 to obtain a quick understanding of deformations that might exist. The simpler approach was thought to be sufficient to provide input for this conceptual independent design.

From these limit-equilibrium stability analyses it was concluded that the performance of the existing OCSP® system could be significantly improved by using ground improvement in combination with light-weight fill. This conclusion is not surprising as the inability to meet current design criteria in terms of both FS value and deformation results from the large driving force relative to the strength of the existing BCF clay. The results of the analyses also demonstrated that:

- The tailwall extension can be eliminated. The minimum tailwall length can be 1.5H, where H is the height of the tailwall measured from the finished grade to the dredge line.
- The ground improvement should extend at least 225 feet in width and to the top of the glacial till, which is roughly 110 feet below the top of the BCF clay layer. The minimum composite strength of the improved ground should be 5,500 psf. The composite strength in this context is the weighted average of the improved ground (for example, stone column, DSM, jet grout column) and the ground between zones of improvement.
- Light-weight fill should be placed above the groundwater elevation and extend to the end of the tailwall. The light-weight fill is placed above the groundwater elevation so that during construction the light-weight fill is placed in dry conditions. Although this is not a requirement for light-weight fills, it makes construction simpler. The maximum unit weight of the light-weight fill will need to be 25 pcf to be consistent with the composite strength and ground improvement dimensions described above.
- The backfill should be compacted granular fill with a minimum friction angle of 40 degrees. The extent of the backfill has not been determined in this independent design, although it should be placed the length of the tailwall as a minimum.

At this point, efforts to optimize this design have not been made, and some significant questions regarding the concept design still need to be resolved. For example, the ability to control spoils from the ground improvement work needs to be evaluated with ground improvement contractors and with regulatory agencies to determine what is feasible and what is required. Conclusions from these evaluations will likely define the type and extent of ground improvement.

9.4 Numerical Modeling of Independent Design

To further examine the performance of the OCSP® system, a FLAC^{3D} model was developed for the independent design of the OCSP® system. This model considered the interpreted wall-specific soil profile and a modified layout of the sheet piles and their depth of penetration. For this evaluation, the Section 2-2 used for the evaluation of the as-built OCSP® system was selected for the FLAC^{3D} model of the independent design. A typical cross-section for the independent design case is shown in Figure 9.1-2.

9.4.1 Independent Design Model Description

Similar to the previous models developed for the as-designed and the as-built evaluations, a total length of 689 feet and depth of 185 feet was used in the FLAC^{3D} model to analyze the independent design at Section 2-2. The cell width was 27.5 feet with a central angle of 60 degrees at the wall face curvature. 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 depth of the model was selected at the boundary between the BCF clay and the underlying glacial till at elevation -150 feet MLLW. A “quiet boundary” was assigned at this depth to avoid the potential from seismic wave reflections.
- The exposed facewall for the section analyzed has a total height of 81 feet ranging between elevation +30 feet MLLW at the top and elevation -51 feet MLLW at the over-dredge depth.
- The tailwall has a width of about 144 feet and extends between elevation +24 feet MLLW at the top and elevation -51 feet MLLW at the tip. No tailwall extension was used.
- An improved zone within the BCF clay extends to about 80 feet from the wye connection on the sea side and to 105 feet from the wye connection on the land side. The improved zone is assumed between elevations -41 feet MLLW and -150 feet MLLW, with a constant composite undrained shear strength of 5,500 psf.
- The light-weight fill zone at the top front section of the cell extends the length of the tailwall, approximately 144 feet in length, from the wye connection between elevations +20 and +38 feet MLLW, with an undrained shear strength of 7,200 psf.
- The dredge limits at the base of the wall extend a distance of 105 feet on the land side from the wye connection.

The FLAC^{3D} model for the independent design 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 9.4-1 shows the mesh size and the soil zones used for the evaluation of the independent design. With the exception of the improved soil zone, the soil parameters used in the analyses are the same as those used for the as-built evaluation, as summarized in Table 7.4-1. Within the improved zone, a constant undrained strength of 5,500 psf is assigned.

The procedures used for the static and dynamic analyses of the independent design model, elevations for the dredge depths, water levels for the loading conditions, and the magnitudes of the live load were similar to those considered for the as-built conditions evaluation.

9.4.2 Results of FLAC^{3D} Analyses

The results of the FLAC^{3D} analyses are presented for the static and the dynamic loading conditions in Table 9.4-1, and the plots for the predicted stresses and displacements in the structural elements and soil zones are shown in the Figures 9.4-2 through 9.4-16.

TABLE 9.4-1
Summary of Independent Design 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 (ksf)[b]	Tension	12.2	16.3	16.0	23.0	28.5	28.5
	Compression	2.1	1.0	-	-	2.1	3.1
	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	13.9	22.2	22.2	28.5[c]	40.3[c]	47.2[c]
	Compression	-	2.8	-	2.1	3.1	4.5
	Allowable Horizontal Tensile Stress	33.3	25.0	25.0	38.5	45.5	50.0
3. Facewall Maximum Horizontal Displacement (inches)		12.5	22	23	22 - 38	49 - >120	110 - >120
4. Tailwall Maximum Horizontal Displacement (inches)		11	19.5	20	-	-	-
5. 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] Localized stress concentration effects are excluded.

Based on the results presented in Table 9.4-1, the maximum membrane tensile stresses developed in the facewall and the tailwall under static loading conditions were about 16.5 and 22.5 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 smaller than the values estimated in FLAC^{3D} analysis. Using these values as a conservative upper bound, the estimated interlock stresses for the static loading conditions are generally within the allowable tensile stresses. The locations of the maximum membrane stresses in the facewall and the tailwall were observed to occur near the original mudline prior to the dredging. The zone of the maximum membrane stresses in the tailwall 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.

For static loading conditions, the lateral wall movements at the top of the sheet pile wall range between 10 and 20 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 was about 23 inches and generally occurred near the cell centerline at elevation -4 feet MLLW for the long-term drained loading conditions. The corresponding lateral movement for the wye connection at the maximum movement was about 20 inches. This behavior was consistently observed for all static loading conditions considered in the analyses.

For static loading, ground improvement was expected to have limited effects on facewall movement because much of the wall movement results from the active earth pressures imposed within the backfill zone above the zone of ground improvement. Although the reduced weight of the LDCC tends to reduce wall movement, relative to the overall height of the OCSP® system, the change of wall face pressures is relatively small.

The magnitudes of permanent deformations in the FLAC^{3D} analyses for the three design events were estimated from the Michoacan, Western Washington, and Puget Sound earthquake records. Figures 9.4-17 through 9.4-22 show the x-displacement-time histories for the three design earthquake ground motions. For the seismic loading conditions, the lateral permanent wall movements at the top of the sheet pile walls at the end of shaking events were less than 4 inches, less than 72 inches, and potentially greater than 120 inches for the OLE, CLE, and MCE, respectively. In all three cases, a sliding block was developed along a non-circular slip surface extending from the toe of the facewall and through the tailwall extension. The maximum horizontal tensile stresses developed in the facewall and tailwall sheet piles are generally less than the allowable tensile stress once local stress concentrations are disregarded. 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.

The large facewall movements for the CLE and MCE resulted from a combination of displacement between the toe and the top of the sheet pile and displacement from global movement below the tip of the sheet pile. Movement of the soil above the tip of the sheet piles range from 3.5 feet for the CLE to more than 10 feet for the MCE. These displacements suggest that one of the consequences of improving the ground was to transfer more energy to the bottom of the backfill because of the higher stiffness in the improved ground, resulting in large movement in the backfill above the improved ground. The large permanent displacements within the backfill mean that it may be necessary to stabilize materials within the active pressure zone in the backfill behind the OCSP® facewall to prevent large facewall movements.

9.4.3 Performance Evaluation Relative to Design Criteria

The estimated maximum membrane stresses and deformations of the independent design of the OCSP® system are shown in Table 9.4-1 and are summarized as follows:

- **Static Loading.** The estimated static deformations range between 30 and 12.5 inches for long- and short-term static loading conditions, respectively. The deformation estimate for the short-term static loading condition was less than the threshold of 18 inches for the maximum lateral displacement of the design criteria for the static loading conditions. However, the long-term drained and undrained deformation estimates exceeded the threshold for the maximum lateral displacement of the design criteria. The interlock tension stresses in the facewall and the tailwall were generally observed to have a FS well above the requirement.
- **Seismic Loading.** The estimated permanent lateral deformations based on results of the seismic analyses indicate that the threshold for the maximum permanent deformation of 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® walls using the ground motions of two earthquake records in the OLE, CLE, and MCE evaluations. The FS values for the interlock tension stresses in the facewall and the tailwall were generally greater than 1.1. However, minor stress concentrations were observed in the tailwall sheet piles in the upper 6 feet near the wye connection.

Since results of the FLAC^{3D} analyses indicate that displacements are greater than a few feet for the CLE and MCE seismic loading cases, the accuracy of the displacement estimate becomes questionable because of limitations in the large-strain FLAC^{3D} modeling procedure. At large displacements, the effects of constitutive behavior begin to have greater impact, resulting in larger differences between calculated and actual (for example, Mohr-Coulomb model may give higher displacements than when using a hardening model). These limitations are also related in part to difficulties in characterizing movement within any soil system once displacements exceed a few feet. Although higher strengths within the backfill material and BCF clay could have been assigned and evaluated, it was decided by the CH2M HILL design team that the practicality of further refinements to the independent design was insufficient to justify additional analyses and that no further FLAC^{3D} evaluations would be performed for the independent design.

9.5 Conclusions

An independent design evaluation was conducted for the OCSP® system. In this independent design, the basic geometry, sheet pile configuration within the OCSP®, and environmental and seismic loading conditions at the POA were maintained; however, changes were made to the backfill type and the resisting capability of the BCF clay in an effort to improve global stability and reduce seismic displacements. The proposed changes involved (1) using low-density cellular concrete (LDCC) in the upper 15 feet of backfill to lighten the loads within the OCSP® system and (2) improving the resisting capacity of the BCF clay below the OCSP® system by using ground improvement. The method of ground improvement was not specified; however, whatever method was selected had to provide a composite shear strength of 5,500 psf within a 225-foot-wide zone extending in front of and behind the OCSP® facewall.

Results of structural evaluations for this modified wall show that required FS values for interlock and pullout capacity will be met. FS values requirements are also satisfied when corrosion is considered for the 50-year design life. Global stability requirements are satisfied for both end-of-construction and long-term undrained operational loading conditions; seismic displacements using simplified chart-type displacement predictive methods also suggest that displacement criteria identified by the POA will be met. These displacement estimates do not account for large strength losses in the BCF clay that could occur if amounts of displacement are greater than the values estimated by the simplified chart-based displacement methodologies, nor do they account for the dynamics and soil-structure interaction mechanisms involved with the OCSP® system. When these effects were considered by conducting FLAC^{3D} analyses for the independent design, results suggested that much larger displacements could occur during the CLE and the MCE events. The cause of the large movements is believed to be related in part to the FLAC^{3D} modeling method.

It was concluded from this independent design that procedures could be used to improve global stability and reduce seismic deformation through the combination of light-weight fill and ground improvement. However, even with these modifications, it was unclear whether the PIEP design criteria would be met. This conclusion suggested that additional increases in the strength or zone of improved ground would be required. However, the concept of ground improvement was already at its limits from a cost and technical perspective for independent design, suggesting refinement was not practical. Based on this conclusion, it is not clear that alternate procedures could have been introduced during the original design to meet the PIEP design criteria, which implies that an alternate approach to port expansion is needed.

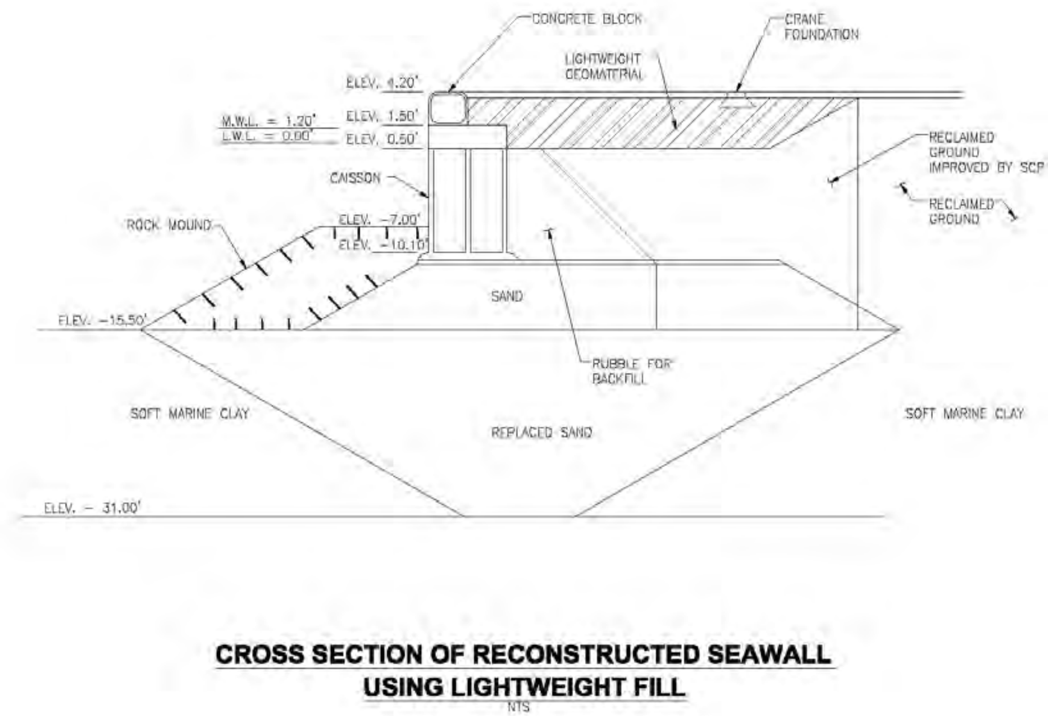

Figure 9.1-1. Cross Section of Reconstructed Seawall Using Light-weight Fill

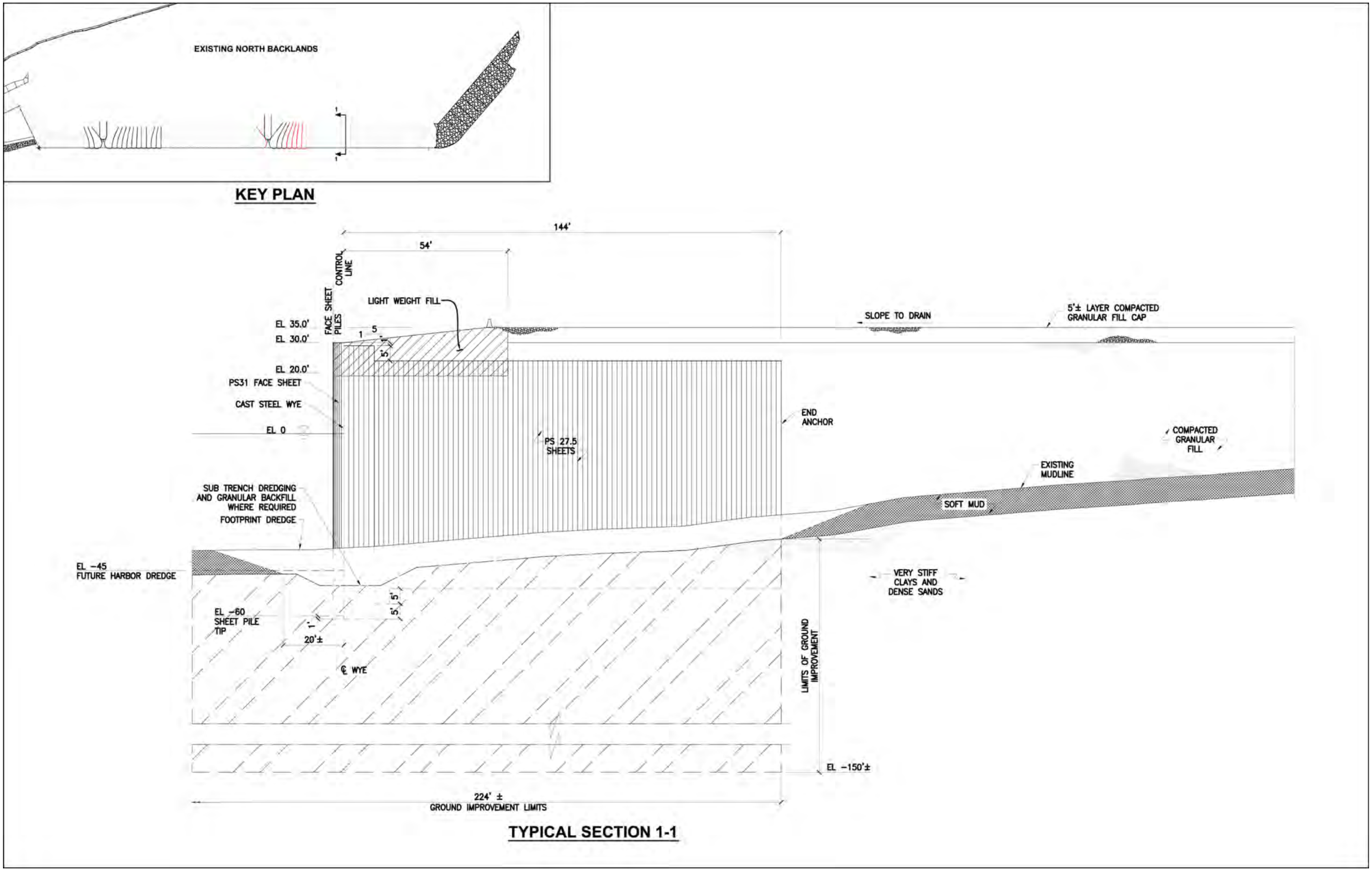

Figure 9.1-2. Typical Section for Independent Design

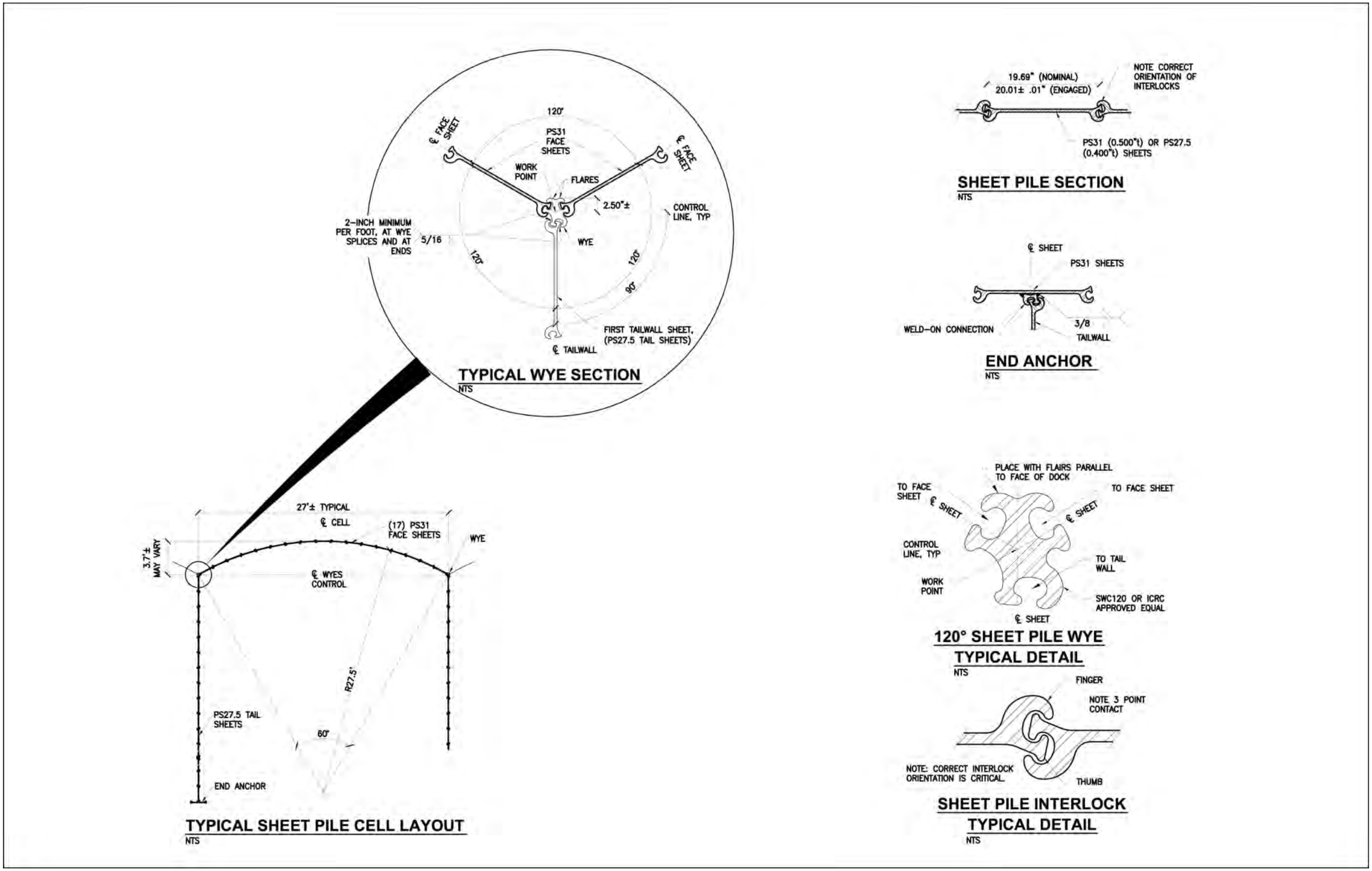

Figure 9.1-3. Open Cell Details for Independent Design

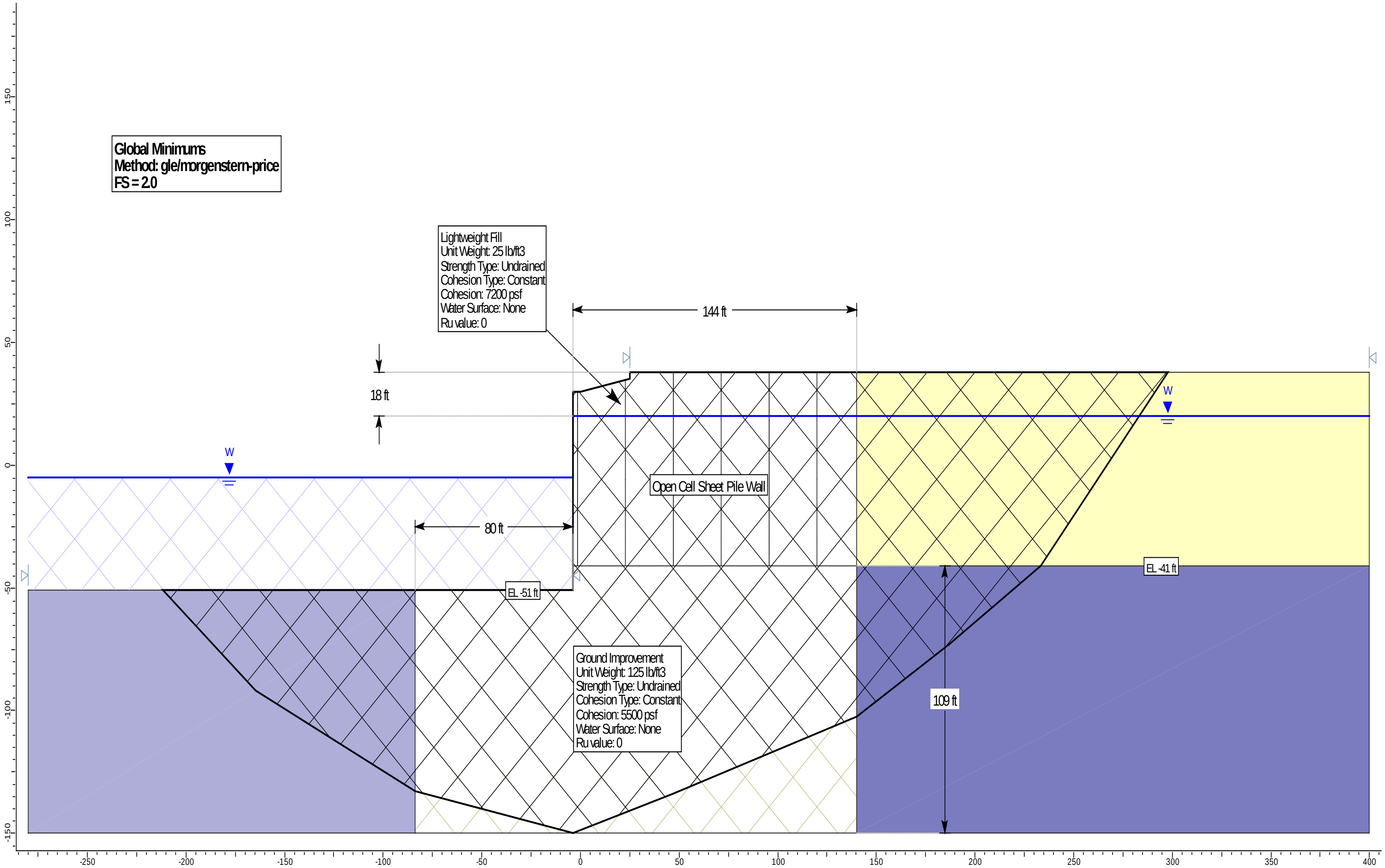

Figure 9.3-1. Results of Limit Equilibrium Stability Analyses for Long-Term Static Undrained Case (End-of-Construction)

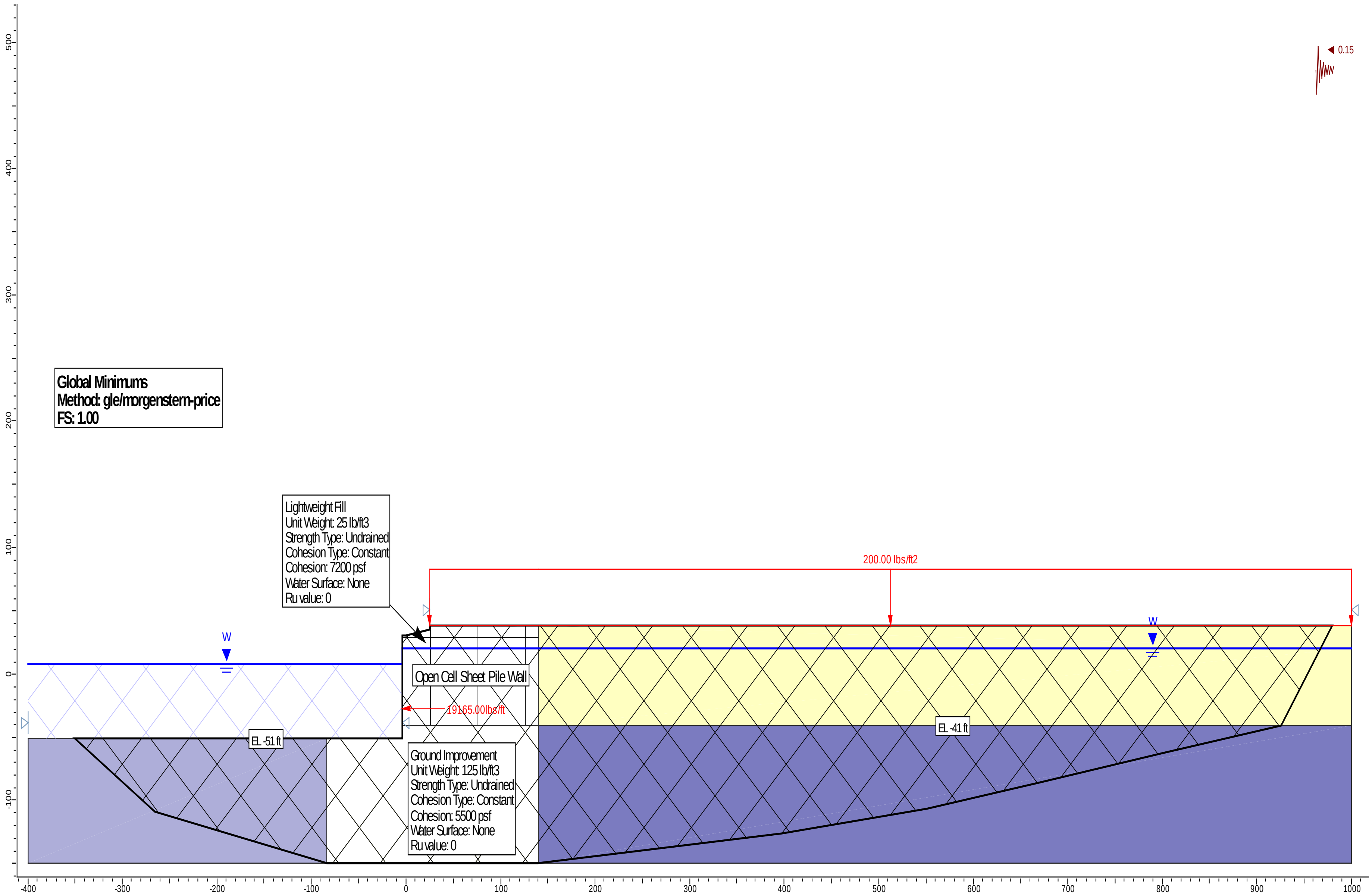

Figure 9.3-2. Yield Acceleration for Independent Design

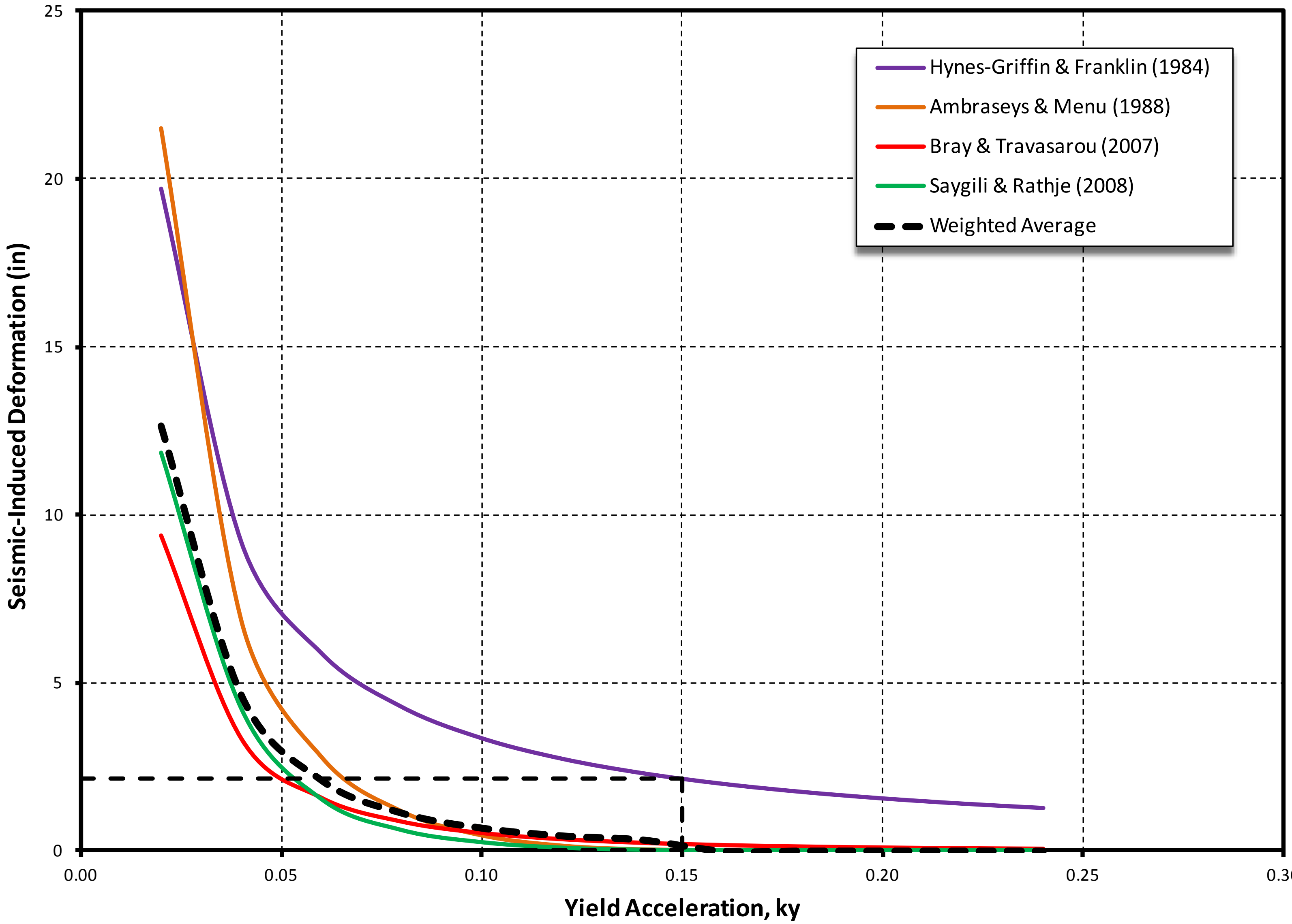

Figure 9.3-3. Estimated Deformation for OLE Event for Independent Design

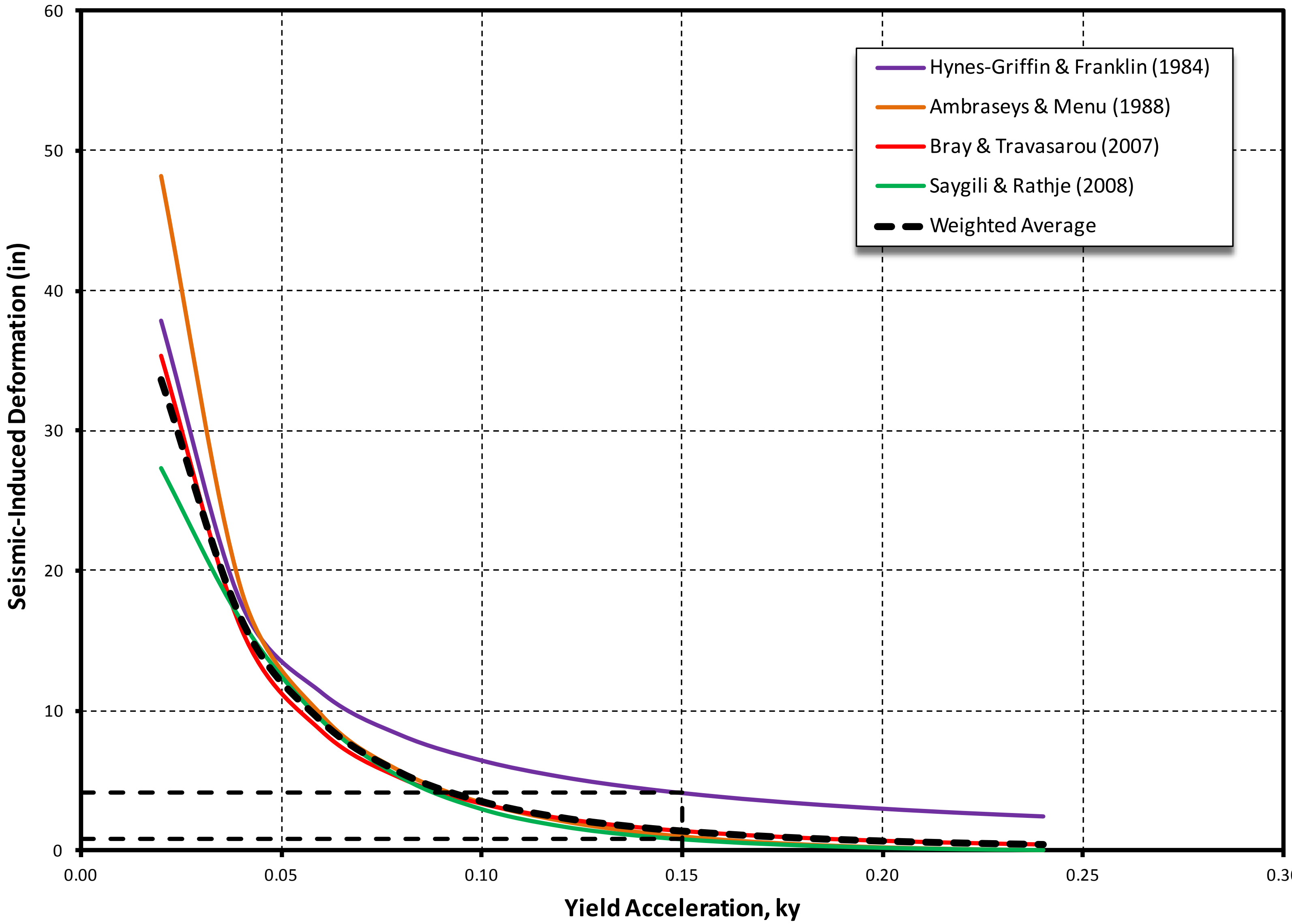

Figure 9.3-4. Estimated Deformation for CLE Event for Independent Design

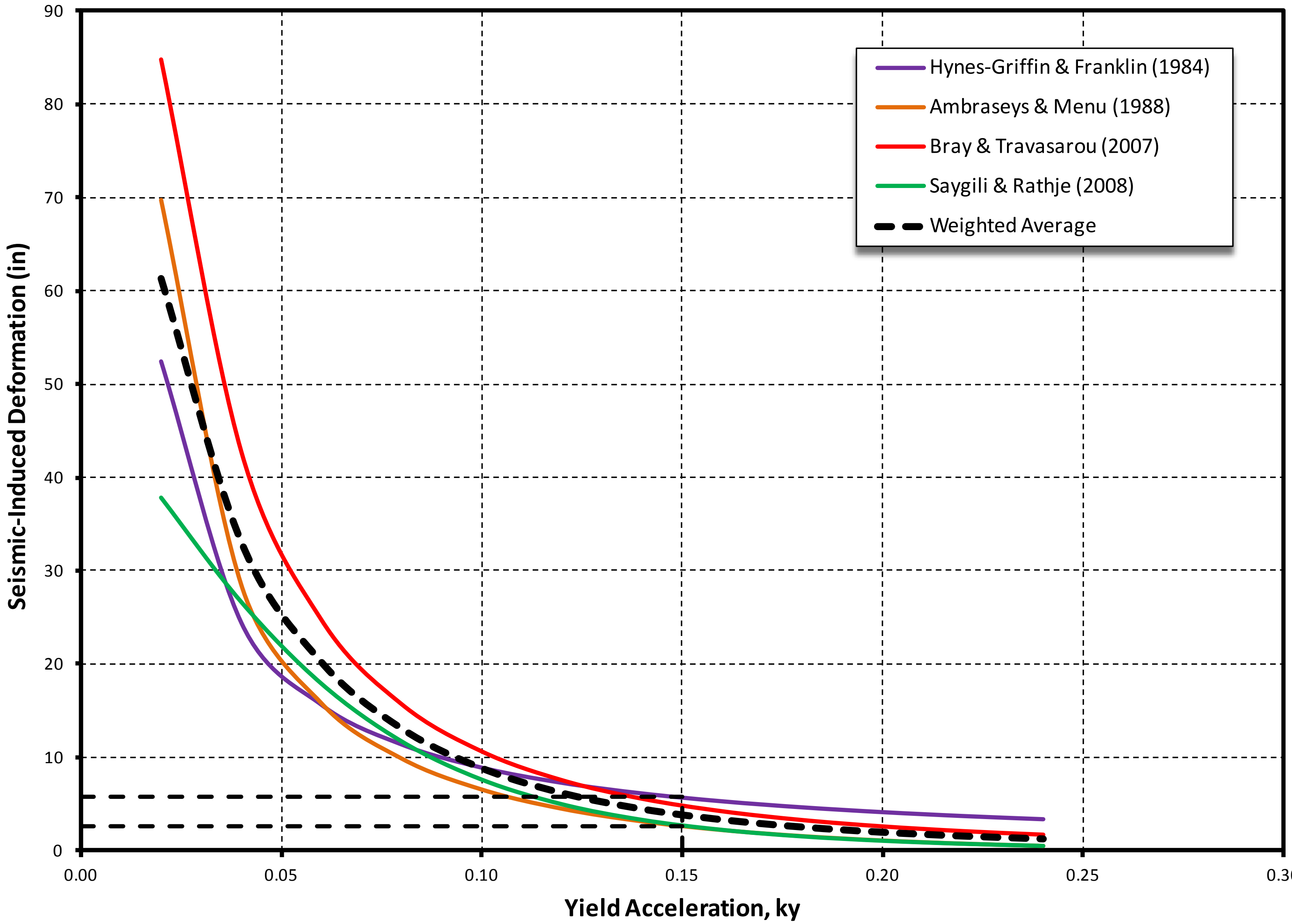

Figure 9.3-5. Estimated Deformation for MCE Event for Independent Design

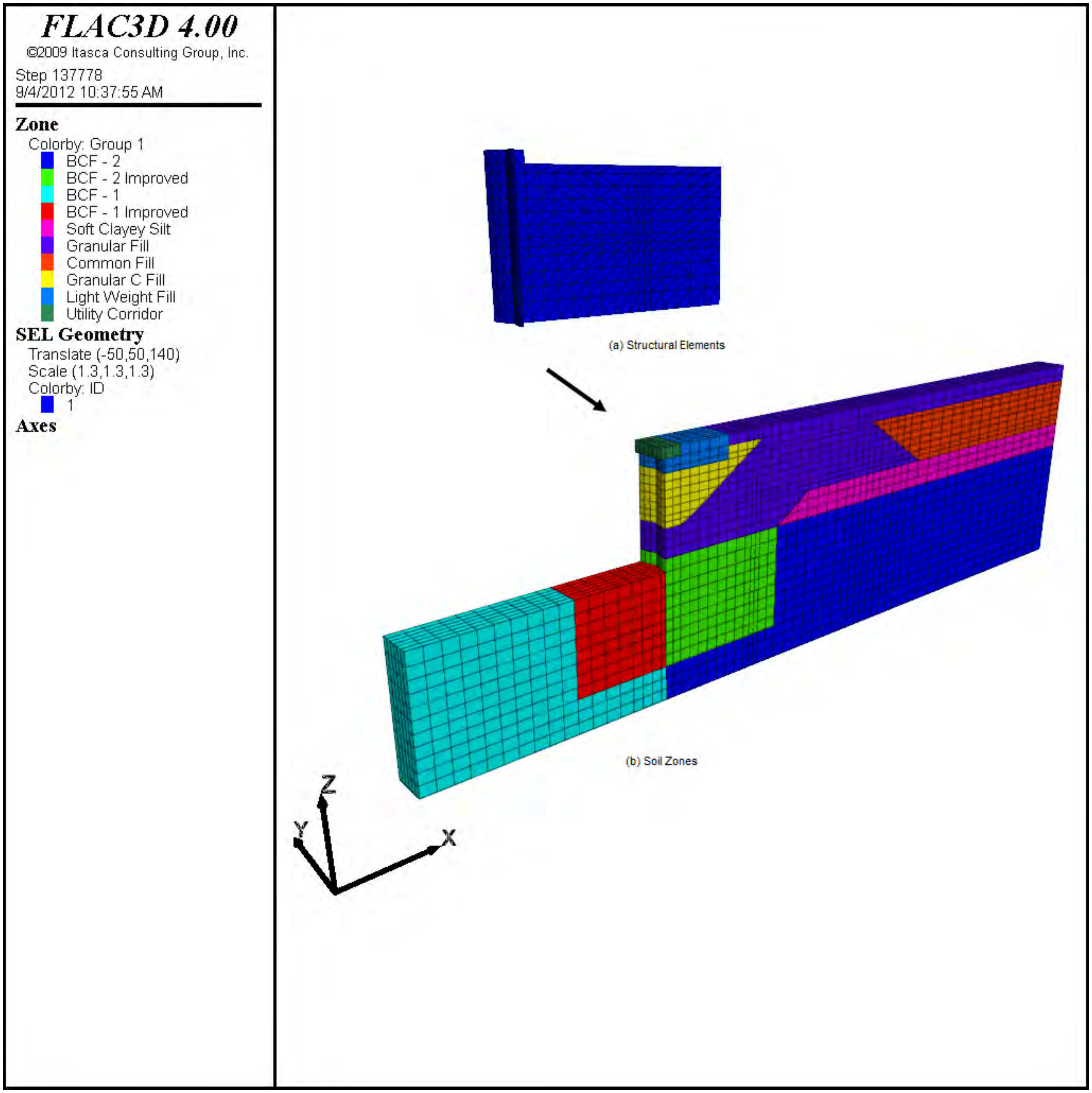

Figure 9.4-1. Independent Design FLAC^{3D} Analysis Grid

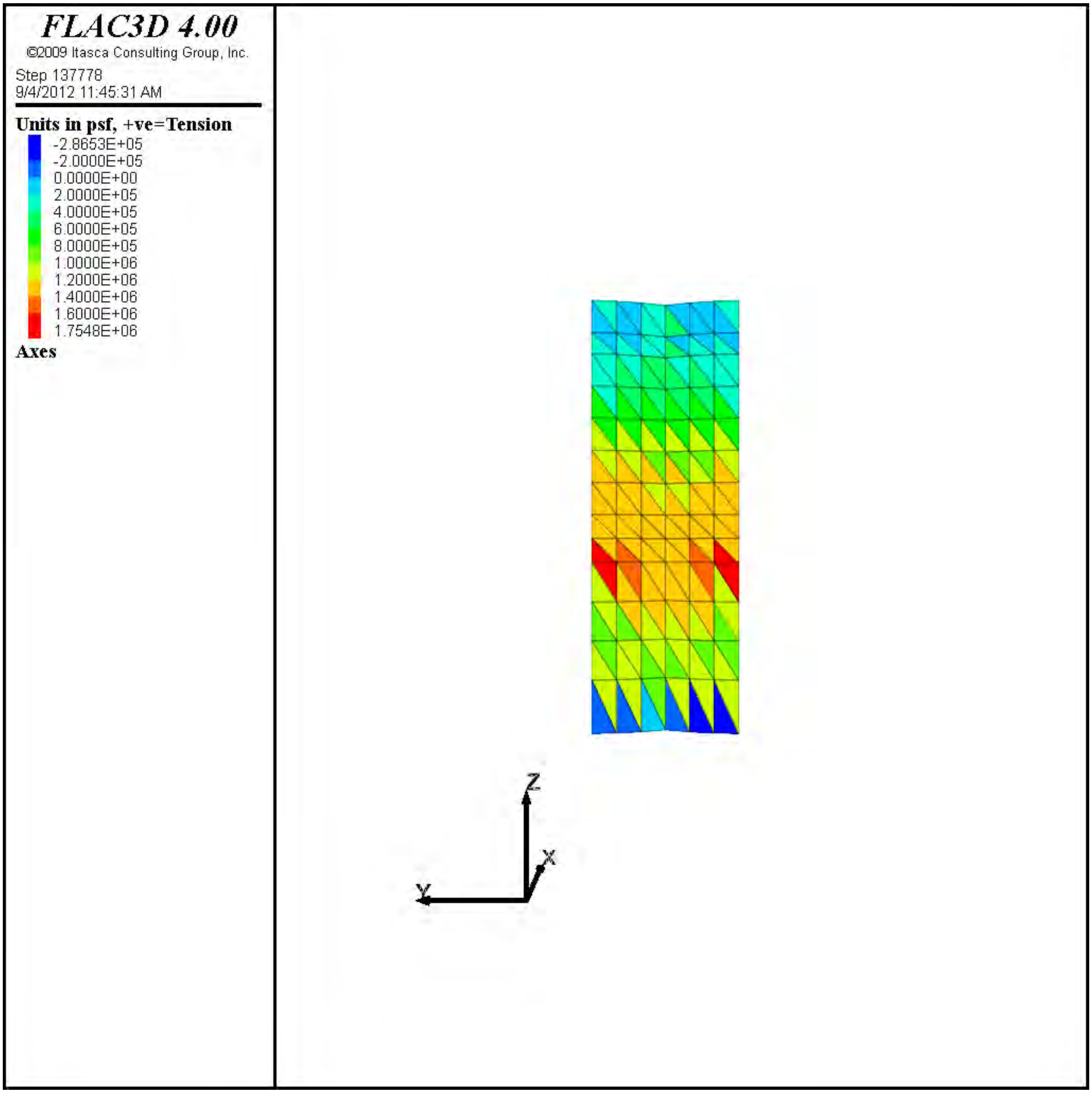

Figure 9.4-2. Facewall Membrane Stresses – Independent Design Static Short-Term

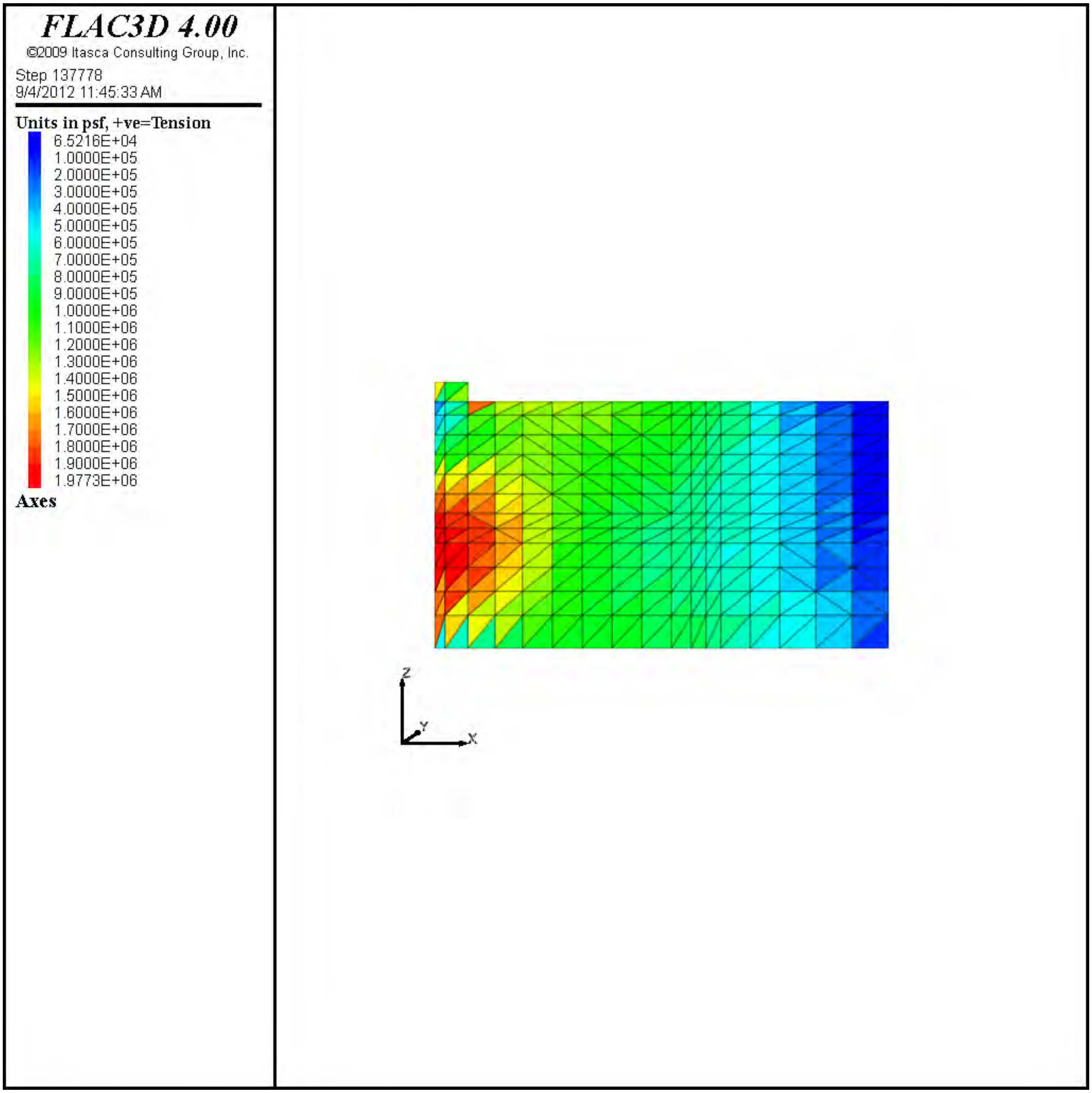

Figure 9.4.3. Tailwall Membrane Stresses – Independent Design Static Short-Term

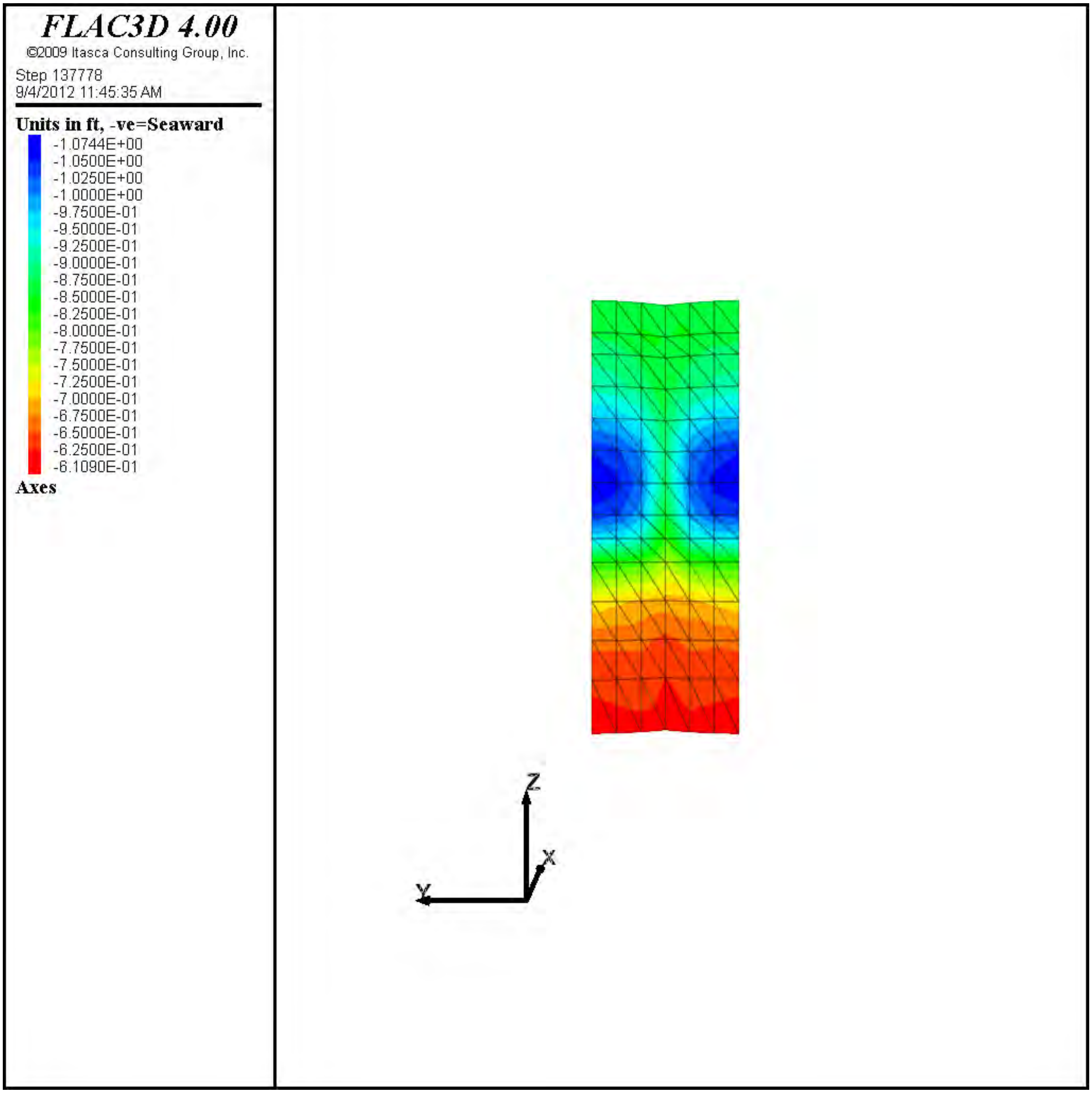

Figure 9.4-4. Facewall X-Displacement Contours – Independent Design Static Short-Term

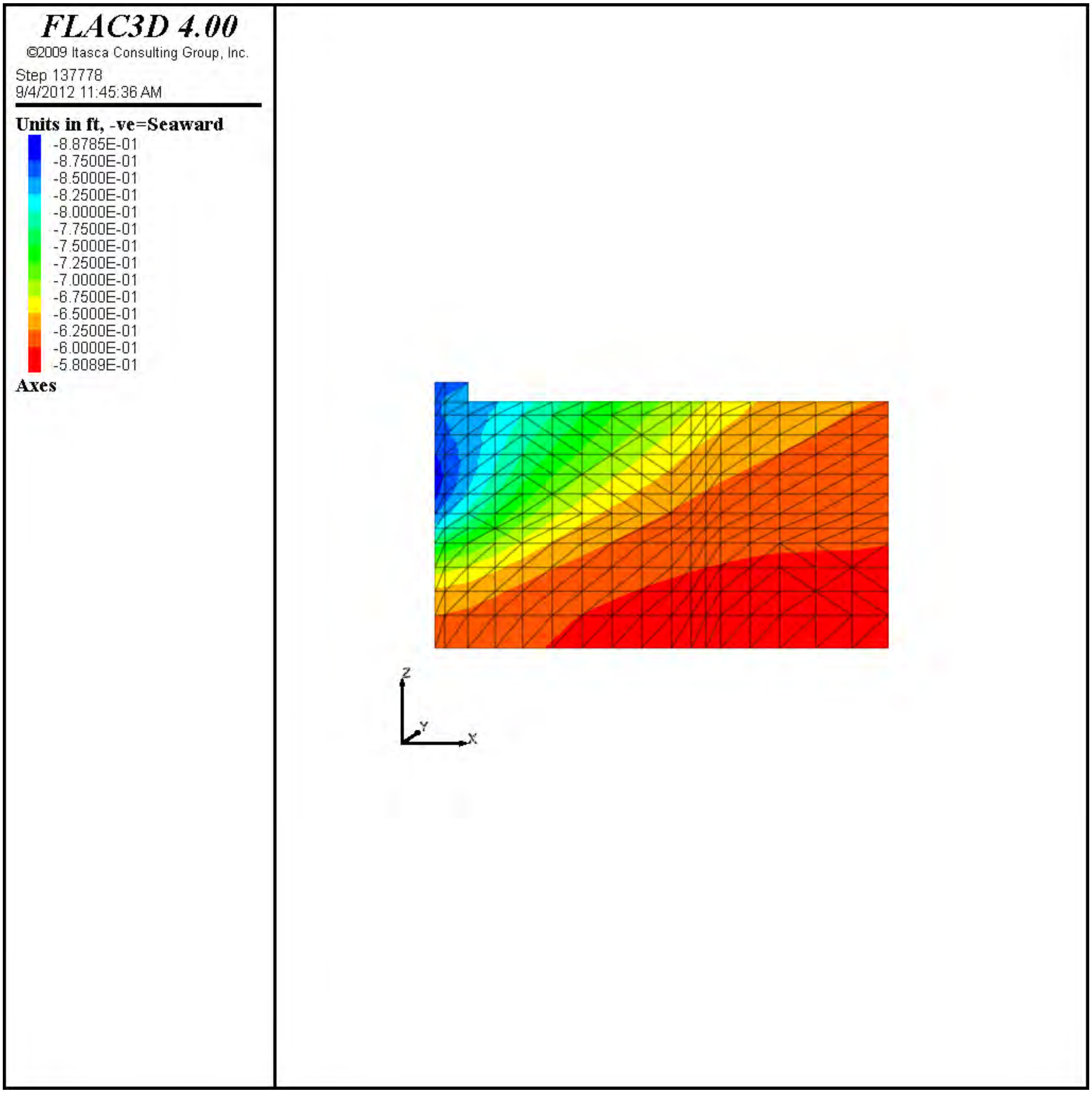

Figure 9.4-5. Tailwall X-Displacement Contours – Independent Design Static Short-Term

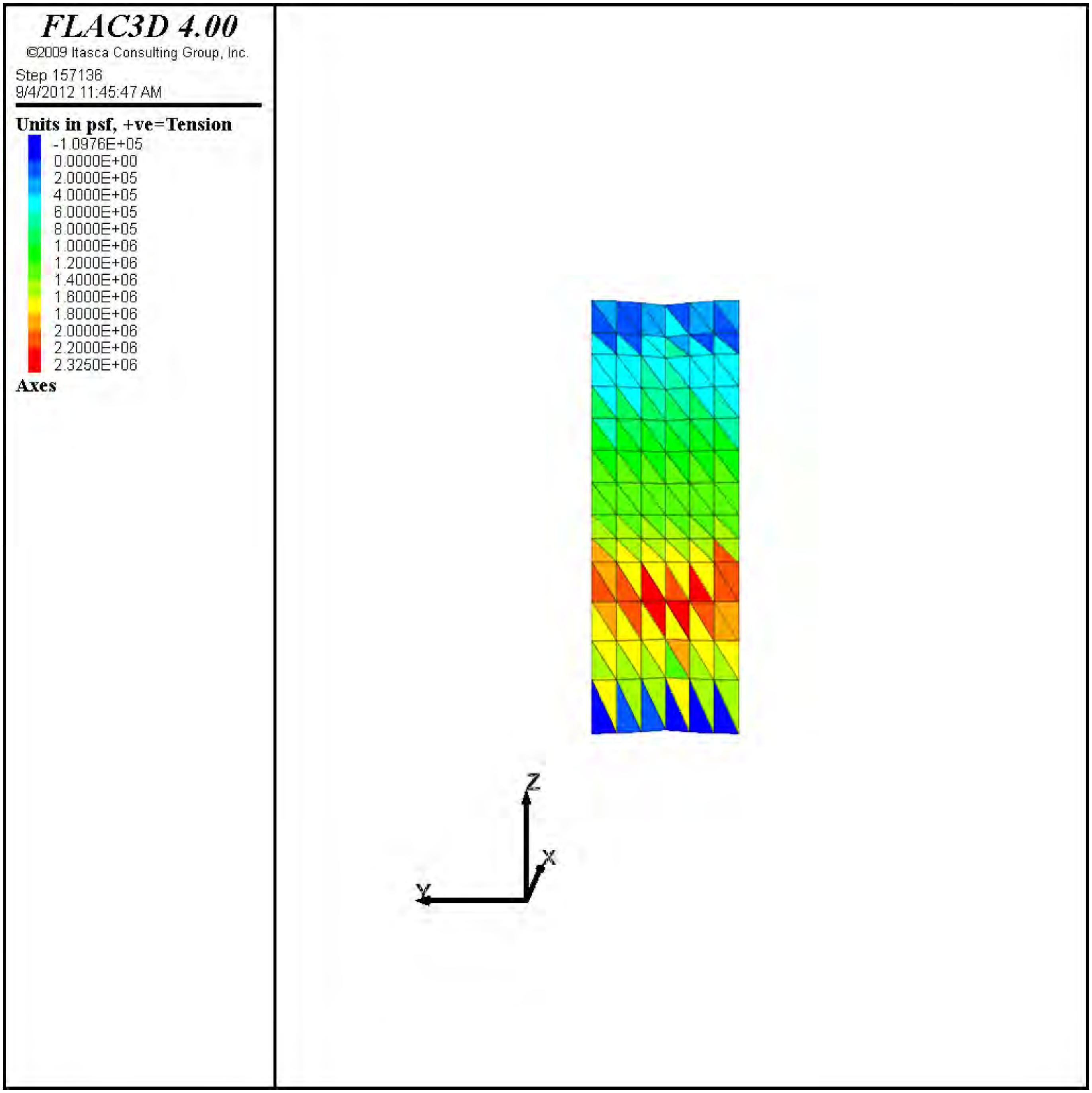

Figure 9.4-6. Facewall Membrane Stress – Independent Design Static Long-Term Drained

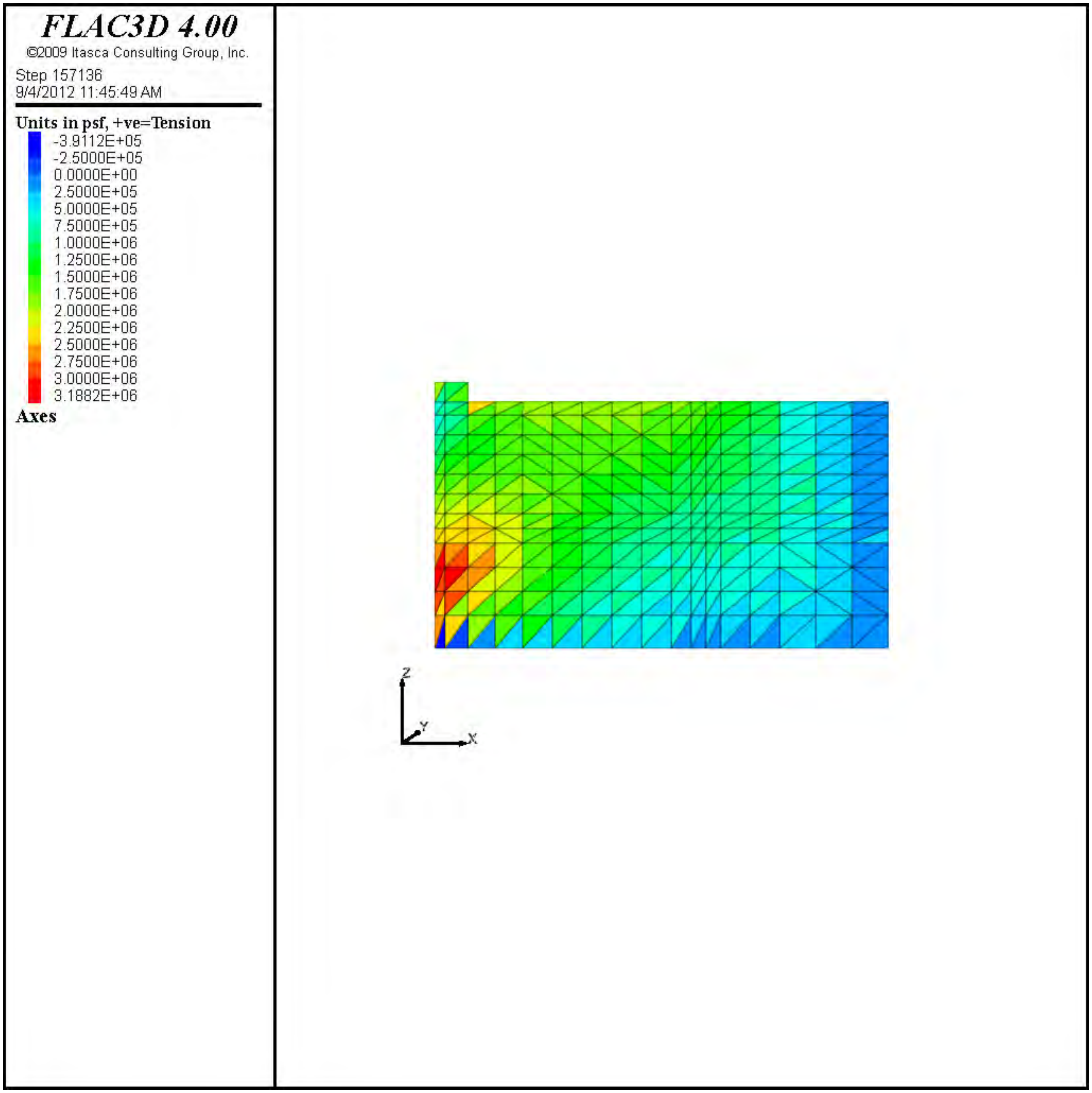

Figure 9.4-7. Tailwall Membrane Stress – Independent Design Static Long-Term Drained

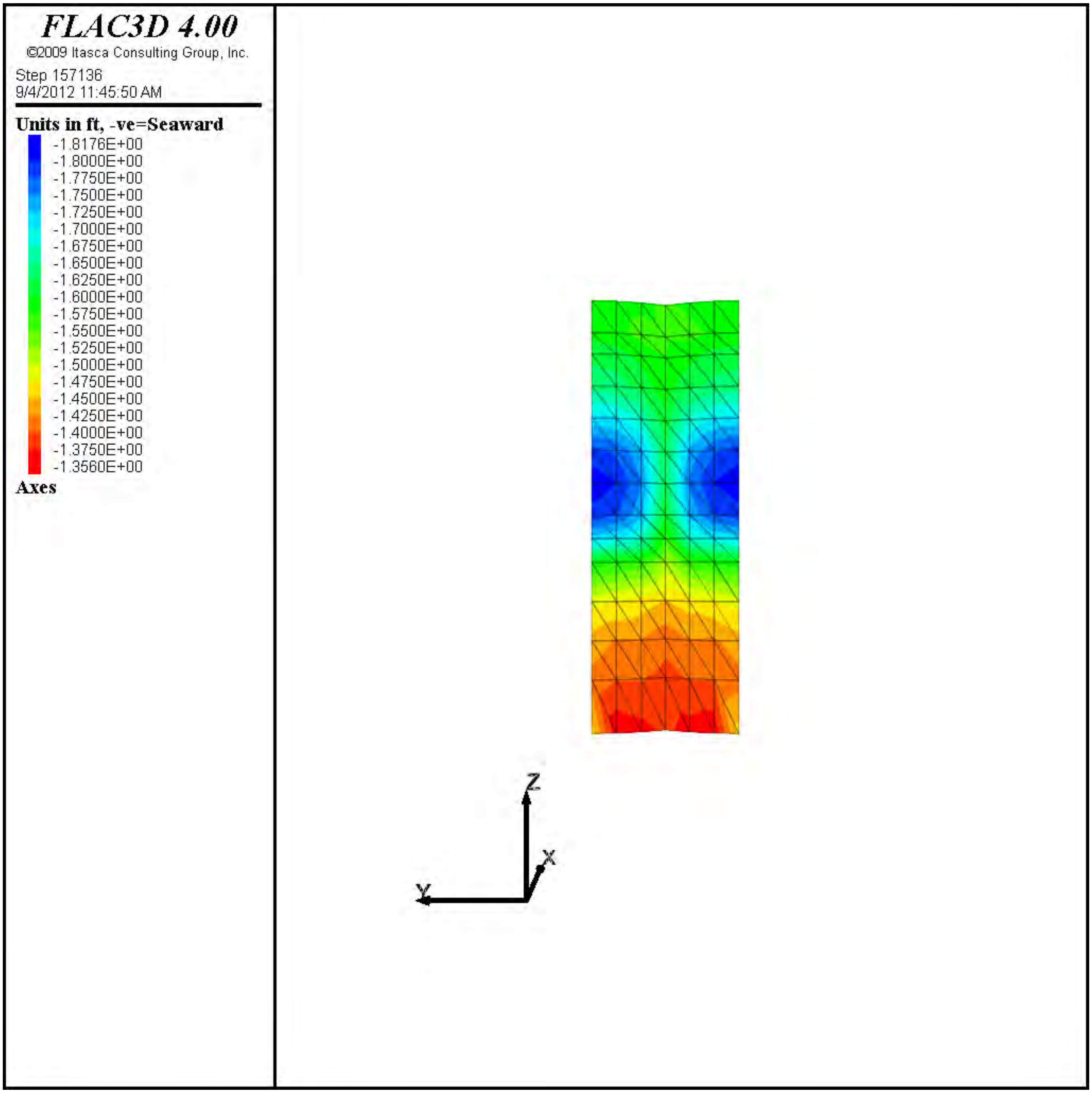

Figure 9.4-8. Facewall X-Displacement Contours – Independent Design Static Long-Term Drained

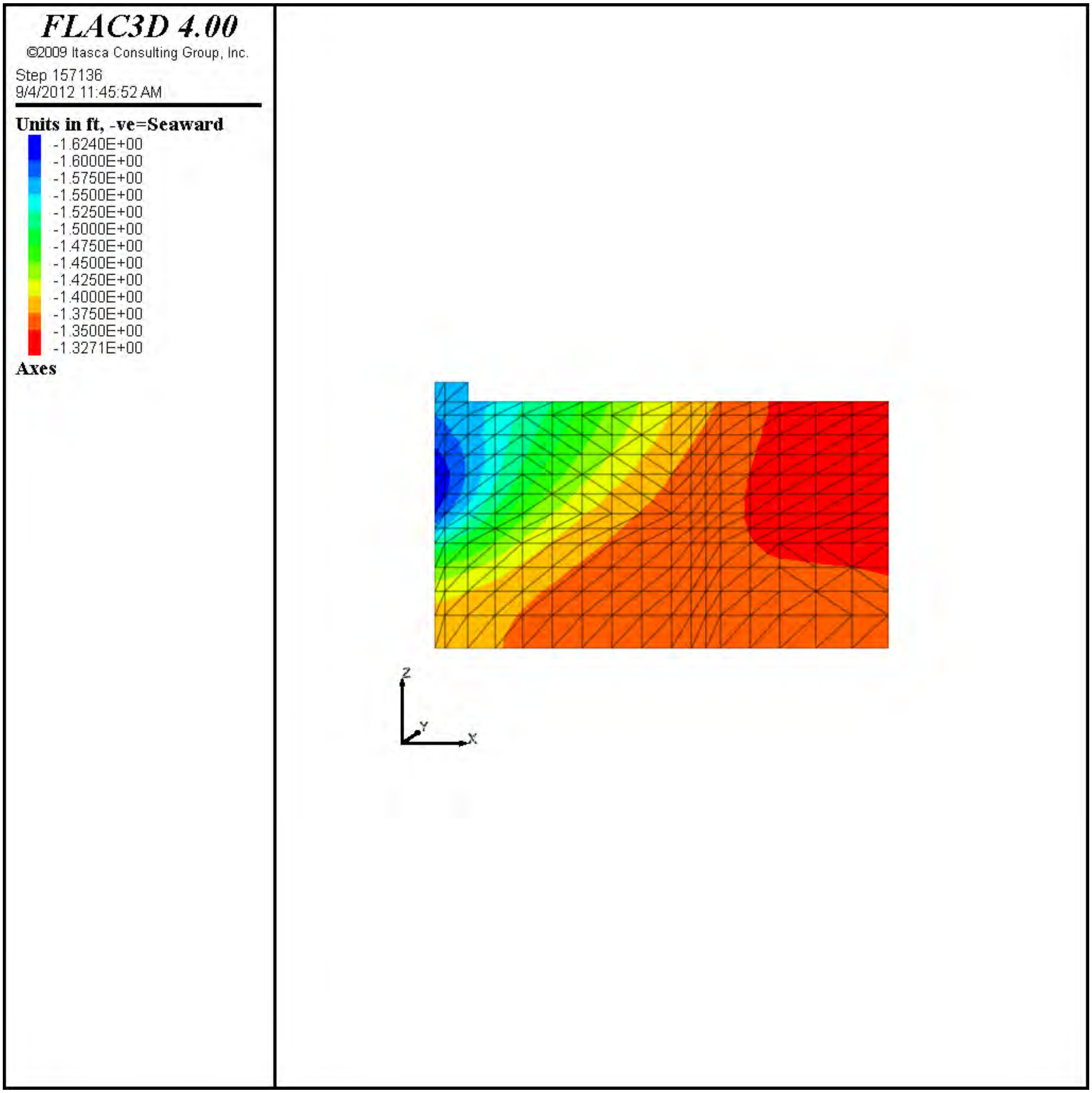

Figure 9.4-9. Tailwall X-Displacement Contours – Independent Design Static Long-Term Drained

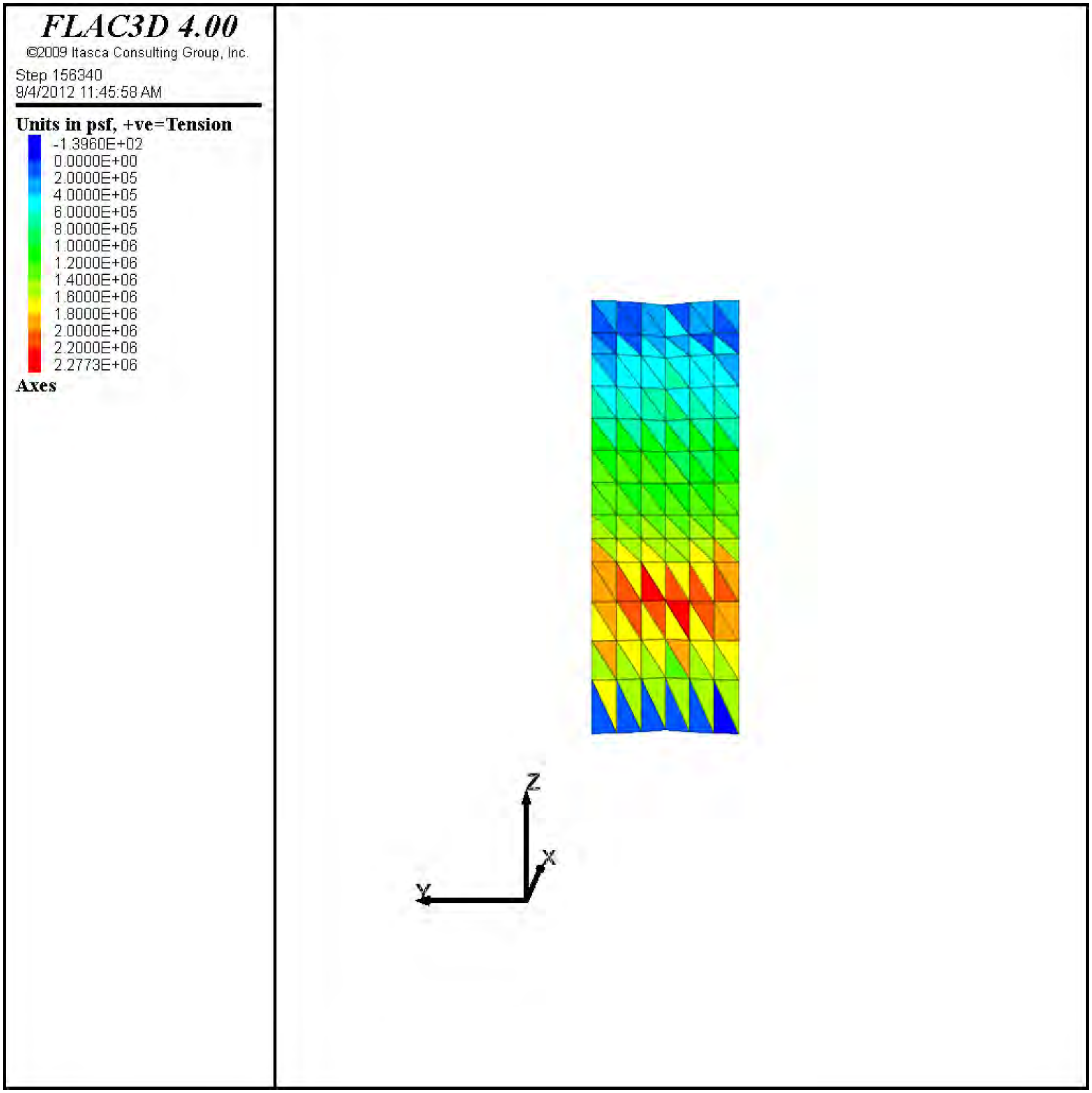

Figure 9.4-10. Facewall Membrane Stress – Independent Design Static Long-Term Undrained

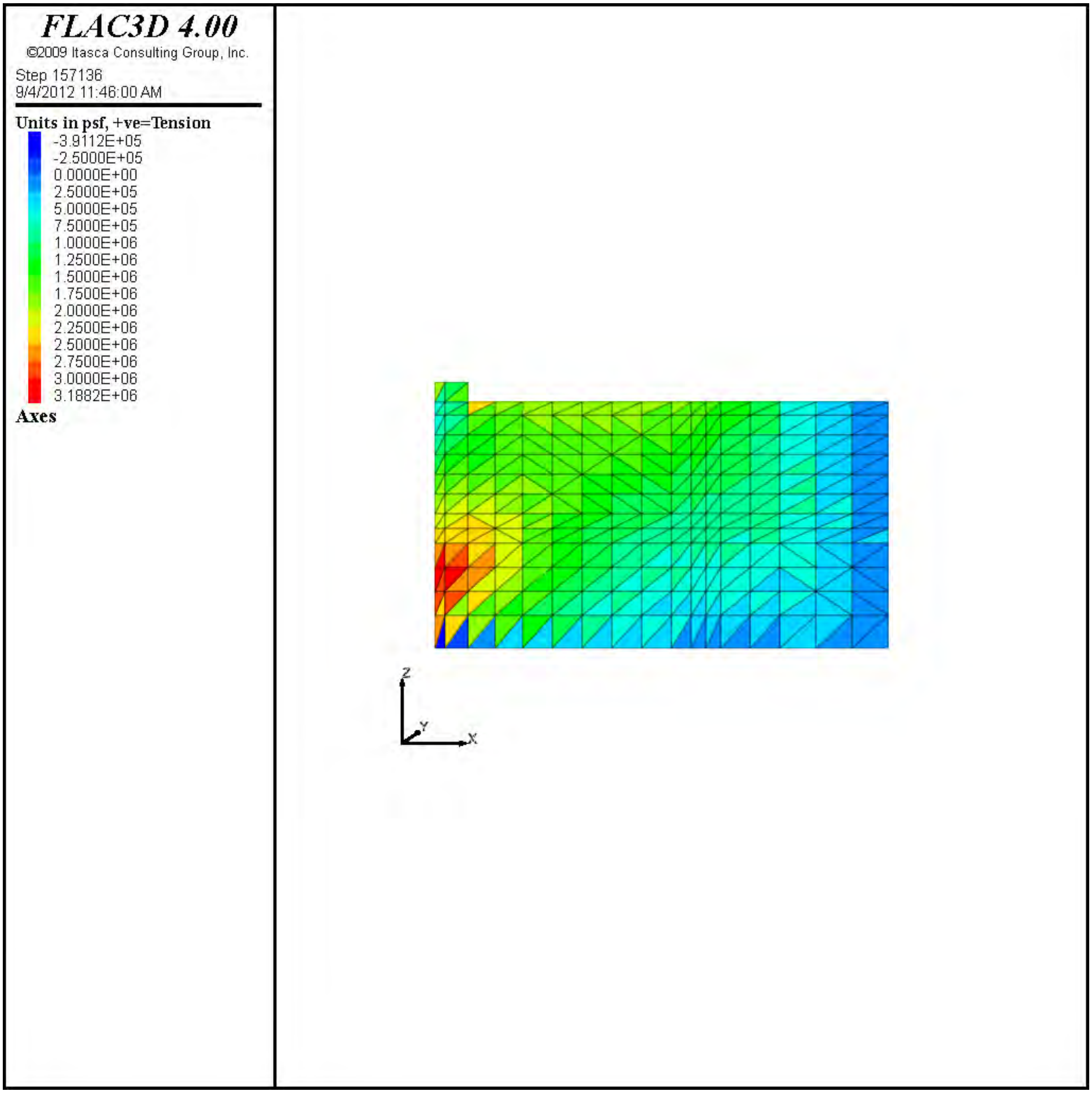

Figure 9.4-11. Tailwall Membrane Stress – Independent Design Static Long-Term Undrained

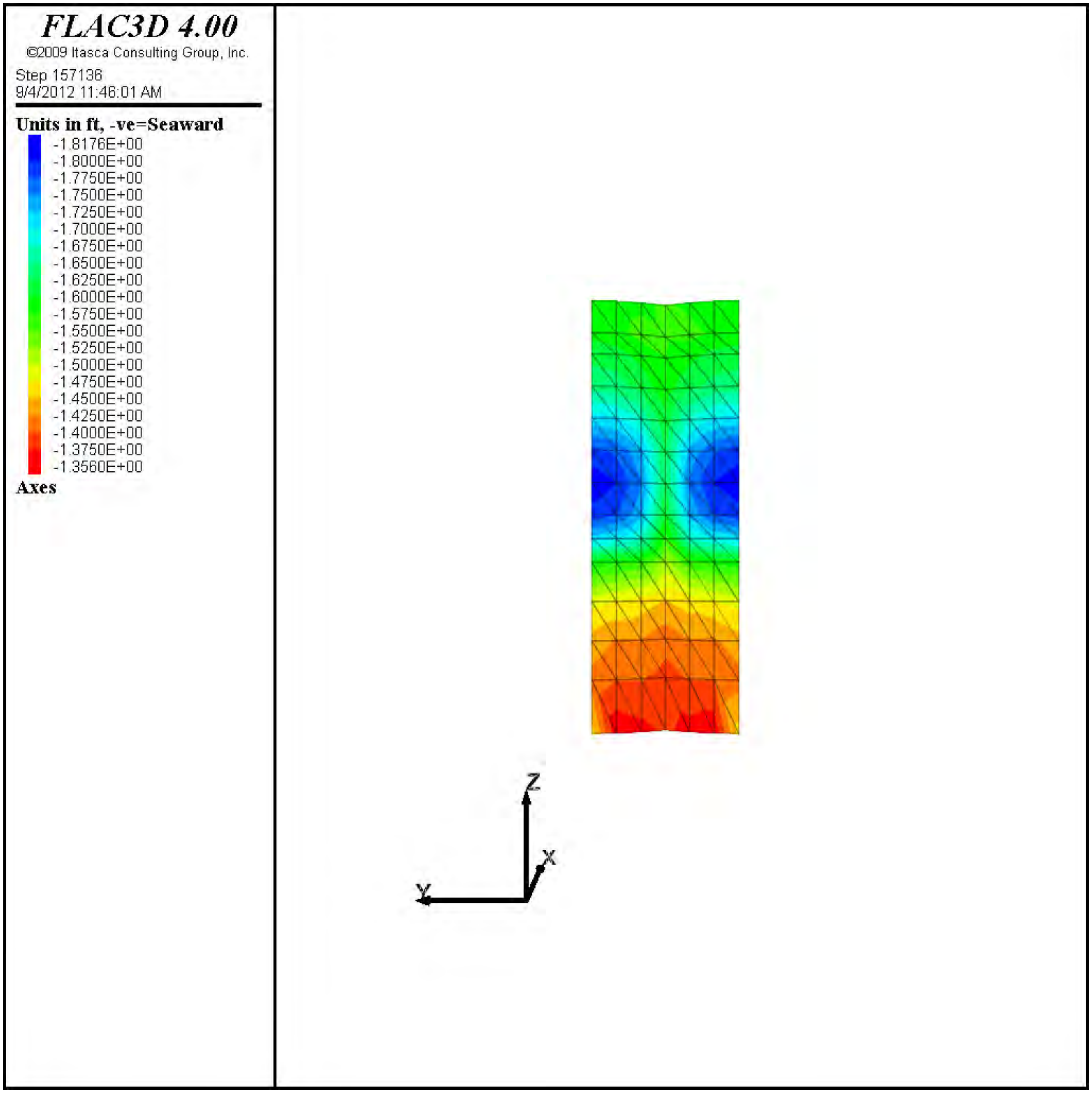

Figure 9.4-12. Facewall X-Displacement Contours – Independent Design Static Long-Term Undrained

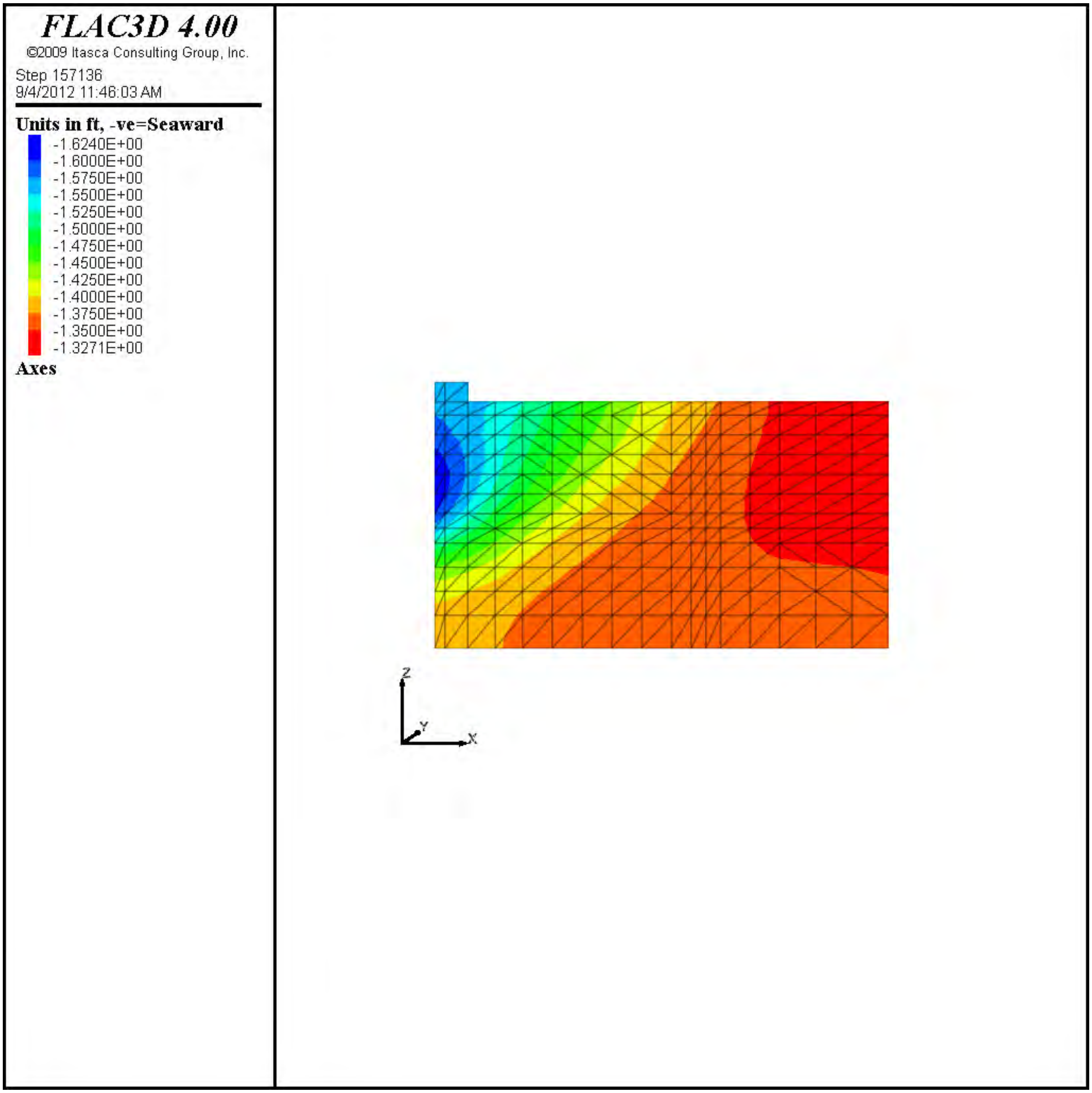

Figure 9.4-13. Tailwall X-Displacement Contours – Independent Design Static Long-Term Undrained

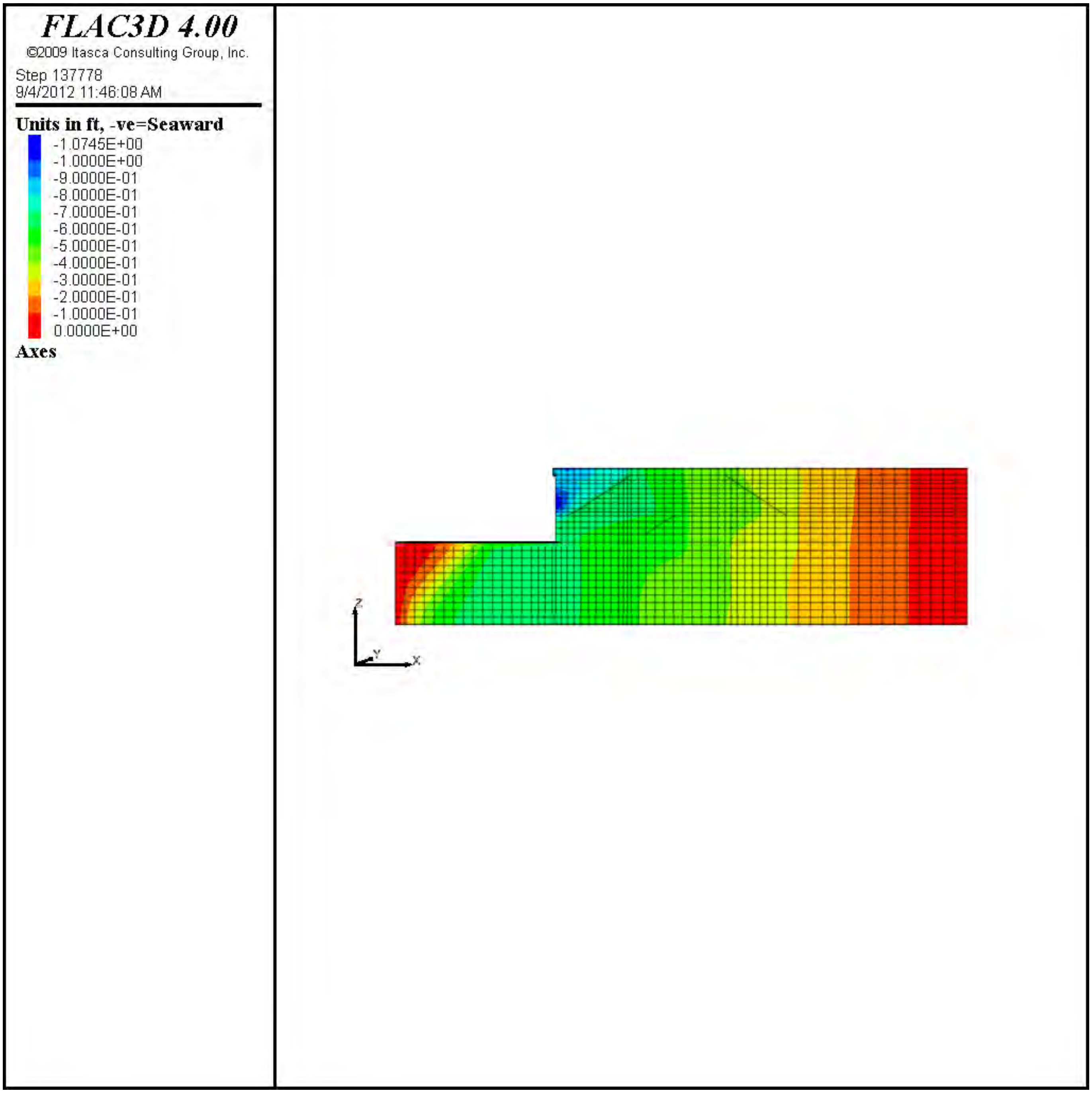

Figure 9.4-14. Soil X-Displacement Contours – Independent Design Static Short-Term

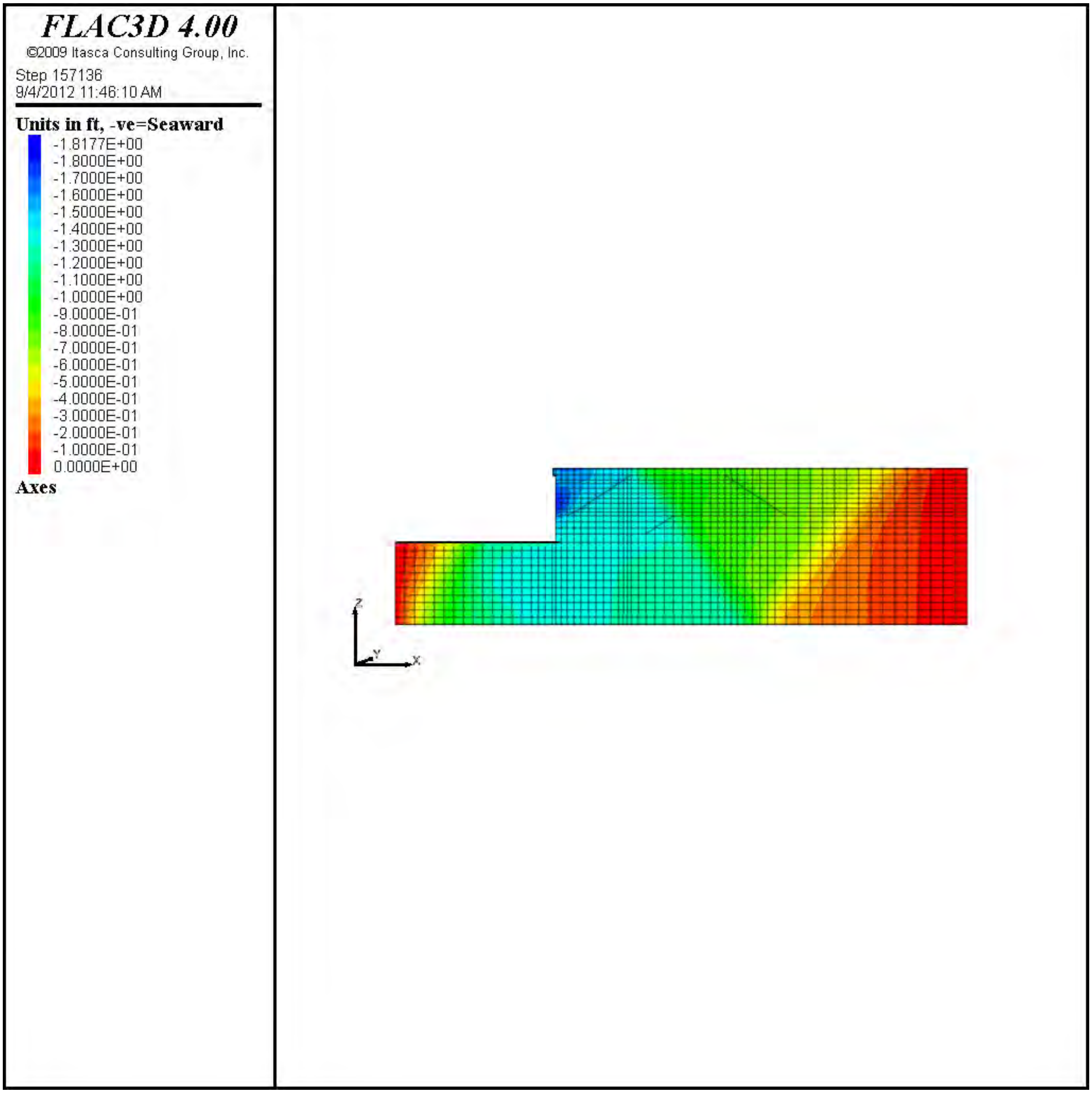

Figure 9.4-15. Soil X-Displacement Contours – Independent Design Static Long-Term Drained

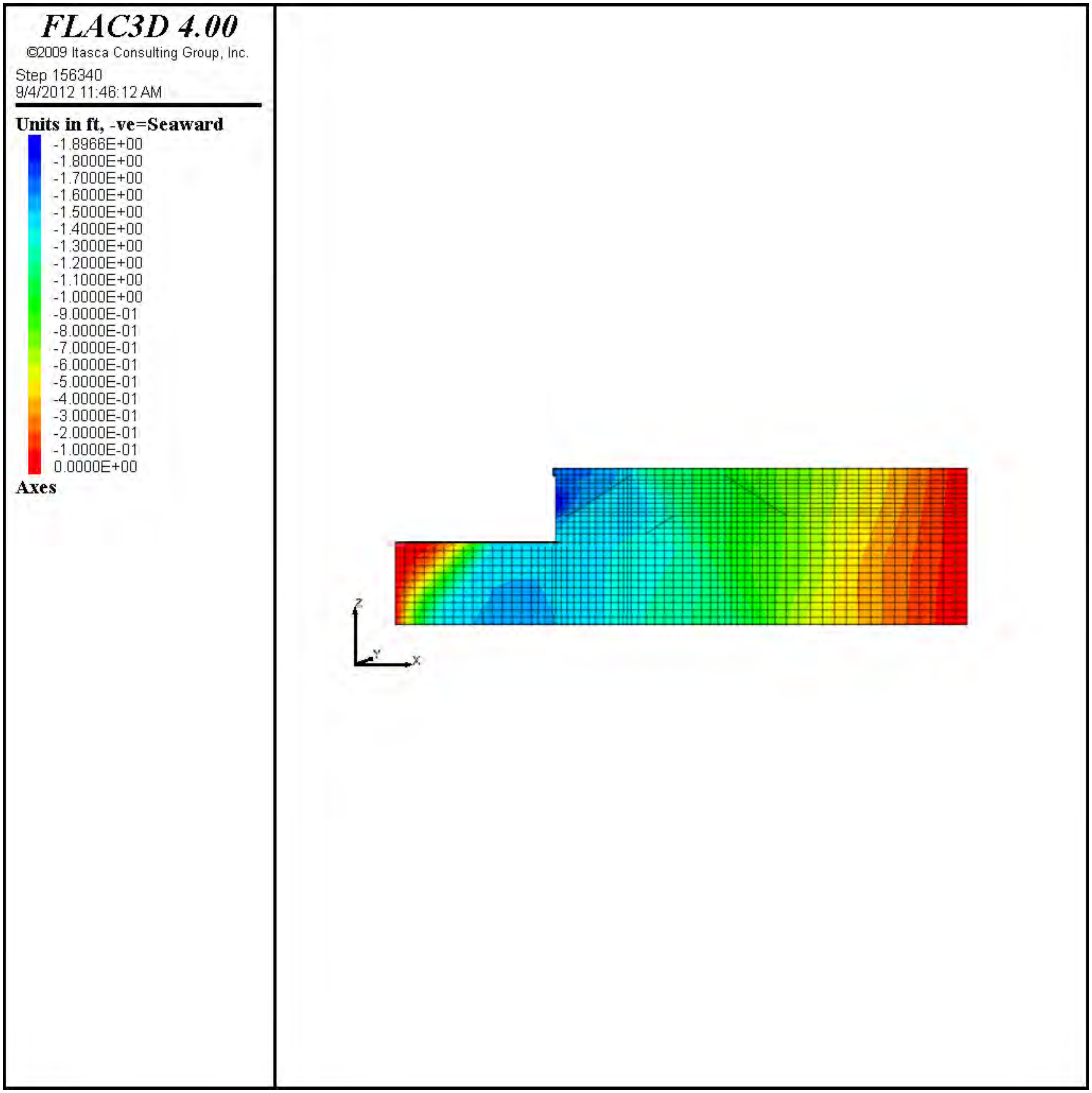

Figure 9.4-16. Soil X-Displacement Contours – Independent Design Static Long-Term Undrained

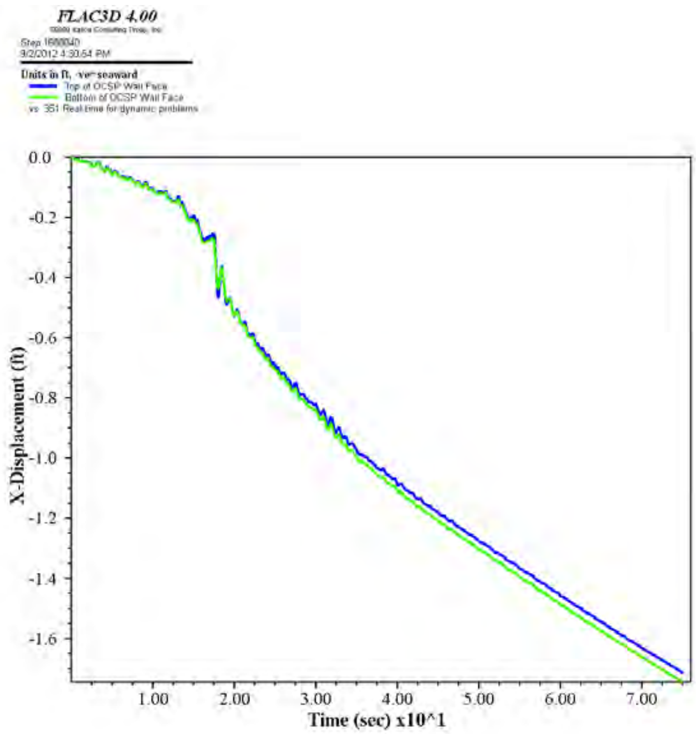

Figure 9.4-17. OLE Facewall X-Displacement-Time History – Independent Design (Michoacan Earthquake Record)

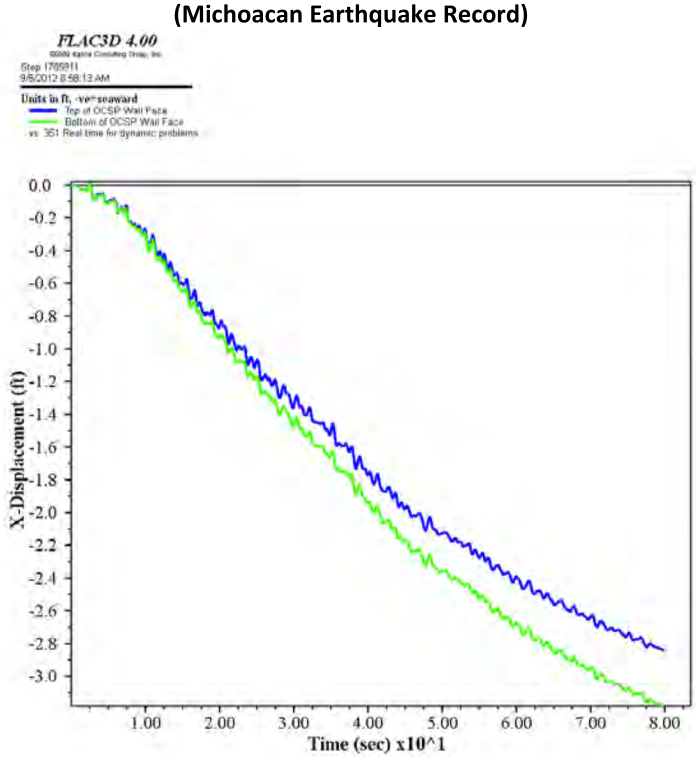

Figure 9.4-18. OLE Facewall X-Displacement-Time History – Independent Design (Puget Sound Earthquake Record)

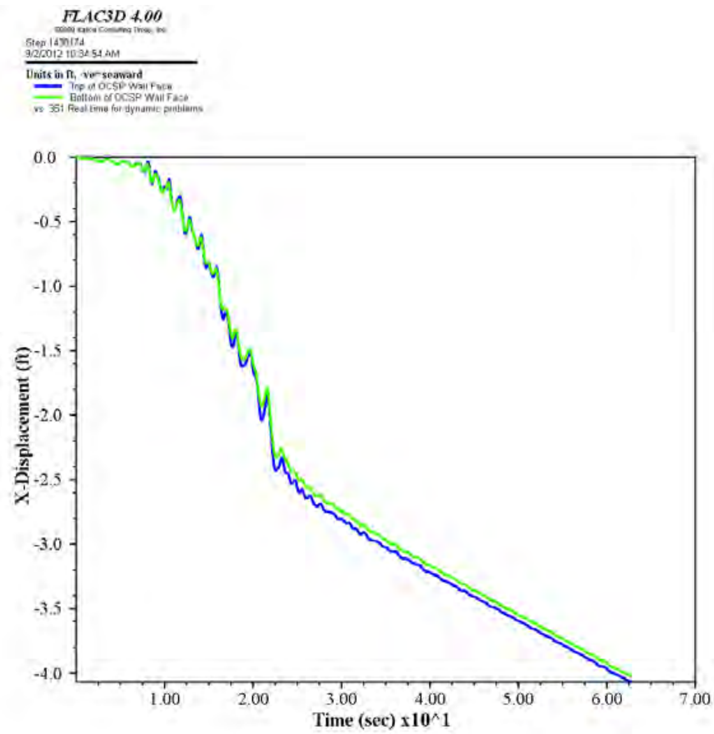

Figure 9.4-19. CLE Facewall X-Displacement-Time History – Independent Design (Michoacan Earthquake Record)

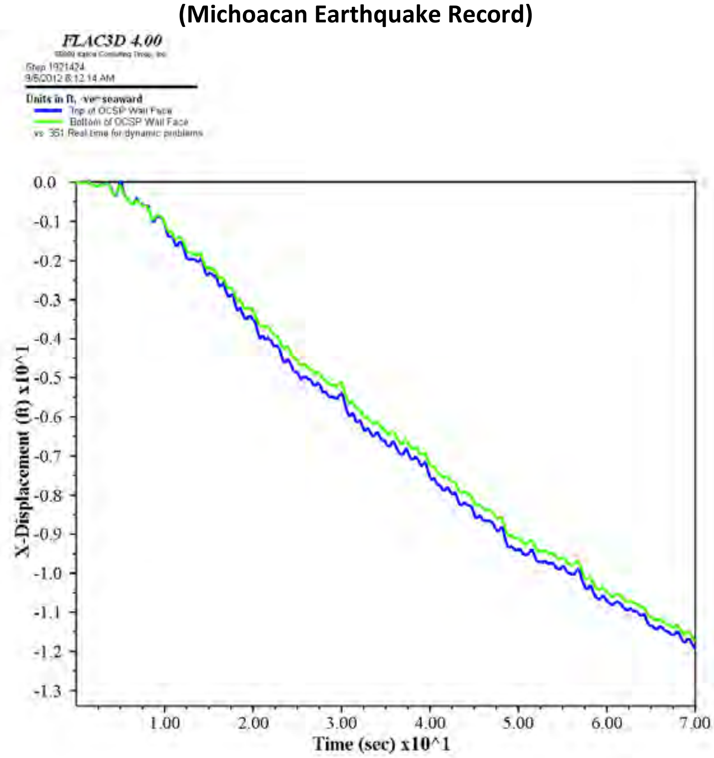

Figure 9.4-20. CLE Facewall X-Displacement-Time History – Independent Design (Western Washington Earthquake Record)

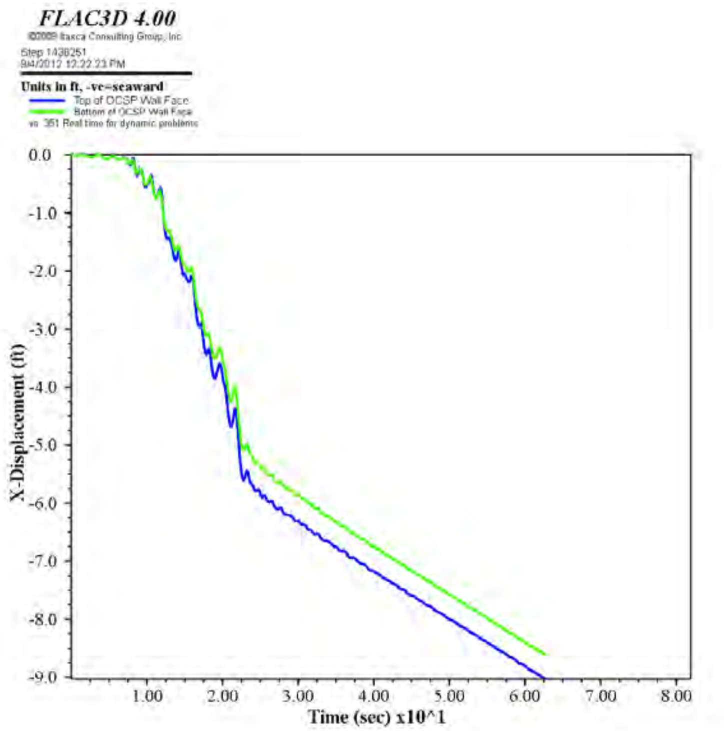

Figure 9.4-21. MCE Facewall X-Displacement-Time History – Independent Design (Michoacan Earthquake Record)

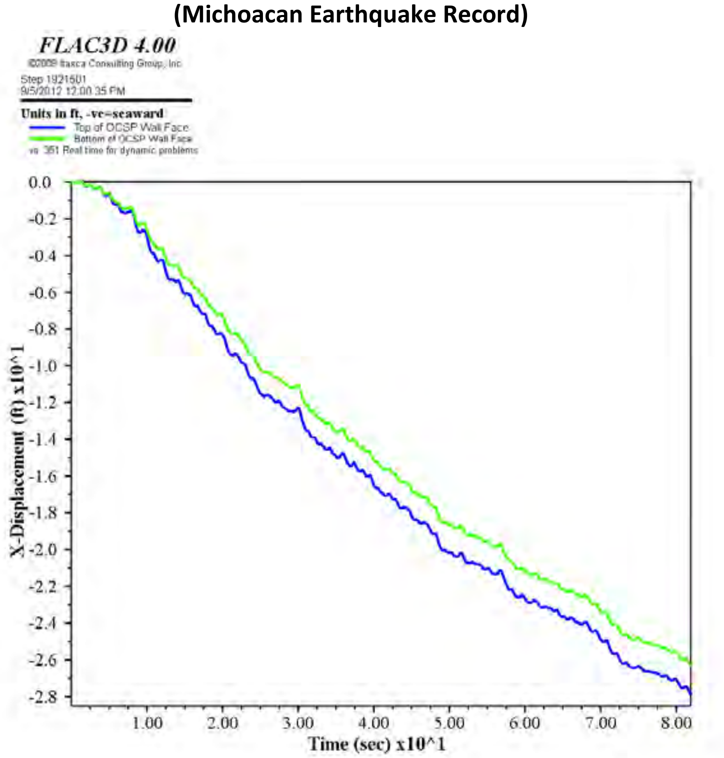

Figure 9.4-22. MCE Facewall X-Displacement-Time History – Independent Design (Western Washington Earthquake Record)

SECTION 10

Conclusions and Recommendations

For this suitability study, CH2M HILL reviewed project information for the PIEP and conducted engineering analyses to evaluate performance of the OCSP® system being constructed at the Port of Anchorage as part of the PIEP. The review and performance evaluation were performed to determine whether the basis of design for the OCSP® system is consistent with standard design requirements, whether the as-built OCSP® system will meet design and performance criteria for gravity and seismic loading, and whether alternative design or construction methods need to be considered to enhance the long-term performance of the planned PIEP. The engineering analyses included assessments of seismic hazards, hydrological considerations, geotechnical stability, structural performance, and project construction. These assessments were based on conventional design methods, numerical modeling of the soil-structure system, and evaluations of project constructability. An independent design was also identified and evaluated to determine whether simple changes could have been made to the OCSP® system to better meet the PIEP design criteria. The following sections summarize the conclusions and recommendations of the CH2M HILL suitability study.

10.1 Conclusions

The general conclusion from this suitability study is that the OCSP® system at the PIEP, as currently designed and constructed, is acceptable only in shallow water areas at the Dry Barge Berth. Where water depths increase to the south, the PIEP does not meet planned design and construction requirements. These deeper water areas include the Wet Barge Berth and the North Extension 1 and North Extension 2 berths. Of specific concern is that the suitability study shows the following:

- The OCSP® system is inadequate relative to global stability and seismic displacements based on the PIEP design criteria.
- There are significant construction deficiencies involving sheet piles being damaged and out of interlock. These deficiencies appear to have been caused by a combination of construction methods and site conditions, including relatively long sheet pile lengths, imposed load from an access dike, subsurface obstructions from rip rap, and unique soil conditions.

These general conclusions regarding current conditions of the PIEP were reached after performing a year-long evaluation of design and construction of the PIEP. Conclusions reached for each major work task are summarized below:

- **Definition of Design.** The basis of design used by the original designers is generally appropriate. However, documentation of the rationale and limitations of the criteria were not always presented. In the absence of a single design code that is applicable to the design and construction of an OCSP® system, the document *PIEP Design Criteria Summary* (MARAD and ICRC, 2012) was developed by ICRC in conjunction with MARAD, the POA, PND Engineers, and other consultants for the PIEP to describe the basis of design for the project. This document lists codes and criteria applicable to the design of the PIEP and in general serves as an adequate basis of design. Although CH2M HILL's review identified a number of other relevant design codes and criteria that could have been included in the *PIEP Design Criteria Summary*, these additional codes and criteria do not appear to have resulted in conditions or design decisions that led to significant deficiencies in design or construction.

  Additional conclusions from reviewing the basis of design are as follows:

  - *Groundwater and tidal elevations*: Groundwater elevation behind the face of the OCSP® system should be increased from elevation +18 feet to +20 feet MLLW for design, based on recent groundwater measurements. For seismic loading, the tidal elevation in front of the OCSP® system should be reduced from levels used by the original designers for seismic design (elevation +16.5 feet MLLW and +11.5 feet

MLLW) to elevation +7.5 feet MLLW to adequately address the daily fluctuation in tidal elevation at the POA. Further, a hydrodynamic component of load should be included in seismic design. The basis for these recommended changes is summarized in Section 2. These changes will result in more severe loading than what was used in the current design. The higher loads are believed to be more representative of long-term loading conditions that will occur at the POA.

- *Live load level and location*: The live load that represents container storage and other operational loading for the facility should be 1,000 psf and should extend from the bulkhead throughout the backlands, rather than using 200 psf within 200 feet of the bulkhead and 1,000 psf behind this location, as used by the original design team. Twenty percent of the live load (that is, 200 psf) should be used during seismic design evaluations. Current design assumes no contributions from live load during seismic events. The assumptions used in the original design regarding live load location and live loads during seismic design are somewhat unconservative relative to likely operations of the POA facility.
- *Over-dredge depths and scour*: A 6-foot over-dredge depth should also be included in all stability calculations. Potential scour beyond over-dredge values could further increase the water depth at the pierhead line. This condition does not require a change in the design criteria; however, it emphasizes the need for annual monitoring of scour at the face of the OCSP® bulkhead wall.
- *Global and internal factors of safety*: Factor of safety values for global and internal stability in the PIEP design criteria, as well as load factors for different load combinations, appears to be reasonable. One of the critical cases for global stability involves long-term undrained loading to account for the significant changes in tidal elevation at the POA. This case needs to be systematically evaluated for the project.
- *Seismic design and performance criteria*: The three-level seismic criteria involving an OLE, CLE, and MCE are reasonable; however, additional explanation on the rationale used to define deformation limits needs to be presented within the design criteria. As currently written, it is unclear which of these criteria are determined by structural requirements, by continued port operational requirements, or by performance of other services used to support the facility, such as underground utilities. Relative to performance expectations at other port facilities, the deformation limits for the CLE and MCE appear to be overly restrictive; however, operational constraints not currently defined in project documentation may justify current limits.
- *Other criteria*: Other criteria used by the original designers as a basis of design, including the service life and performance limits, appear to be reasonable at 50 years, although new codes such as the 2012 AASHTO LRFD Bridge Design Specifications are requiring 75 years for the design of new bridges.

- **Seismic Hazard Assessment.** The seismic hazard analysis performed by the original design team for the OCSP® system used the best available methods in 2008 for establishing firm-ground motions, as well as the effects of local geology on these motions. Firm-ground peak horizontal ground accelerations and response spectra were consistent with the then-current state-of-the-practice, and methods of evaluating effects of local geology on ground motion propagation from firm-ground levels to the ground surface were suitable for OCSP® system design. Independent ground-response analyses conducted as part of this suitability study also show that ground motions used by the original designers were reasonable, as discussed in Section 3 of this report.

  Additional conclusions from the seismic hazard review are as follows:

  - *Ground motion predictive equations*: There have been some changes in scientific thinking over the past 4 to 5 years regarding ground motion predictive equations (GMPEs), and these changes may result in changes to the firm-ground response spectra used as a basis for estimating firm-ground motions. These changes in the GMPEs will warrant further review in future seismic studies to determine if the design response spectra should be changed to account for updates to the GMPEs.
  - *Synthetic earthquake records*: Two synthetic earthquake time histories were developed by URS to represent the mega-thrust events in the absence of recordings for very large (M>9) earthquakes. Recent

earthquakes in Chile and Japan now provide records that are representative of large (M>8.8) seismic events. Once these records are processed and available to the public, they will likely provide a better basis of design than synthetic records. These natural records should be considered for any future work at the POA.

- *Local site effects:* The results of the local site analysis suggest that the ground motions developed by CH2M HILL are somewhat conservative, and therefore, may result in higher estimates of ground displacements. On the landward side of the wall, only small variations in ground motions were observed between the original designers and those by CH2M HILL at the input level (that is, elevation -150 feet MLLW) for the FLAC^{3D} analyses. This suggests that the FLAC^{3D} analyses for the OCSP® would not be affected significantly, regardless of which results are used. Somewhat higher differences were observed at the level used for the conventional stability analyses (that is, elevation -50 feet to -100 feet MLLW). These differences could result in lower factors of safety (FS) and higher displacements in the limit-equilibrium stability analyses. However, the amount of variation is within the limits of uncertainty in seismic response analyses, and therefore, the motions are considered appropriate for making design evaluations.
- *Gravel backfill characterization*: Evaluations of local site effects using ground response modeling methods were based on estimated stiffness of the backfill material. Now that backfill is in place and has been densified by vibracompaction methods, shear wave velocity measurements should be made in the backfill to obtain a better basis for ground response estimations. These measurements should also establish the velocity variation between the backfill and the underlying estuarine and BCF clay deposits following consolidation from the backfill material. In the absence of these measurements, a range of shear wave velocity values was assumed to characterize the local site effects.
- *Seismic hazard analysis*: Hazard evaluations that depend on the magnitude of ground shaking, such as liquefaction and seismic-induced slope displacement, considered source magnitudes greater than the magnitudes obtained from deaggregation of the uniform seismic hazard. Although the modal magnitude from the deaggregation of the uniform hazard correctly identifies the most likely magnitude, the potential for a very large earthquake magnitude—similar to the magnitude 9.2 event occurring in the 1964 Alaska earthquake—also must be considered. For the POA area, the larger magnitude could affect liquefaction potential and seismic deformation estimates.

- **Hydrological Analyses.** Hydrological information for the PIEP was developed by the U.S. Army Engineer Research and Development Center (ERDC). This information included field investigations and a numerical modeling study to evaluate sedimentation and dredging requirements associated with the expansion and deepening of the POA. The potential for ice loading to the OCSP® facewall system and moored vessels was also evaluated by the project designers as part of their design effort. An independent assessment of tidal conditions, scour depths, and ice loading from these past studies was conducted as part of this suitability study. This assessment resulted in some recommended changes in the basis of design, and it included comments on future maintenance requirements, as discussed in Section 2. These changes in the basis of design do not represent a fatal flaw to design plans.

  Additional conclusions from the hydrological review are as follows:

  - *Sedimentation rate*: The location of the OCSP® system in Knik Arm could result in accelerated sedimentation in some locations during the summer months. The POA and USACE will need to consider this potential when planning future maintenance dredging operations
  - *Localized scour*: A potential exists for localized scour beyond the values used as a basis of design. The location of this scour could occur at the base of fender piles and could be on the order of 5 to 7 feet. Propeller wash could also increase scour depths in localized areas. The potential for localized scour has important implications on global stability of the OCSP® system. For this reason this potential needs to be monitored as part of future maintenance operations, and where needed mitigated through use of scour protection systems.

- *Ice loading*: Ice loading is an important design consideration, as ice loads result in localized impact loads to the face of the OCSP® system, and they result in increased mooring forces to vessels. A design compressive strength of 300 psi in combination with an ice sheet thickness of 24 inches is considered appropriate for design. Ice loading on the moored vessels will be significantly greater than at the existing berths. Additional mooring lines will be required during heavy ice floes. Moving ice could also result in some abrasion of fender piles and the facewall; however, based on past observations this does not appear to be a significant issue for design.

- **Geotechnical Engineering Analyses.** Geotechnical analyses were conducted to evaluate the suitability of the as-built OCSP® system based on conventional design methods. These conventional design methods involved limit-equilibrium evaluations of stability and earth pressures, as well as an assessment of the settlement potential resulting from the addition of over 50 feet of gravel backfill on the landward side of the OCSP® facewall. Sensitivity studies were conducted to assess the effects of parametric variation, such as earthquake shaking level and tidal elevation, on OCSP® system performance. Details of these analyses are included in Section 5 of this report.

  Key conclusions from the geotechnical engineering analyses are as follows:

  - *BCF clay strength*: An extensive amount of information has been collected by the original designers to characterize geotechnical conditions at POA. In general, this information provides a suitable basis for performing geotechnical design evaluations for the OCSP® system. The main uncertainty with the characterization information appears to be the strength characteristics of the BCF clay. This uncertainty results in part from methods used to conduct strength tests on the BCF clay and in part from the potentially unique characteristics of this soil deposit. Additional static and cyclic strength tests were conducted on high-quality samples of BCF clay during this suitability study to address these uncertainties. These tests included special constant-volume, ring shear tests to evaluate changes in strength that occur during large displacements, similar to what could occur during a seismic event. Results of the constant-volume ring shear tests show that under large displacements, BCF clay from the POA site behaves similar to constant-volume ring shear tests on soils from Fourth Avenue—the same soil that underwent large movement during the 1964 Alaska earthquake—and soils at Port MacKenzie.

  - *Backfill settlement*: Settlement will occur in the future as the BCF clay responds to backfill loads. This settlement is expected to be within levels identified by the original design team. During construction, provisions should be made for potential additional settlement after the facility is operational.

  - *Internal stability of OCSP® wall*: Internal stability requirements for tailwall pullout and interlock tension appear to be satisfied. Although localized liquefaction within the backfill could occur during seismic loading, and this occurrence would reduce the frictional resistance at the OCSP® wall, the change in the FS appears to be acceptable for this loading case. FS values within the interlocks also appear to be acceptable, even though they are less than 2.0 for seismic loading.

  - *Global stability of the OCSP® wall*: The global stability of the OCSP® system was evaluated for gravity and seismic loads. Results of these analyses determined that for non-seismic cases, FS values for short-term undrained loading at the end of construction and long-term undrained loading during a combination of operations and very low tidal elevations are lower than the target values in the PIEP design criteria. Likewise, factors of safety for seismic loading were lower than defined in the PIEP design criteria. These low factors of safety potentially represent an unacceptable condition. The original design team found acceptable FS values when they considered these cases. The differences in FS values between those given by the original designers and CH2M HILL's appear to be mainly related to the geometry of the critical slip plane, the interpretations of strength data for BCF clay, and a number of small changes in criteria that combine to make more critical conditions.

  - *Seismic displacements*: Estimated displacements from simplified seismic displacement analyses exceeded the recommended performance criteria for seismic loading under the CLE and MCE. If the strength of BCF clay does not decrease with cyclic loading, exceedances do not represent a life safety issue, although they could lead to extensive repair requirements. This performance observation was based on more severe assumptions by CH2M HILL regarding groundwater elevation, tidal height, and live-load application during seismic events (compared with original design criteria). If the BCF clay at POA undergoes large strength reduction similar to that observed at Fourth Avenue during the 1964 Alaska earthquake and similar to the results of constant-volume ring shear tests conducted during this suitability study, the displacements during the CLE and MCE could exceed 10 feet. This behavior would be consistent with what was observed in the 1964 Alaska earthquake at Government Hill, Fourth Avenue, L Street, and Turnagain Heights.

- **Structural Analyses.** Independent structural analyses based on conventional methods were conducted for the baseline as-built condition. These analyses were without consideration for construction deviations. The results show that the FS values for cell internal stability (interlock strength and tailwall pullout) are satisfactory for both static and seismic load cases. These results are discussed in Section 6 of this report.

  Additional conclusions from the structural evaluation are as follows:

  - *Damage assessment*: Damage occurred during OCSP® sheet pile installation. This damage resulted in sheets being out of interlock and sheets with inadequate penetration. This damage affects the stability of some sections of the OCSP® system, and it results in zones of weakness that could lead to extensive damage under additional dead and live load, and during a large seismic event. These failures could be particularly significant if they occur in a facewall sheet or at the forward segment of the tailwall near the wye as a stress concentration could lead to unzipping and subsequent load transfer to other cells.
  - *Corrosion of sheet piles*: Life-cycle performance relies on the OCSP® corrosion protection system. This system includes galvanizing and an impressed current cathodic protection system. Although protected, corrosion will still occur with time, and this corrosion will result in loss of structural capacity. Estimates of the reduction in wall thickness after 50 years suggest reduction of structure thickness. FS values for tension of the sheets at the highest wall sections of the North Extension could result in reduction of the FS to 15 percent under the design criteria near the end of the 50-year design life. Where the wall heights are lower, such as the Dry Barge Berth, the FS is adequate at 50 years. Regular inspection and maintenance will be essential for maximizing service life. Accelerated low water corrosion (ALWC) is present at the existing POA facilities, and it could further reduce the FS in localized areas if not controlled by cathodic protection.

- **Numerical Modeling.** Advanced soil-structure interaction analyses were conducted using the three-dimensional computer program, FLAC^{3D}. These analyses were conducted for the highest section of OSCP® wall system. This approach to modeling was used to obtain a better understanding of the overall global stability and seismic displacements of the OSCP® system, as well as the interaction between the face of the OSCP® walls and the tailwalls when subjected to gravity and seismic loads. By using the FLAC^{3D} method of analysis, the combined effects of inertial loads and stiffness of the soil-structure system could be better approximated. Details for the FLAC^{3D} analyses, including the evaluation of local defects, are provided in Section 7.

  Key conclusions from the numerical modeling work are as follows:

  - *As-designed model response*: Results of static analyses completed for the as-designed case were generally consistent with results obtained by the original designers. This similarity is based on estimated stresses in the facewall and tailwall and estimated displacement during gravity loading. These results showed that displacements and wall stresses were within allowable levels identified by the original designers when the model used boundary conditions consistent with the original design. For seismic loading, the displacement results of numerical modeling were larger than those estimated by the original designers. At least part of this difference is attributed to differences between the 2- and 3-dimensional modeling conducted by the original designers and CH2M HILL. The differences included the method of representing the stiffness and the load-displacement within the facewall and tailwall. Methods used in this suitability study are believed to more accurately represent expected OCSP® performance.

- *As-built model for gravity loading*: For the as-built case under gravity loading, FS values from c-ϕ reduction analyses using FLAC^{3D} were lower than required by the PIEP design criteria and lower than results obtained by the original designers. Displacements at these lower factors of safety were greater than desired; however, the exceedance in displacements was not particularly significant. The primary cause of this difference appears to be assumptions made regarding the strength characteristics of the BCF clay, as well as differences in groundwater elevations and live load values.
- *As-built model for seismic loading*: Results from seismic analyses conducted with FLAC^{3D} as part of the as-built analyses suggest that the OCSP® system nearly meets design criteria and performance requirements for the OLE. However, deformations for the CLE and MCE exceed the project design criteria. In some cases, the predicted deformations for the CLE and MCE events are greater than 10 feet. Although this amount of deformation is beyond the reasonable accuracy of the FLAC^{3D} numerical modeling method, it provides support to the results of simplified geotechnical analyses, which found that the amount of movement during seismic events could be very large, similar to what was observed during the 1964 Alaska earthquake.
- *Comparison to limit-equilibrium methods*: Results from the as-built analyses using FLAC^{3D} are also generally consistent with displacements predicted using simpler limit-equilibrium methods associated with conventional geotechnical engineering design and described in Section 5 of this suitability study. The numerical results were very valuable in terms of defining likely zones of shear to be investigated in the simpler limit-equilibrium method. Factors of safety determined by the two methods for static loading were very similar. These observations give confidence in using the simpler limit-equilibrium methods to perform sensitivity studies on design variables.
- *Local defect modeling*: Numerical modeling of two local mechanisms using the FLAC^{3D} model provided valuable insight regarding the development of shear along the tailwall and effects of sheet pile sections that are out of interlock. Because of the complexity of the local defect model, particularly for seismic loading, these analyses were conducted for static loading only (see Section 7.5.2 for further discussion of the approach used and limitations of these analyses). Results of the interface modeling show that shear resistance is determined by soil friction and that the knuckle does not seem to contribute additional bearing capacity to pullout. However, the knuckle does contribute by forcing shear away from the steel-soil interface. Results from evaluations of local defects show that defects (that is, split or unzipped sheet piles) below the mudline are much less serious than those above the mudline. The primary issue with unzipped sheet piles above the dredge elevation is wash-out of granular fill behind the wall, leading to sinkholes at the ground surface. Defects also result in redistribution of shear stresses within the sheet pile system. Although the defects in the facewall are serious, defects along the tailwall are potentially more critical and could result in “unzipping” of the defective sheets. Such an occurrence could be catastrophic to the wall, as stability at the face depends on the ability to mobilize pullout resistance. The effects of repeated seismic loads, particularly those associated with the CLE and MCE, are expected to accelerate the unzipping mechanism. As discussed in Section 6, the OCSP® system is able to accommodate some failures of a tailwall through load redistribution; however, the risk associated with unzipping of the tailwall is significant, and therefore, in the absence of good predictive methods, it is essential to assure that the tailwalls are installed without interlock damage.

- **Constructability.** A number of issues related to the construction of the OCSP® system were identified as potentially contributing to the construction problems that have been observed. These observations relate to the approach taken to install the OCSP® walls. Difficult environmental conditions such as restricted work hours and extreme tidal and current conditions contributed to the past difficulties in constructing the OCSP® system. Details of these analyses are included in Section 8 of this report.

  Additional conclusions are as follows:

  - *Method of construction*: Constructing the taller sections of the OCSP® structure from the dike on the land side of the wall appears to have been one of the main causes of construction problems. The fundamental issue lies with unbalanced soil pressure on the OCSP® wall, which prevents the sheet piling from driving straight.

- *Obstruction from large rock*: The use of rip rap to stabilize the dike slopes during earlier phases of construction increased the likelihood that rocks would be encountered during driving of the sheet piling, even when the contractor was required to remove these rocks. Evidence clearly shows that the sheet piling cannot be driven on alignment through rocks.
- *Environmental constraints*: The listing of the Cook Inlet beluga whales and the associated permit conditions severely limited the time available for pile-driving. The inability to have a sustained work period and the need to cease operations led to inefficiencies in operations. In this case, when combined with the effects of tidal cycles on dike stability, it also resulted in additional sloughing and soil pressures on the sheet piles during renewed pile driving. These permit conditions will hamper any future pile-driving operations at the POA. OCSP® construction is pile-driving-intensive, and any future construction of the system should evaluate whether there is sufficient time allowed to complete the work.
- *Construction experience*: Construction of an OCSP® system in this extreme environment requires finesse and experience. Some of the issues encountered during construction might have been avoided if the construction contractor had experience on a past similar project. However, the length of the sheets in combination with the environmental conditions (that is, tidal fluctuations, currents, beluga whale sightings, etc.) made this a unique construction effort. In hindsight, the problems encountered during construction are not that surprising.

- **Independent Design.** A concept for an independent design was identified and evaluated. The independent design preserved the geometry and type of sheet piles, but other changes were made to the OCSP® system to better meet the PIEP design criteria. It was further assumed that nothing had been constructed at the project site, and the contractor was able to avoid issues encountered during previous OCSP® construction. The independent design concept that was identified and evaluated involved replacing the upper 15 feet of granular backfill within the OCSP® cells with low-density cellular concrete (LDCC) to lessen loads, as well as increasing the resistance in the BCF clay by using ground improvement. The intent of these changes was to increase global factors of safety for gravity loading and reduce seismic displacements. Details of these analyses are included in Section 9 of this report.

  Key conclusions from this design effort were as follows:

  - *Structural performance*: Structural design criteria of the OCSP® system interlocks and tailwall pullout were still satisfied by the independent design. Corrosion issues associate with life-cycle performance were improved by the independent design due to lowering initial sheet pile stresses.
  - *Gravity loading response*: Global stability for static loading was improved such that factors of safety criteria were met. This improvement was the result of greater soil resistance along the slip surfaces, as well as reduced load on the soil-structure system from the use of light-weight fill.
  - *Seismic performance*: Simplified estimates of displacement using published displacement charts suggested that seismic displacements would be small and would meet the displacement requirements for OLE, CLE, and MCE events. Because of the small displacement predictions, the potential for large strength reductions similar to those predicted by the constant-volume ring shear tests on samples from the POA and observed during the 1964 Alaska earthquake at Fourth Avenue, L Street, Government Hill, and Turnagain Heights was avoided.
  - *Numerical modeling*: More rigorous numerical modeling using FLAC^{3D} was not as successful as the simplified limit-equilibrium analyses, suggesting that displacements could still be very large. Part of the large displacement predicted by the FLAC^{3D} analyses occurred within the backfill zone and appeared to be the result of larger energy transfer through the improved ground, and part was due to deformation within the improved ground. These results suggest that additional ground improvement would be required to meet the PIEP design criteria. The additional ground improvement would have meant increasing the

strength of the granular backfill and increasing the strength of the ground improvement in the BCF clay above the 5,500 psf used as a basis for design.

- *Practicality of independent design*: Given the cost and extent of changes with LDCC in the granular fill and ground improvement in BCF clay, as well as construction constraints that would be needed to meet environmental protection requirements, it was concluded that further refinement of the independent design was not justified at this point.

10.2 Recommendations

This report presents results from reviews and analyses performed for the OCSP® system constructed at the POA. The primary recommendations from this suitability study are as follows:

- A study of alternative wharf design concepts should be conducted before further evaluation of the existing OCSP® system. These alternative concepts should include a design based on a conventional pile-supported wharf and a design based on a combination of the existing OCSP® system and piles. As part of this study, the maximum acceptable wall height for the OCSP® system at the POA should be defined. The evaluation should also determine whether any part of the existing OCSP® system can be salvaged for use or modified use.
- Any future design evaluation should include revisions to the original design criteria as discussed in Section 2 of this suitability study report. These revisions should include:
  - Using live loads equal to 1,000 psf extending from the pierhead line into the backlands to provide the POA maximum flexibility in use.
  - Increasing the groundwater elevation behind the wall to elevation +20 feet MLLW to account for more recent groundwater measurements.
  - Decreasing tidal elevation in front of the wall to elevation +7.5 feet MLLW during seismic evaluations to better account for the possibility of a low tidal condition during seismic loading. Even with this lower water level, there is the potential for lower tidal levels 25 percent of the time, and these lower levels increase the loads on the OCSP® system. This risk potential needs to be understood and accepted by the POA and MOA. If a lower level of risk is required, then the tidal elevation should be reduced below +7.5 feet MLLW.
  - Reviewing results of the PSHA with URS to confirm that recent changes in GMPEs and seismic source characterization do not warrant revision of the original seismic hazard model.
  - Conducting supplemental shear wave velocity measurements in the existing backfill material to characterize the shear wave velocity of the backfill and to confirm the effects of backfill weight on the underlying BCF clay. This recommendation is made whether a marginal wharf, OCSP® system, or hybrid OCSP® system is developed.
  - Evaluating seismic performance of backfill and existing soils under earthquake magnitudes similar to the 1964 Alaska earthquake to confirm that the large duration of shaking can be handled by the design, even though this event has a very small likelihood of occurrence.
  - Reassessing performance criteria for seismic loading to confirm the maximum amount of displacement that can be allowed for the OLE, CLE, and MCE. The displacement limits potentially have significant impact on design and construction costs, so it is important that the design team, including the POA, consider the cost implications of the performance limits.
- The evaluation of any alternate port facility at the POA site needs to consider the potential for large loss in strength of the BCF clay during seismic loading. This is particularly critical for the CLE and MCE events, as the predicted levels of ground shaking for both events are significantly greater than what occurred in Anchorage during the 1964 Alaska earthquake. Any new facility must be designed such that displacements during seismic loading are very limited (for example, less than 6 inches) to avoid the potential for large strength loss.

Alternatively, the facility should be designed to meet displacement criteria, assuming that a large strength reduction occurs during ground shaking. If, under the reduced strength, displacements are shown to meet the PIEP design criteria, then a higher level of confidence exists with the new design.

- A combination of simplified and numerical modeling should be used to evaluate the performance of the selected alternative under gravity and seismic loading. It was clear from the numerical modeling conducted by CH2M HILL using FLAC^{3D} that a number of assumptions have to be made when evaluating response during gravity and seismic loading cases with this numerical modeling method. This is particularly the case where the model must account for potentially large changes in strength in the BCF clay, but it also includes boundary conditions for seismic loading, basic assumptions regarding groundwater and tidal elevation effects, and operational loading effects. In view of the complexity and uncertainties of the loading requirements, it is important that independent checks be conducted—including the use of simplified methods—to confirm that results are meaningful. One of the positive things done by the original design team was to enlist the expertise of Drs. Hashash, Robertson, and Mayne to serve as initial advisors on the development of the project. Given the difficulties that have occurred during design and construction, a similar independent review team should be included in any future alternative analyses.
- The original designers performed a significant amount of high quality work for the design of the OCSP® system. This high-quality work included field drilling and sampling, cone penetrometer testing, seismic geophysical testing, and much of the laboratory testing program. State-of-the-practice methods were also used to characterize expected levels of seismic shaking. Engineering analyses dealing with settlement and global stability were also generally high quality. This information provides an excellent basis of continued design, whether it is for a modified OCSP® system, a marginal wharf, or some hybrid system, and should be integrated into future design to the extent practical.
- Construction of any future facility involving an OCSP® system needs to be done either from floating barges or a pile-supported trestle. Under no circumstances should an access dike similar to that used previously be allowed for construction. Further, it is critical that areas containing large rock be clearly identified where wharf piles or sheet piles are to be driven. If there is uncertainty in the location of the rock, probing or spudding should be used before construction to confirm that penetration can be achieved without damaging the piles. The design plans for future OCSP® work must include a workable method of construction for the contractor to perform. Any future methods for construction must be vetted with the construction industry to ensure they are constructable prior to the tendering of the contract.

SECTION 11

Limitations

This report has been prepared for the exclusive use of USACE and the POA for specific application to the PIEP, in accordance with generally accepted engineering practice applicable at the time of this suitability study. No other warranty, express or implied, is made.

The analyses, conclusions, and recommendations contained in this report use geotechnical, structural, hydrological, and construction information obtained from design reports, contract documents, and construction records prepared by others for the PIEP. This existing database of information was collected by the project designers over a period of 10 years or more and involves multiple components of design and construction. Although an effort has been made as part of this suitability study to fully account for the existing database of information through review of project files and discussions with project participants, the extent of data was such that some key information could have been overlooked or misinterpreted. Although not expected, conclusions or recommendations could be affected by information that has not been considered or correctly interpreted, and therefore, any information that has been missed or misinterpreted should be brought to the immediate attention of CH2M HILL to determine its significance relative to the conclusions and recommendations in this report.

This suitability study represents the opinions and conclusions of CH2M HILL. It is an independent study that did not include discussions with the original designer, other than to confirm that all key information had been received. The intent of the suitability study was to obtain, to the extent practical, an independent opinion on the suitability of the existing OCSP® facility relative to standard engineering design and constructability, without bias from the original designer's views, assumptions, or past experience. This approach was taken to preserve the transparency of the study, given concerns about design and constructability that had been raised by the USACE, the POA, and the general public. Information conveyed in a draft of this report was reviewed by and discussed with the USACE, the POA, and the MOA Geotechnical Advisory Commission. Responses to comments from these groups have been integrated in the report to the extent practical; however, these changes have only clarified methods and conclusions reached by CH2M HILL and did not change the opinions and conclusions documented in the original draft report.

Work described in this report is based on subsurface geotechnical information collected primarily by the project designers but is supplemented by a limited field exploration and laboratory testing program conducted by CH2M HILL as part of this suitability study. As in any geotechnical project that involves field explorations, subsurface information from investigations conducted by others and as part of this suitability study indicates conditions present at specific locations and times of sampling and testing. This information does not necessarily reflect variations that may exist between test locations or the changes in strata and engineering properties that may occur with time in a dynamic environment, such as that at the POA. If the subsurface conditions described in this report are found to differ from conditions encountered during future exploration and construction efforts, the findings in this report may need to be reevaluated.

The interpretation of BCF clay shear strength represents a key part of this suitability study. BCF clay strength changes under various stress states and loading conditions, particularly cyclic loading and large-displacement conditions. CH2M HILL's conclusions regarding the BCF clay strength are based on published information regarding previous failure analyses of large ground movements near L Street, Fourth Avenue, Turnagain Heights, and Government Hill in Anchorage during the 1964 Alaska earthquake, as well as a limited number of constant-volume ring shear tests conducted for this suitability study. This available information suggests the potential for a large loss of strength within the BCF clay during a seismic event. If this loss of soil strength occurs during a future seismic event, there is a potential for large seismic-induced ground displacements at the PIEP and consequential damage to the POA facilities and potential public safety issues. Other interpretations regarding the effects of seismic loading on BCF clay can be made and are possible; however, the risk of large ground movement should not be discounted and must be understood by the MOA and the POA for the OCSP® system as currently constructed.

Levels of seismic loading used as a basis for evaluating seismic response at the PIEP in this suitability study are based on work completed by the project designers. The methods used by the project design team to estimate levels of earthquake ground shaking followed state-of-the-practice methods being applied at the time the work was carried out in 2008. The nature of all seismic studies is such that uncertainties exist in the predictive methods and results. Although an effort has been made during this assessment to understand and account for these uncertainties in the estimation of ground motions with currently available methods, higher levels of ground shaking could occur. Further, the profession's understanding of earthquake ground motions continues to evolve and, as a result, the basis of the ground motion recommendations could change in the future. These changes could necessitate reconsideration of the recommendations used as a basis of seismic design within this suitability study.

As a final limitation, in the event that any changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by CH2M HILL. CH2M HILL is not responsible for any claims, damages, or liability associated with the use of information in this report, including interpretation of subsurface data or reuse of the subsurface data, without the express written authorization of CH2M HILL.

SECTION 12
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