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	Engineering and Design EVALUATION OF I-WALLS	
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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

CECW-CE

Technical Letter
No. 1110-2-575

1 September 2011

EXPIRES 1 June 2016
Engineering and Design
EVALUATION OF I-WALLS

1. Purpose. This Engineer Technical Letter (ETL) provides updated technical criteria and guidance for evaluation of existing I-walls. General guidance for performing I-wall evaluation is provided along with detailed updates to existing guidance that focus on three I-wall performance items: the flood-side gap that was discovered at I-walls in New Orleans after Hurricane Katrina; the rotational stability failure mode found to be the critical failure mode for most I-walls evaluated nationwide in the Phase II evaluation; and criteria for consideration of deflections. This guidance is for I-walls designed to provide flood risk reduction from inland flooding, not from coastal storm surges with significant wave and vessel impact loads. I-walls in coastal areas shall be evaluated in consultation with the Headquarters, U.S. Army Corps of Engineers.

2. Applicability. This ETL applies to all USACE commands having civil works responsibilities. This ETL applies to levee systems that are USACE operated and maintained and levee systems that are federally authorized and locally operated and maintained.

3. Distribution Statement. Approved for public release, distribution is unlimited.

4. References. References are at Appendix A.

5. Background. Investigations of the hurricane risk reduction systems in Louisiana identified possible deficiencies in the guidance used to design I-walls. The design deficiencies centered on the phenomenon of a gap formed by flood loading between sheet pile and the soil on the flooded side of the wall. This gap contributed to several breaches of I-walls in New Orleans prior to their overtopping related to global stability or seepage. The U.S. Army Corps of Engineers (USACE) therefore issued guidance regarding these deficiencies (Headquarters, U.S. Army Corps of Engineers (HQUSACE), 2006a). The compilation of data and site inspections required in this guidance was considered Phase I of a multiphase approach to evaluating existing I-walls under USACE jurisdiction throughout the United States. Phase II Interim Guidance was then prepared and disseminated (Headquarters, U.S. Army Corps of Engineers (HQUSACE), 2006b) to help Districts evaluate and identify projects that may be at risk of poor performance. This effort resulted in the identification of over 50 projects with potential performance concerns. Most of the projects not meeting the criteria of the Phase II guidance failed to meet factors of safety for rotational stability or the check of a minimum ratio of 2.5 for depth of penetration to wall height.

6. Discussion. This ETL provides guidance for detailed evaluation of I-walls and was developed under Phase III of the I-walls evaluation program. It resulted from research of I-wall

behavior and failure modes that expanded on the knowledge gained from the performance of the New Orleans Hurricane and Storm Damage Risk Reduction System (HSDRRS). Additional numeric model studies were performed on wall sections and soil types representing typical projects from around the United States. Analysis criteria were developed from the research and from past full-scale load tests. Risk and reliability were used to evaluate the research data and to determine factors of safety and deflections that provide appropriate performance.

7. Action. The guidance in Appendix B shall be used for the evaluation of existing I-walls.

FOR THE COMMANDER:

4 Appendixes:
As listed in Table of Contents

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EM 1110-2-1902
Slope Stability

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APPENDIX B
GUIDANCE FOR EVALUATION OF I-WALLS

B-1. Objective.

a. General. Evaluation of I-walls shall be performed by carefully considering all potential failure modes. This appendix provides criteria for evaluation of existing I-walls, including a detailed description of evaluation of stability and seepage. An I-wall is defined as a slender cantilever wall, deeply embedded in the ground or in an embankment. The wall rotates when loaded and is thereby stabilized by reactive lateral earth pressures. A description of performance requirements for the evaluation is provided that includes performance modes, basic criteria, and requirements for supporting analysis information. The failure modes to be evaluated are described and defined, as are the methods and tools to be used to analyze them. Safety factors to compare to limit equilibrium analyses are included. Guidance and design methodology for the flood-side gap between the sheet pile and soil are given.

b. Limitations.

(1) This document addresses stability, seepage, and deformation only. It does not contain new guidance for evaluation of the strength or serviceability of structural components.

(2) The guidance is for I-walls intended to provide flood risk reduction for inland flooding, not for I-walls in coastal storm surge risk reduction projects where waves are a significant portion of the load and vessel impact is possible.

B-2. Requirements for Evaluation.

a. General Evaluation Performance for I-walls.

(1) It is essential that the evaluation of the I-walls and their ancillary features critical to the operation of a levee system ensures the integrity to the top of the levee. The stability and operational adequacy of an I-wall during a storm event are of paramount consideration during inspection, assessment, and evaluation. The I-wall shall have sufficient robustness to survive without incurring the type of damage to the system that would impact its ability to prevent catastrophic flooding and maintain interior drainage operations.

(2) Wall deformation under load is another requirement for evaluation. For more frequent events, it is desirable for I-walls to have little or no permanent deformation after loading although for unlikely events some amount of permanent deformation may be allowed. Requirements for deformation control are provided in this ETL through the use of maximum water elevations likely to result in acceptable deformation for different soil types and frequency of flood events.

b. Engineering performance guidance for I-walls. I-walls will generally be evaluated using traditional limit-state type analyses, for which requirements and factors of safety are provided in paragraph B-5. In certain soil conditions and water levels, unacceptable levels of deflection may occur before soil strength limit states are reached. Factors of safety and restrictions on water elevations in this ETL are intended to address these conditions. If the results of the analyses in paragraph B-4 do not meet the performance requirements in paragraph B-5, then advanced analyses or remedial measures are required as described in paragraph B-6.

c. Collection and Incorporation of Existing Information.

(1) Evaluation of existing I-walls can be different from the design of new I-walls because additional performance information is often available. Construction and operation information is useful to verify that the I-wall is performing as the designer intended, to estimate degradation of the I-wall caused by service conditions, and to reduce the uncertainties associated with the hydrologic and site conditions, engineering properties, and I-wall dimensions to a practical minimum.

(2) Before an evaluation is performed, the following steps should be taken:

(a) Search all available records of design, construction, and operation especially with regard to the flood of record, location of overtopping sections, and operational procedures associated with overtopping.

(b) Review historical data concerning signs of leakage, excessive seepage or piping, and records of repairs.

(c) Locate as-built drawings, specifications, O&M manual(s), computations, subsurface explorations and testing records, hydrographs, inspection reports, and instrumentation data.

(d) Perform a field inspection that focuses on the observed vertical and horizontal distortion of the I-wall alignment, signs of dredging activities, global instability or erosion of the riverbank or levee, visible damage to the I-wall, postconstruction modifications, encroachments, and vegetation.

(e) Review existing survey data to ensure that the project datum requirements are met.

(f) Verify hydraulic levels and return frequencies.

(g) Review potential for overtopping of the I-wall prior to inundation of the area behind the levee system and provisions for scour protection.

d. Evaluation According to Project Information.

(1) Ordinary project information available.

(a) Ordinary project information includes visual, photographic, historical, and anecdotal information about the water levels, I-wall and foundation performance, and signs of distress. In this category, maximum water stage that the wall has been subjected to has not been higher than 1 ft (0.3 m) below the evaluation water level on the I-wall. Therefore, past performance does not provide sufficient information for judging expected performance at the water level used for evaluation. Ordinary levels of site information shall provide evaluators with a high level of confidence in the expected design loads at a site. Available construction data for existing projects must correlate closely with design computations and specifications. Foundation stratigraphy, material parameters, and site geometry can be established with a reasonably high level of confidence.

(b) Existing survey data may be used only if the uncertainty associated with the hydraulic and geodetic data is minimal and the top of wall elevations can be established with a high level of confidence.

(c) If the evaluator does not have a reasonably high level of confidence in the information, sensitivity analyses may be performed using realistic upper and lower bounds on key parameters to determine which hydrologic and site data, engineering properties, and wall dimensions have a critical effect on the I-wall response. Based on the results of the sensitivity analyses, the following project information requirements may be reduced:

- Subsurface investigations.

- Testing.

- Wall information required (sheet-piling type/section).

(d) Results of the sensitivity analyses may show that the I-wall has a reserve capacity (the calculated safety factors exceed the minimum required values). If the results of the analyses show that the safety factors are significantly greater than the minimum requirements (calculated factor of safety values that exceed the minimum requirements by 0.5), then verification of only the sensitive critical parameters should be performed.

(2) Well-Defined project information available.

(a) Well-defined project information includes the visual, photographic, historical, and anecdotal information described for the ordinary project information. Additionally, a well-defined site involves comprehensive levels of knowledge concerning both project design and performance history. Accordingly, a high level of confidence must be attained concerning both subsurface conditions/parameters and loading conditions to qualify as a well-defined site. Foundation stratigraphy, material parameters, and site geometry can be established with a high level of confidence, and borings are sufficiently deep and closely spaced so that the likelihood of encountering unforeseen foundation conditions is extremely low. Information needed to provide a high level of confidence is project specific, and the foundation investigation needed to support criteria for this level of understanding is the responsibility of the district and major subordinate command (MSC). For example, consideration should be given to variability of geology, depth of sheet-pile penetration, cross-section geometry, and height of water loading.

(b) Past performance can be an important factor in assessing future performance. When past performance is evaluated, it is important to consider any significant changes that could potentially affect the performance of the wall relative to stability and seepage. A good example would be any major changes with respect to vegetation, encroachments, animal burrows, or any other change that did not exist at the time when the past loading event occurred. The same could be true if the wall performed poorly previously and a contributing factor (vegetation, encroachment, etc) has since been eliminated. Problems observed during lower loading conditions will likely be exacerbated during larger events. Likewise, walls that perform well during a past load event provide information that can be used to inform the analyst's judgment on anticipated performance during future loading events of a similar magnitude. If the maximum water stage that the wall has been subjected to is higher than 1 ft (0.3 m) below the evaluation water level, and the wall performed satisfactorily with respect to deflection and seepage, then past performance provides information that can be used to inform judgments on the expected performance at the water level used for the evaluation. Criteria under the Well-Defined category may be used for evaluation even if the other project information available is classified as Ordinary. When more than one water level is evaluated, lower water levels may qualify as being Well-Defined while higher levels may not.

(3) Limited project information available. If some of the project information (one or more critical parameters such as wall tip depth or soil strength) does not exist, then an appropriate investigative program should be executed to ensure that information consistent with an ordinary site classification exists before evaluation. If construction records of walls are missing, then field investigations should be performed to determine the I-wall dimensions (top of wall elevation, pile tip elevation, type of piling, connection of the cap to sheet pile, etc.) accurately. Results from nondestructive methods used to determine wall geometry should be corroborated with results from sufficient destructive test sections to establish a reasonable degree of confidence in the evaluation results. In some cases, a project with Limited project information may meet the requirements of this ETL when very conservative assumptions for design parameters are used. In these situations, the analyst should consult with MSC and Directorate of Civil Works, Headquarters, U.S. Army Corps of Engineers (CECW-CE), on a case by case basis to determine the appropriate approach to use in the evaluation.

e. Vegetation and Encroachment Considerations.

(1) Assessing vegetation growth, primarily trees, adjacent to I-walls and its potential to affect performance with respect with stability and seepage is a very complex issue. Each situation is unique, and there is no prescriptive process on how to conduct such an assessment. Each situation requires site-specific considerations by experienced, knowledgeable personnel. Important factors to consider include the proximity of the vegetation growth to the I-wall, the density of the vegetation growth, type of vegetation, past performance under significant load, geology and geotechnical properties of the foundation, construction methods, duration of head pressures significant enough to initiate seepage and/or piping, velocities, and the ability to detect issues if they were to arise during a flood event. Large trees will tend to be of more concern than smaller trees, and dense vegetation is of more concern than sparse vegetation. When wall stability is being considered, the potential for windthrow (uprooting and overthrowing of a tree by wind) of nearby trees also needs to be considered. Large trees with deep-rooted systems, when uprooted, can

remove a significant amount of earthen material required for global stability. In addition, the potential for trees falling on and impacting the I-wall itself needs to be evaluated.

(2) A similar approach can be taken when considering encroachments. Not all encroachments are necessarily harmful with respect to I-wall performance. Encroachments that shorten the seepage path and increase the exit gradient can adversely affect I-wall performance in certain situations. These are generally those that involve excavations for structures or other features where the foundation lends itself to a potential seepage and/or piping concern. Again, each situation is unique and will require a case-by-case assessment.

B-3. Geotechnical Information for Evaluation. The following shall be used for analyses described in paragraph B-4.

a. Site Information. Proper analyses cannot be performed without understanding the project stratigraphy, the strength of the materials, seepage and pore-water pressure conditions, the strength of backfill materials, and all loads and load conditions to which the structure may be subjected. Without adequate foundation exploration and testing, the safety factors provided to assess the structure performance are meaningless. Lower factors of safety are permitted by this guidance in cases where there is a high level of confidence in the site foundation conditions. Conversely, higher factors of safety are required when there is not a high level of confidence information on either foundation or structure properties.

b. Selection of Soil Shear Strength Parameters.

(1) The selection and application of material properties for analyzing the stability of walls and slopes is detailed in EM 1110-2-1902 and EM 1110-2-1913. Existing data shall be used for the preliminary evaluation as much as possible. If Limited project information, as discussed in paragraph B-2d(3) above, requires that additional subsurface information be obtained, EM 1110-2-1913 discusses testing methods to determine shear strength for both drained and undrained type loading conditions. Often direct shear, unconfined compression, and triaxial compression testing methods are used to determine shear strengths when design conditions require accurate slope stability assessment. Along with laboratory tests, field tests can be useful for determining shear strengths. In locations where I-walls are located within embankments, shear strengths shall be determined both at the center line and at the toe of the embankment as appropriate. Care shall be taken in determining shear strengths of soft soils. Larger sample sizes, sensitive cone penetration testing (CPT) equipment, vane shear tests, or specialized laboratory testing may be considered.

(2) Appropriate soil shear strength parameters shall be used to evaluate existing I-walls. Foundations that have consolidated under current embankment loads may still generate pore-water pressures when sheared and fail in an undrained condition when loaded. For free-draining soils, effective stress strength parameters shall be used in the analysis with the pore pressures determined from the loading conditions. In fine-grained soils, either total stress strength parameters and stresses or effective stress strength parameters and stresses shall be used, depending on the load duration and permeability of the fine-grained soils.

(3) Reliability of design is a result of both the practice used in the selection of parameters and of the safety factors used in design. If selected parameters are less conservative, safety factors must be raised to yield the same level of feature reliability. The criteria presented in this ETL are based on the selection of soil shear strengths using what is commonly referred to as the one-third two-thirds rule for individual soil strata. Using this rule the Mohr-Coulomb shear strength is selected by creating a shear stress versus normal stress plot of available test data and positioning the shear strength line such that about one-third of the data points lie below the line and about two-thirds of the data points lie above the line. Judgment is used in the process of selecting shear strength based on general rules (i.e., the effective stress shear strength of a normally consolidated clay passes through the origin so that the effective cohesion is zero). This rule is generally equivalent to a statistical approach of using linear regression to find the mean minus 0.5 standard deviations.

B-4. Stability and Seepage Failure Modes for Evaluation.

a. General.

(1) Failure modes to be evaluated are rotational stability, global stability (also known as translation or deep-seated failure), and seepage. The rotational failure mode was found to be the controlling failure mode for almost all wall sections analyzed numerically for the work performed under the I-walls Phase III program. Translation or a mixture of translation and rotation failure modes was found for I-walls founded on levees in soft clay materials analyzed in the post-Katrina Interagency Performance Evaluation Task Force (IPET) study and in the I-wall in the Tell City, IN, full-scale load test site of a stiff clay over a soft clay.

(2) Drained soil properties, undrained properties, or both may be used for the evaluation depending on the timing of the high-water event relative to the permeability of the soil. Sites with cohesionless (free-draining) soils will be analyzed using drained soil properties only. Studies performed for Phase III showed that sites with materials predominantly CL or CH according to the Unified Soil Classification System and with flood hydrographs of no more than one week in total flood duration above the ground surface next to the wall can be analyzed using undrained properties only. All other sites will be analyzed with both drained and undrained soil properties as either may be present during a flood event.

b. Soil-Sheet-pile Gap in Analyses. Formation of the gap between the soil and sheet pile is associated with undrained soil conditions. Research for Phase III of the I-walls program showed that a gap is developed to a depth of about $2c/\gamma'$ and possibly up to $4c/\gamma'$ (where c is the cohesive strength and γ' is the effective soil unit weight), or to the kickback point near the bottom of the sheet pile where passive pressures exist to balance wall rotation, whichever is higher. Incorporation of the gap in analyses is found in the description of each failure mode described in paragraphs c-e below.

c. Rotational Stability.

(1) In the rotational failure mode the wall rotates about a point near the tip of the wall. Equilibrium is achieved by a balance of water pressure and of active and passive soil pressures

that depend on the wall deflection. Driving loads are primarily from the water (flood) force, and resisting pressures are the passive pressures near the ground surface on the land side of the wall and near the tip of the sheet pile on the flood side of the wall. The pressures on both sides of the wall are computed using soil pressure equations, and the point of rotation is found that provides force and moment equilibrium.

(2) A factor of safety can be found using the analysis mode of CWALSHT (U.S. Army Engineer Research and Development Center 2007) or by trial and error, specifying a passive factor of safety in design mode that results in a calculated pile tip at the correct elevation. The drained condition is analyzed as described in EM 1110-2-2504. Seepage effects shall be included in the drained analysis.

(3) CWALSHT performs analyses correctly for drained soil conditions but is not capable of performing a true total stress analysis needed for a precise analysis of the undrained soil condition. However, CWALSHT can be used to evaluate I-walls in undrained soils using an empirically derived procedure that uses both effective stress and total stress strength parameters to approximate a solution for a fully saturated, undrained condition that accounts for the gap. The procedure is based on the following assumptions:

(a) That a gap will form between the wall and fine-grained soils wherever hydrostatic pressure is greater than the active pressure; in most I-wall problems this will occur to the point of rotation.

(b) That the sum of the water pressure and effective passive soil pressure on the flood side of the wall below the point of rotation is approximately equivalent to the passive pressure computed with a total stress analysis (which is true where the active pressure coefficient K_a and the passive pressure coefficient $K_p = 1.0$).

(4) The procedure for performing undrained analysis in CWALSHT is as follows:

(a) Model the geometry of the wall and ground.

(b) Specify factors of safety for the active and passive earth pressure computations in the usual manner for CWALSHT analyses. For evaluation of existing I-walls, the factor of safety for active pressures will be 1.0 and the factor of safety for passive pressure will be solved by CWALSHT.

(c) Specify the moist or saturated unit soil weights depending on whether soils are above or below the water table.

(d) Specify undrained shear strengths (i.e., unconsolidated undrained or Q-case) with cohesion c set equal to the undrained shear strength S_u and with the internal friction angle ϕ set equal to zero for all fine-grained soil layers (on both sides).

(e) Specify a water level in CWALSHT on the right-hand (driving) side consistent with the flood elevation of the river or canal. Specify a water level in CWALSHT on the left-hand (resisting) side at the ground surface or drain elevation. The seepage type should be “none.”

(f) The factor of safety is found using the analysis mode or by specifying by trial and error a passive factor of safety in design mode that results in a calculated pile tip at the correct elevation.

(5) CWALSHT uses wedge methods or equations to compute earth pressures. In some cases, the earth pressures computed by CWALSHT may have significant error. This has been found to be true for compacted clay levees on soft foundations where the passive resistance may be limited by the critical block resistance rather than the passive wedge. Therefore, the resistance offered by a potential slip surface in the foundation material beneath the levee may be less than that offered by a slip surface through the higher strength levee fill material in a passive wedge. A method for adjusting the earth pressures in CWALSHT for I-walls on levees is provided in Appendix D. The minimum passive resistance is found using a slope stability program and the net pressures adjusted by the use of an applied lateral pressure or force.

(6) Wall friction should be included in the analyses. Representative wall friction values for the drained condition can be found in Tables 3-2 and 3-3 of EM 1110-2-2504. For the undrained case, soil friction forces are represented by adhesion. Adhesion can be computed as shown in Figure 4-5a of EM 1110-2-2906.

(7) The traditional limit equilibrium analysis as performed by CWALSHT considers the wall as a uniform sheet with no thickness. Research using numeric models showed that the concrete cap on an I-wall can increase rotational stability and reduce deflections for walls under the drained soil condition to some degree. However, the cap was found to have little effect for conditions with undrained clays. Effects on performance depend on the width and depth of the concrete cap, along with other variables including whether any settlement may occur beneath the cap. Incorporation of this into a limit equilibrium analysis would require full study of the relationship between the cap and theoretical soil pressures that was not performed under Phase III. Therefore, I-walls will continue to be analyzed under the assumption of a wall of uniform thickness. Incorporation of the concrete cap into an evaluation can be performed using a numerical model as described in paragraph B-6b. The evaluation using a numerical model should account for potential settlement of soil beneath the cap, and shear stresses in the soil around the cap should be carefully reviewed for reasonableness.

d. Global Stability (Deep-Seated Failure).

(1) In this failure mode the wall is assumed to displace along with the soil mass in which it is embedded when it slides or rotates under a slope stability type failure mechanism. This failure mode is most likely to be critical when I-walls are located on top of levees in very soft soils. Global stability is evaluated using typical slope stability software for the “with” and “without” gap analyses. Methods of global stability analysis shall satisfy all conditions of static equilibrium. A flood-side water-filled gap can be included by removing the flood-side soil to the bottom of the gap and replacing it with a mechanical pressure to represent the hydrostatic water load against the wall. Use of tension crack options in software packages can be used but should be checked for correctness as older versions of common slope stability software packages had errors in the treatment of submerged tension cracks (see paragraph (4)(a) below). Land-side piezometric conditions used for stability analysis shall be based on seepage analysis as described in paragraph e below, Seepage.

(2) Formation of a flood-side gap was an important factor in the breach of I-walls prior to overtopping in New Orleans during Hurricane Katrina and was investigated by IPET. Therefore, a gap should be considered when an I-wall or I-wall/levee composite system is analyzed for global stability, piling tip penetration, uplift, and piping. Because methods for determining gap depths are considered approximate, global stability needs to be checked for the no-gap and full-gap conditions and possibly the partial gap condition. Under the no-gap and full-gap conditions stability is performed assuming either that no flood-side gap develops or that a gap will extend to the bottom of the sheet piling or to the bottom of the fine-grained material, whichever is deeper and up to 5 ft (1.5 m) below the sheet-pile tip as shown in Figure B-1 (for seepage analyses only). Because saturated granular soils will not sustain a gap, a gap is not presumed to develop in these materials. So when cohesive soils overlie granular soils, the gap depth may propagate to the top of the granular layer but no deeper. The condition where cohesive soils underlie granular soils was not investigated; however, the previous assumption that the gap will extend to the bottom of the sheet pile or to the bottom of the fine-grained material shall be followed.

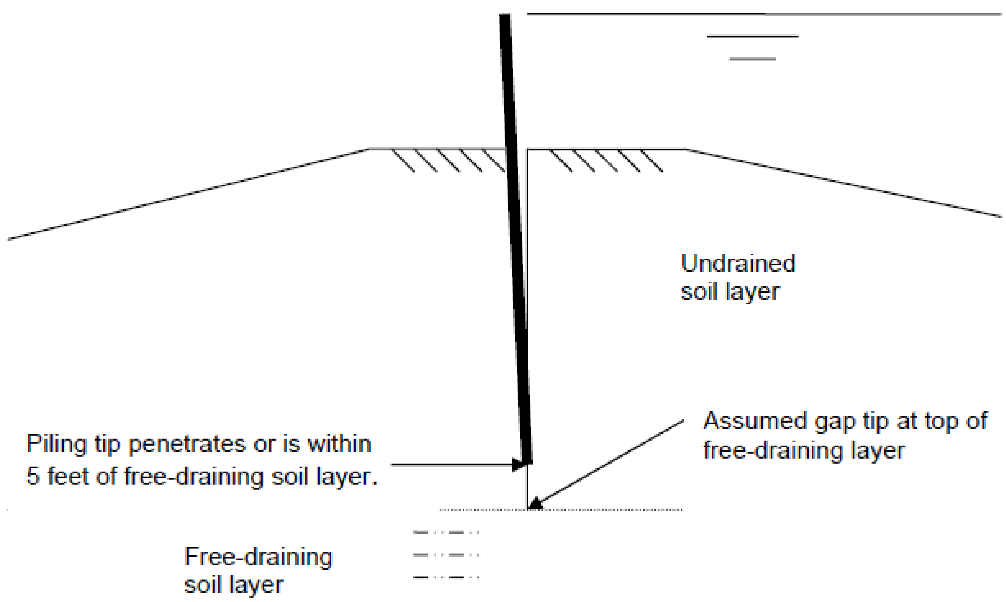

Figure B-1. Potential crack propagation below flood-side gap assumed for seepage analyses

(3) Under these conditions the reinforcing effect of the sheet pile is neglected, and potential slip surfaces are allowed to pass through the sheet-pile location. If global stability satisfies the criteria, the analysis is complete. Neglecting the sheet-pile effect can be a very conservative assumption for I-walls atop levees in soft soils. If global stability is not satisfied when the sheet-pile effect is neglected, a partial gap stability procedure will be used (see paragraph C-5) for I-walls acting as short piles. This procedure restricts the slip surface location from passing through the sheet pile (forcing slip surfaces to depths equal to the sheet-pile tip or deeper) and bases the depth of gap on a comparison of the flood-side active earth pressure to the hydrostatic water pressure. The hydrostatic water pressure is applied to the wall from the selected water level to the bottom of the gap, and active lateral earth pressure is applied to the

wall from the bottom of the gap to the sheet-pile tip as seen in the examples in Appendix C. The use of the partial gap procedure satisfactorily modeled the breaches that occurred along London Avenue Canal in New Orleans. In this situation an I-wall was located atop a levee on a soft foundation. The sheet pile penetrated to shallow depths below the weak layers, and the sheet-pile tips were founded in sand. Although not demonstrated in performance of I-walls in New Orleans, conditions may exist where a partial-depth gap develops in an all-clay site. Lacking further information, it is assumed that a partial gap analyses can develop in an all-clay foundation. The procedure for analyzing this condition is presented in Appendix C.

(4) Three analysis issues need to be considered when creating stability models. The first concerns the proper modeling of water pressures in the flood-side gap, the second is whether or not to include the sheet piling in the stability model, and the third relates to computational limitations associated with the program.

(a) Water pressures. The influence of the flood-side gap may be incorporated in two ways in the computational models: use of a tension crack model or a model that includes the removal of the flood-side soil to the bottom of the gap. Of these, removing the flood-side soil to the bottom of the gap is probably easiest with the least likelihood of error. Many computer programs, for example, UTEXAS4 (Wright 1999) and SLOPE/W (GEO-SLOPE International 2007), allow the inclusion of a tension crack that may or may not be filled with water. Analyses during preparation of the IPET report (IPET 2007) found that using this option produced inaccurate results when the water levels were raised above the elevation of the top of the crack. Some software (including UTEXAS4 and SLOPE/W) has since been corrected; however, designers should verify that the version they are using is up to date and accurate before using the tension crack option to represent the flood-side gap.

(b) Sheet piling. Sheet piling may be embedded in a levee embankment to act as a cantilever wall, a seepage barrier, or a stiffening panel that resists deep-seated sliding. When the sheet pile is driven to lengths deeper than needed for rotational stability, the pile may begin to act as a long pile rather than a short pile. Sheet pile used as a stiffening panel is considered only in the partial gap stability procedure for short piles (or truncated at a short pile length in the stability model) or if using advanced modeling (finite element/difference) procedures. For other cases embankment stability shall satisfy the slope stability criteria in paragraph B-5 using analyses that neglect the sheet piling in the embankment stability model. Satisfying these criteria will constrain the levee and foundation to small deformations.

(c) Computational limitations. Automatic search routines in various commercial software for embankment stability analysis may not return the most critical slip surface (least factor of safety), and engineers should exercise due diligence before accepting the results of searches for critical failure surfaces. For instance, some software allows tension crack input to the bottom elevation of the sheet piling, and will also perform searches with all failure surfaces passing through a given point. However these two input options may not be able to be used concurrently with consistent results. Engineers have avoided this problem by removing flood-side soils completely, thereby eliminating the tension crack. Additionally, I-walls on flat ground (without embankments) can result in very high safety factors for deep-seated failure surfaces. Most computer programs will not calculate such high safety factors, so search routines for these conditions usually fail to generate complete results. Engineers may need to perform several searches with varying search parameters to determine the minimum safety factor confidently for deep-seated sliding.

(d) Special considerations. Breaches of I-walls during Hurricane Katrina did not result from overstressing the sheet piling. The soils in New Orleans are very soft with low shear strength and modulus of elasticity compared to the lateral stiffness of the sheet piling. Therefore, the I-walls acted like a rigid body when the soils deformed. With this understanding, IPET performed limit equilibrium stability analyses by assigning the sheet piling a high strength, which forced the failure surface below the sheet-piling tip (IPET 2007). Therefore, in soft soils, the slope stability of a small soil mass may be increased by considering the sheet piling as reinforcement. However, including the sheet piling in the stability analysis is complex. If sheet piling is used to improve stability for soft soil conditions and the minimum required safety factors are not met using analyses that neglect the sheet piling in the embankment stability model, then the foundation, embankment, and wall displacements may be unacceptable. The maximum water levels for I-walls provided in this ETL are expected to control I-wall deflections. In soft soil conditions the partial gap stability procedure may be used as described in paragraph B-4d(2). For conditions that are not similar to those described in New Orleans, a numerical deformation analysis (e.g., nonlinear finite element analysis) may be used to verify that the sheet piling will improve stability with acceptable displacements. Modeling this soil-structure interaction problem using numerical deformation analysis is discussed in paragraph B-6.

e. Seepage. In the seepage failure mode, water seeping under the wall can lead to loss of the supporting soils on the land side of the wall. Consequences of inadequate seepage control include piping, heave, or excessive quantity of flow. Controlling piping and heave are critical to maintaining the integrity of the I-wall and limiting deflection. On London Avenue Canal in New Orleans, seepage was a significant factor in the breach of the wall prior to overtopping. The two breach areas on the London Avenue Canal are characterized by a sand foundation overlain by fine-grained marsh deposits. Because the sheet piling ended within the sand aquifer, pore-water pressure landward of the sheet piling increased quickly after formation of the gap. The sand aquifer was likely saturated so that only a small volume of flow was needed to raise pore-water pressures significantly.

(1) Since steel sheet piling is permeable at the joints, it may not be 100 percent effective as a cutoff. Seepage analysis shall be performed in accordance with the applicable portions of EM 1110-2-1901 and EM 1110-2-1913. I-walls shall be checked for seepage erosion (piping) by evaluating the critical seepage gradient as described in EM 1110-2-1901 for uplift and heave. An event tree approach is being used to estimate the probability of failure in USACE reliability studies, but for I-wall evaluations the focus is on preventing initiation of piping. The seepage analysis shall consider the flood-side gap, which will shorten the seepage path. If a free-draining layer is present and close to the sheet-piling tip, as shown in Figure B-1, or if the sheet piling penetrates the free-draining layer, a seepage analysis shall be performed. In this case, the vertical distance between the piling tip and the top of the free-draining layer would be considered to be the flood-side blanket thickness. The head at the land side of the barrier can then be calculated using typical methods of analysis and tools. When the wall foundation materials

consist entirely of clay, the lesser potential for developing a steady-state seepage condition may negate the need to check for piping.

(2) When a flood risk reduction system will consist of I-wall/levee composite systems, sheet piling required for rotational stability may not fully penetrate the levee embankment. Accordingly, seepage beneath and/or even through the levee must also be addressed. The geotechnical engineer shall use applicable guidance in EM 1110-2-1913 to assess uplift pressures and/or exit gradients beneath levee embankments. The analysis of embankment seepage, often termed through seepage, is complicated by the presence and effectiveness of the sheet pile as a seepage barrier. Through seepage can reduce the stability of the land-side slope, and it is this aspect that is a design concern. There are no specific criteria for the horizontal seepage gradient since adequacy of design is addressed by slope stability criteria presented in EM 1110-2-1913. If stability becomes critical, the pore-water pressures used in the analysis can be estimated from flow nets or finite element analyses as discussed in (3) below.

(3) Several procedures are available to analyze seepage and uplift. Graphical methods (flow nets); analytic or closed-form solutions that have been solved for specific conditions; method of fragments; and finite elements are common tools. Advances in the hardware and software associated with modern digital computers have greatly reduced the time and effort to perform numerical analyses, and analysis of seepage by finite elements has become routine for many designers. Finite elements are often used where the substrata system is considered too complex for generalized characterization, and the flood-side gap for seepage analysis is easily incorporated with this method. Several computer programs couple results from finite element seepage analysis with limit equilibrium slope stability programs to aid in estimating pore-water pressures.

B-5. Evaluation Performance Criteria. I-walls will be evaluated for all of the performance requirements in this paragraph. In general, expected performance is evaluated by a factor of safety for the failure modes described in paragraph B-4. However, additional requirements are provided for situations where factors of safety are insufficient to address deflection control and resilience. For this ETL, resilience is defined as the capacity of a component, unit, or system to withstand a particular load with permanent deflection small enough to allow the project to continue without rehabilitation.

a. Factors of Safety for Evaluation of Failure Modes. Factors of safety must provide assurance that the I-wall will meet performance objectives for loadings determined to have a reasonable chance of occurrence during the life of a project. For the limit equilibrium analyses, the primary goal is to provide factors of safety that account for uncertainty so that the chance of breach prior to overtopping is very low. The factors of safety can also provide some control of deformations, but cannot be relied on to do so for all cases.

(1) Rotational stability.

(a) Factors of safety for rotational stability were developed from numerical models, scale models, and field test measurements to provide low likelihood of breach prior to overtopping.

(b) In addition, rotational factors of safety were computed for a number of I-walls for which deformation had been compiled. This is shown in deformation plots from the Phase III research shown in Figure B-2, which shows a trend with an exponentially increasing rate of deflection when factors of safety are less than 1.5. A dotted line with squares is outside the trend for the other sections in Figure B-2. This is from an I-wall on levee section from the London Avenue Canal in New Orleans, LA. A model of the London Avenue Canal I-wall using Fast Lagrangian Analysis of Continua (FLAC), a finite difference code, indicated that a rotational failure mode would occur with lower factors of safety than were computed using CWALSHT. The plot shows that CWALSHT may not compute factor of safety accurately for I-walls on levees using default settings, as discussed in paragraph B-4 c. (5). The (orange) dashed line with triangles shows the results when passive pressures are corrected to account for the levee section, as described in Appendix D.

(2) Global stability. Minimum required safety factors are provided to address how well the foundation conditions are known. The factors of safety are associated with limit equilibrium analyses that satisfy all conditions of static equilibrium; however, finite element analyses may also be used to solve for global stability. Criteria are provided for loading causing failure toward the land side. The selection and application of material properties for analyzing the stability of walls and slopes are detailed in EM 1110-2-1902 and EM 1110-2-1913. Failure toward the flood side is also covered in EM 1110-2-1902 and EM 1110-2-1913 and is not included herein.

(3) Seepage analysis. Because of their permeability and duration of loading, steady-state conditions may not develop in fine-grained soils. However, open seepage entrances and noncontinuity in blanket materials may allow steady-state conditions to occur in coarser strata.

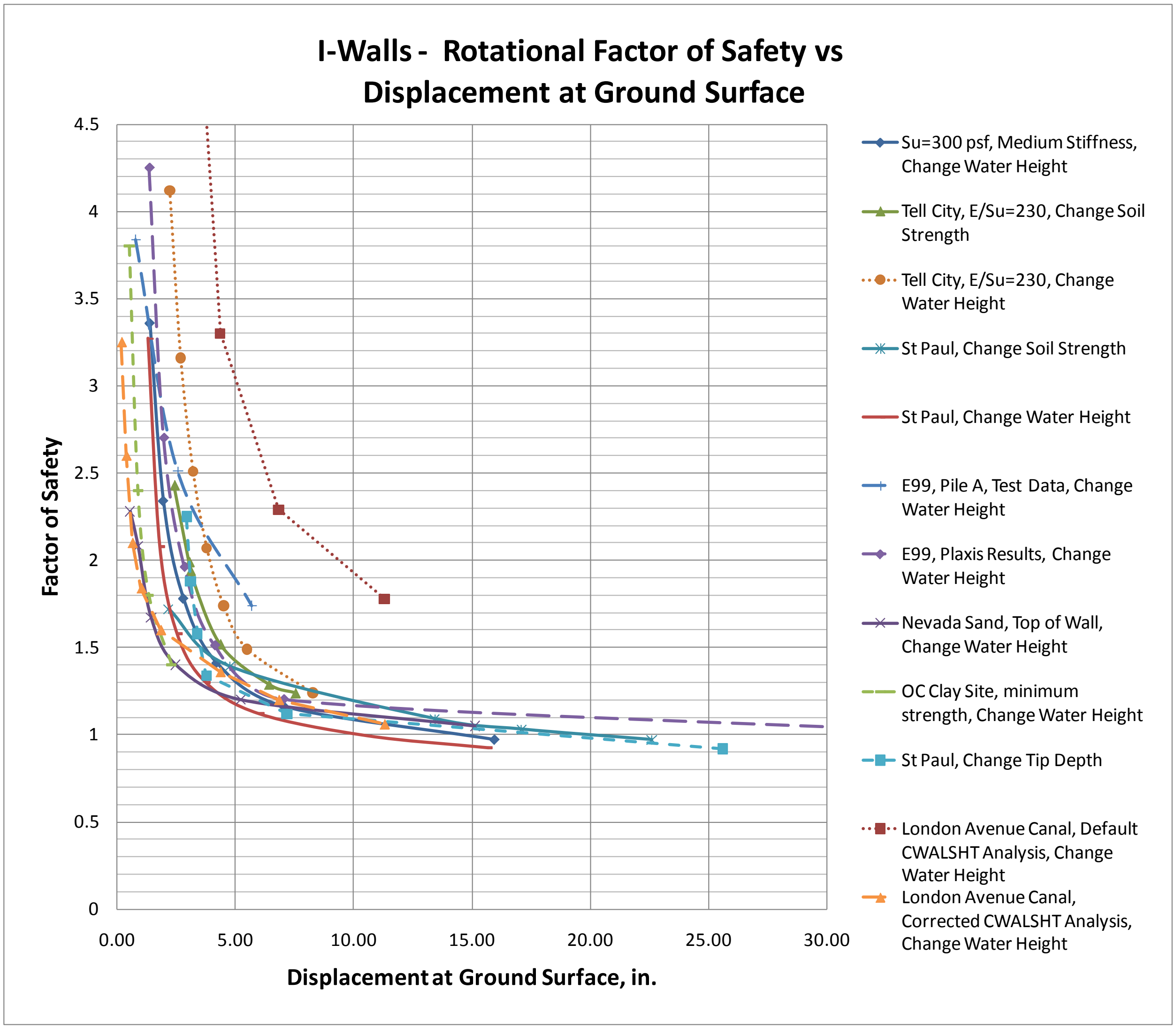

Figure B-2. Rotational factors of safety versus displacement at ground surface for I-walls

(a) For stratigraphy conditions, where a surficial blanket exists over a coarse stratum, the minimum required safety factors for seepage are based on the gradient through the blanket and are defined as:

$$FS_g = \frac{\gamma' \times z_{bL}}{\gamma_w \times h_o} \text{ same as } FS_g = \frac{I_{cr}}{I_e} \quad \text{(B-1)}$$

where

FS_g = factor of safety relative to seepage gradient

γ' = effective unit weight of soil (or average effective unit weight of soil)

z_{bL} = land-side blanket thickness

γ_w = unit weight of water

h_o = excess head (above hydrostatic) at toe

I_{cr} = critical exit gradient = γ'/γ_w

I_e = exit gradient

(b) For stratigraphy conditions where a coarse stratum exists, and a surficial blanket is not present, the minimum required safety factors for seepage are based on the calculated vertical component of the exit gradient I_e in Equation B-1. The horizontal component of the exit gradient is addressed in the slope stability analyses as described in paragraph B-4e(2).

(4) Water levels for evaluation. The following water levels are to be evaluated:

(a) The top of wall.

(b) Other levels below the top of the wall identified as necessary for the project.

(5) Minimum factors of safety. Minimum factors of safety for evaluation of I-walls for the annual chance exceedance of the water levels in (5) above are shown in Table B-1. The annual chance of exceedance of the water level being evaluated shall be used to determine the minimum factor of safety in the table.

Table B-1. Minimum Factors of Safety

	Rotational Stability		Global Stability		Seepage	
Annual Chance of Exceedance	Well Defined	Ordinary	Well Defined	Ordinary	Well Defined	Ordinary
10% and greater	1.7	2.0	1.6	1.8	2.0	2.8
1%	1.5	1.7	1.5	1.7	1.6	2.0
0.2%	1.3	1.3	1.4	1.6	1.3	1.6
0.1% and below	1.1	1.1	1.3	1.4	1.3	1.6

Note:
Interpolation between points in the table using the log of the annual chance of exceedance is permitted, but extrapolation is not. For instance, a wall evaluated at the top of wall of 0.33% annual chance of exceedance and Ordinary project information would be evaluated for rotational stability with a minimum permitted factor of safety of 1.42. However, if the water level at the top of a wall had a chance of exceedance of 0.08%, the minimum permitted rotational factor of safety would be 1.1, and not extrapolated beyond.

(6) Other loads. Wave and impact loads may be of concern for inland levee systems for the rotational failure mode only. If relevant, they should be included in the evaluation.

Evaluation of the wall for the case with water to the top of the wall negates the need to include waves in global stability analyses.

(a) Waves. Where significant wave loads are present with significant chance of occurring concurrently with a flood event, the mean wave force for the event evaluated shall be added to the hydrostatic water pressure.

(b) Impact loads. In most cases, river flow is parallel to the face of I-walls, and significant loads from impact are unlikely during the period of a flood event. In rare cases, impact from free barges may be possible during a flood event because of wall and river geometry or where I-walls are adjacent to water bodies with sufficient width for wind-driven vessels to impact a floodwall. Design impact loads have not been developed for I-walls. In cases where impact loads from vessels are of concern, consult with CECW-CE.

b. Deformation Performance Requirements.

(1) Besides providing assurance against a breach, control of deformation is also required. Research performed for Phase III has shown that there are situations where I-walls with high rotational factors of safety may still have unacceptable deflections, and increasing the depth of the sheet pile cannot significantly reduce this. For this reason, factors of safety from a limit equilibrium analysis cannot be relied on alone to control deflections or guarantee resilience, and additional performance requirements are provided to evaluate this aspect of I-wall performance. Critical deformations in this ETL are defined by significant plastic deformation of the resisting side soils. The effect of deflection at the top of the wall on water stops and adjacent structures should be assessed by the engineer based on the limits in this guidance.

(2) As I-walls reach the limit state, deflection does exponentially increase as shown in Figure B-2. However, conditions can exist where excessive deflections may be experienced even though the soil strengths have not been completely mobilized along the sheet pile or the stability limit state reached. In Figure B-3, results for deformation at the ground surface are shown for an I-wall in sand at the St. Paul, MN, project that was analyzed with varying pile tip depths. Identical deformations are seen for piles with differing tip depths for many water levels. Figure B-4 shows the relationship between factor of safety and deflection from these analyses. The amount of deformation in these cases depends on the stiffness of the soil, the geometry of the section, and the amount of load. Therefore, performance requirements for deflections are partially incorporated into the factors of safety for rotational stability, but high factors of safety do not always indicate low deformation. Because a high factor of safety cannot guarantee low deformation, guidance is provided in this ETL for evaluating I-walls to consider deformation.

(3) Different performance requirements are provided for consideration of deflection depending on the frequency of flood event. Loads with higher chance of occurrence should not result in significant amount of inelastic soil deformation or damage to the structure that would require rehabilitation or replacement of a project (behavior should be resilient). For extremely low probability flood events, deflection resulting in some permanent displacement of the wall or minor structural damage may be permitted. Designing for deflections can be performed for new structures for relatively little extra cost. But as this ETL is for existing structures for which

rehabilitation is likely to be very costly, some chance of permanent deflection has been permitted for low probability events, provided the likelihood of breach remains low.

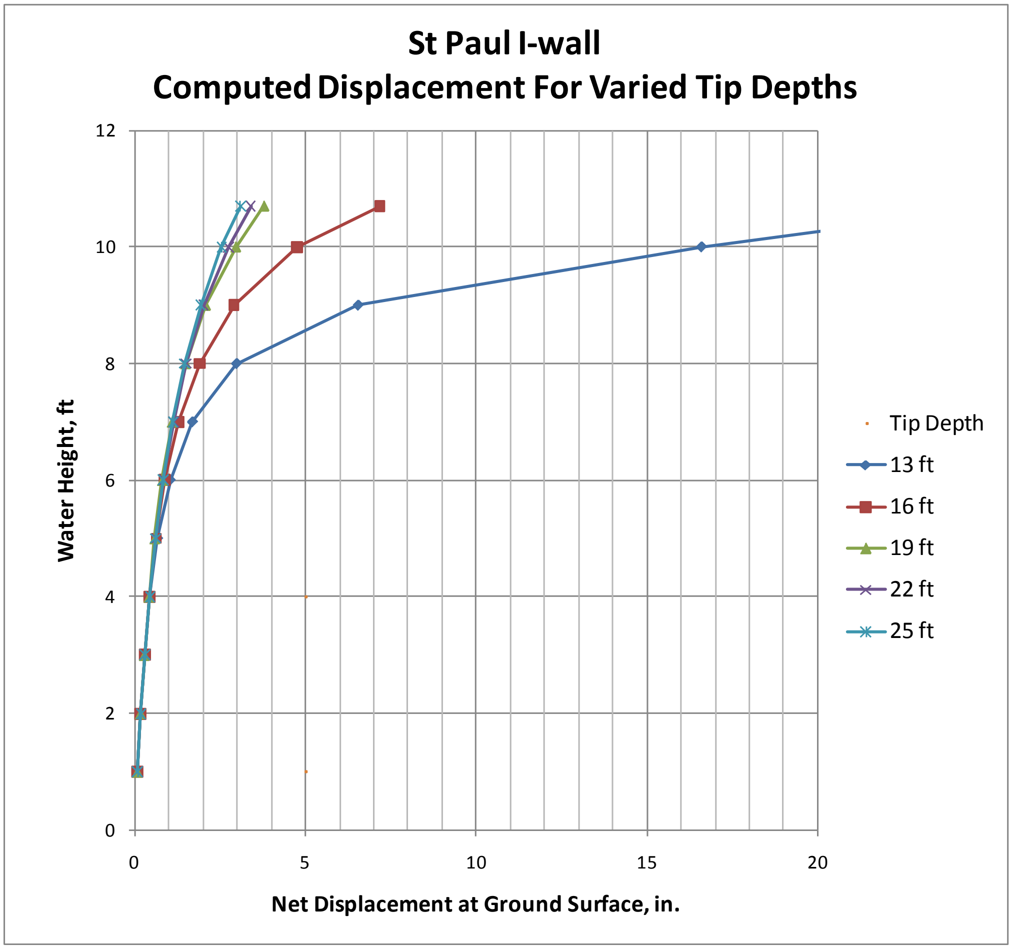

Figure B-3. Computed displacement for varied tip depths, St. Paul I-wall

(4) Both numerical analysis and full scale I-wall tests show that almost any amount of deformation of an I-wall under load will result in some amount of permanent deflection because of unrecoverable displacements of the soil as the wall rotates into the upper soils on the resisting side of the wall. The most important aspect for resilience and deflection control is therefore deformation of the soil at the ground surface. However, because of the variable nature of soil and difficulty in measuring and calculating load deflection characteristics, there is no reliable method of precisely determining deflection of I-walls. The criteria presented herein were developed using probabilistic with uncertainty analyses to compute the chance that a limit deflection is exceeded for walls in sand, soft clay, and stiff clay. For the water level with 1.0 percent chance of exceedance, the limit deformation to provide resilient behavior is 1.5 in. (38 mm) at the ground surface. For the 0.2 percent chance of exceedance, the limit deformation

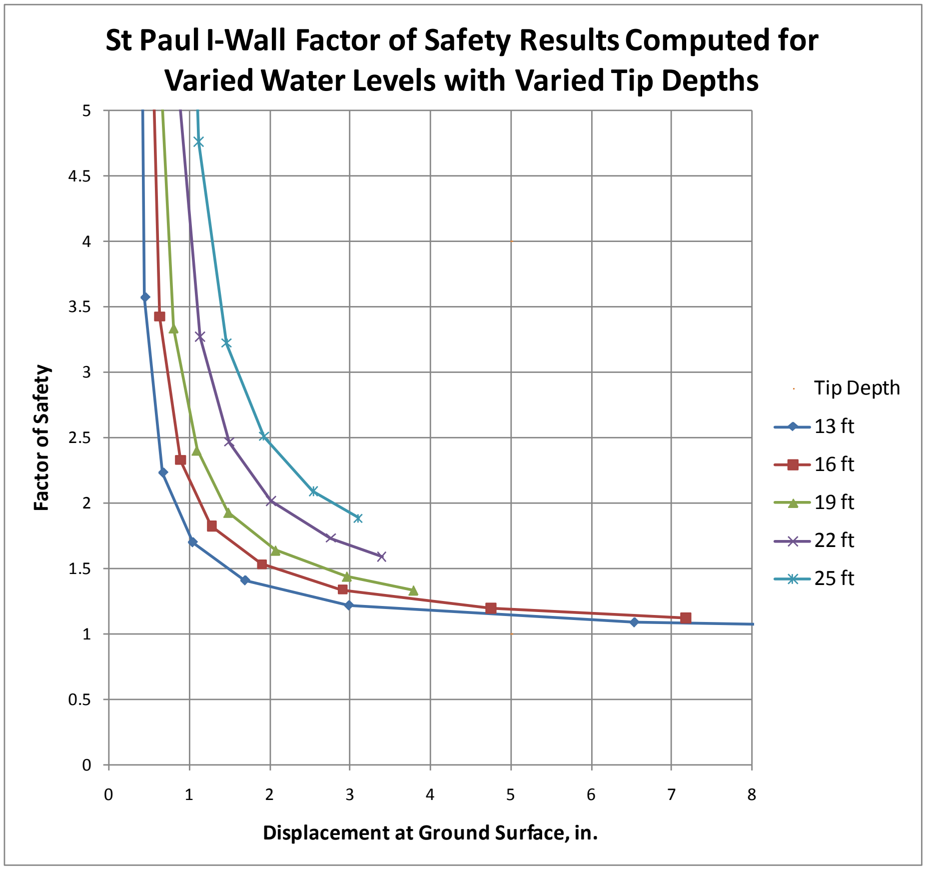

Figure B-4. Relationship between factor of safety and deflection

to allow some movement but not collapse was 3 in. (76 mm). For the 0.1 percent chance of exceedance, the target deformation to prevent collapse was 6 in. (152 mm). These deformation limits were used to develop the criteria in Table B-2.

(5) Water levels for deformation evaluation. For evaluation of deformation, the water heights should be compared with water level for the event of interest. Water height should be evaluated at least at the following levels:

(a) The top of wall.

(b) Other water levels identified as necessary for the project.

(6) Maximum water heights for the annual chance exceedance of the water levels in (5)(a) and (b) above are shown in Table B-2.

(7) I-walls with evaluation water levels not meeting the requirements of Table B-2 shall be evaluated according to the requirements in paragraph B-6. The annual chance of exceedance of the water level being evaluated shall be used to determine the minimum factor of safety in the table.

Table B-2. Maximum Water Heights (in feet (meters)) for Deformation Control

Annual Chance of Exceedance	Foundation Type			
	Sand $\varphi \geq 32.5$, $D_r = 0.50$	Soft Clay $S_u \leq 300$ psf (14.4 kPa)	Stiff Clay $S_u \geq 1{,}500$ psf (71.8 kPa)	I-wall on Levee
1% and above	7 (2.1)	5 (1.5)	8 (2.4)	4 (1.2)
0.2%	9 (2.7)	7 (2.1)	12 (3.7)	4 (1.2)
0.1% and below	11 (3.4)	8 (2.4)	15 (4.6)	4 (1.2)

Notes:

(a) D_r = relative density

(b) The heights in the table are the distance from the elevation of the water flood level to the ground surface on the land side of the wall, as shown in Figure B-5.

(c) Interpolation between points in the table using the log of the annual chance of exceedance is permitted, and linear interpolation is permitted for clay strengths between those shown, but extrapolation is not permitted.

(d) Limited data has been developed for deformation of I-walls on levees. I-walls on levees with water height greater than 4 ft (1.2 m) as shown in Figure B-5 require further evaluation as described in paragraph B-6.

(e) For layered soils, the engineer shall base the maximum water height on a soil height with the predominant soil type resulting in the lowest water height for deformation control. The soil type used shall be controlled by the soil on the land side of the wall extending from the ground surface to elevation of maximum net passive pressure from the CWALSHT analysis. Questionable sections will require further evaluation as described in paragraph B-6.

(f) I-walls founded in soils with properties significantly outside the range of soils in the table shall be evaluated as described in paragraph B-6.

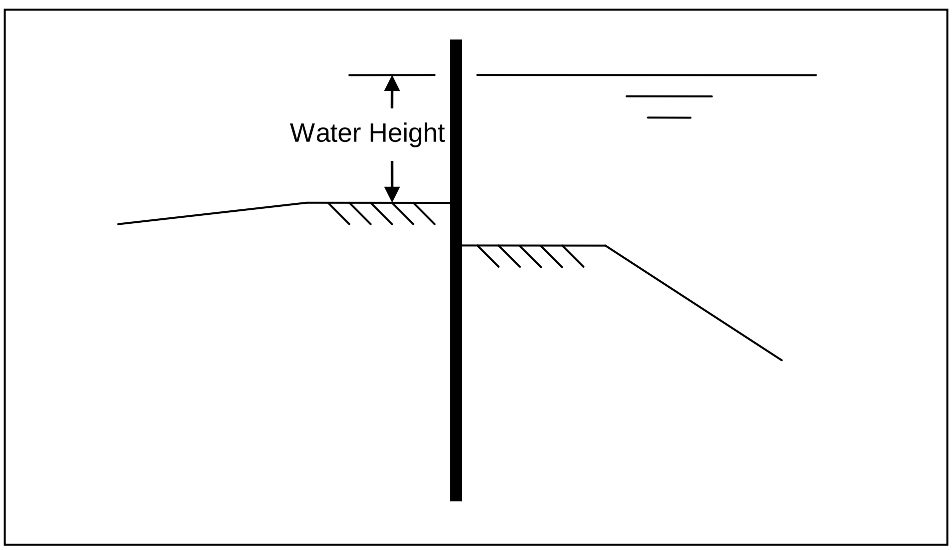

Figure B-5. Definition of water height for Table B-2

c. Structural Criteria. Evaluation of structural components will be performed according to guidance in EM 1110-2-2504.

d. Overtopping Scour Protection. Overtopping is a concern where a section of I-wall will overtop before the landside of the system is inundated. Overtopping is also a concern in locations where waves are significant. Erosion of the soil on the landside of the wall could lead to collapse of the I-wall and to inundation rates that could create a life safety concern. The acceptability or design of a scour protection system is beyond the scope of this ETL. Sections of existing flood risk reduction systems evaluated and found to overtop before the landside is inundated and also to have inadequate scour protection shall be reported to CECW-CE.

B-6. Additional Evaluation. If the I-walls being evaluated do not meet the requirements of paragraph B-5, then the following supplemental data and actions may be performed:

a. A local load test may be performed for each representative reach of I-wall. Local load testing shall include a minimum of three adjacent monoliths that are isolated within a cofferdam and are subjected to full water pressure to the top of the I-wall. Instrumentation shall measure the deflections at the top and at the ground surface of each monolith. Soil borings on the flood side and landside of the wall should be used to characterize the test site and to estimate soil properties. Calibrate a CWALSSI or finite element model (as described in b below) to the deflections measured in the local load test. Use the calibrated models to predict behavior for other applicable overloading conditions. Perform global stability and seepage analyses using parameters calibrated to test results. The review of potential load test plans and instrumentation programs shall be coordinated with CECW-CE.

b. Numerical analysis with a finite element or finite difference program capable of soil-structure interaction may be performed by personnel experienced with the computer program, the soil constitutive model used by the program, modeling of soil-structural interaction problems with proper interface elements, and techniques needed to include the gap on the flood side of the wall. Research for the work in this ETL was performed with the programs FLAC and Plaxis. Inclusion of the concrete cap has been shown to improve stability in these analyses for I-walls in granular soils analyzed using drained soil properties, but settlement of soil under the cap should be considered. Sensitivity analyses shall be performed to determine the effects of all parameters on the results. The model should be validated against known stress states and, if possible, calibrated against known data. Review of the model and results shall be coordinated with CECW-CE. Deflections should be plotted for each foot of water level against the wall. Deflection of the wall at the ground surface should be no more than 1.5 in. (38 mm) at the ground surface for the water surface corresponding to the 1 percent chance of exceedance; 3 in. (76 mm) for the 0.2 percent chance of exceedance; and 6 in. (152 mm) for the 0.1 percent chance of exceedance.

c. Additional foundation information may be obtained to better define stratigraphy or critical soil parameters. Conservative assumptions may result in inadequate factors of safety whereas gathering additional subsurface information may lead to meeting criteria.

APPENDIX C
STABILITY ANALYSIS OF I-WALLS CONTAINING GAPS BETWEEN THE I-WALL AND BACKFILL SOILS

C-1. Organization of Appendix. This appendix describes the procedure for performing the global or deep-seated stability analysis for I-walls where flood-side gaps may develop. The analysis procedure is described in Section I and four example problems are presented in Section II (full-depth gap analysis of I-wall in cohesive soil; partial gap analysis of I-wall in uniform embankment and foundation; partial gap analysis of I-wall in layered cohesive soils; partial gap analysis of I-wall with sheet pile intersecting a cohesionless layer).

Section I
Analysis Procedure

C-2. Source of the Analysis Procedure Reported in This Appendix. The analysis procedure described in this appendix was developed as part of the efforts of the Interagency Performance Evaluation Taskforce (IPET) formed in response to Hurricane Katrina. The method was developed by Prof. Thomas L. Brandon and Prof. Emeritus J. Michael Duncan, Virginia Tech, Blacksburg, VA; Prof. Emeritus Stephen G. Wright, The University of Texas at Austin; Ronald E. Wahl, Geotechnical and Structures Laboratory, U.S. Army Engineer Research and Development Center, Vicksburg, MS; and Noah D. Vroman, U.S. Army Engineer District, Vicksburg. Prior to preparation of this ETL, the analysis procedure was described in an unpublished report by the Geotechnical Criteria and Applications Team (GCAT) where additional examples were added to the ones originally provided in the paper by Brandon et al. (2008), and text was added to assist in the implementation of this method into current practice. The unpublished GCAT report, in its entirety, was edited and formatted for inclusion in this appendix and is therefore an edited version of the paper by Brandon et al. (2008), included with permission of the authors.

C-3. Introduction.

a. Following Hurricane Katrina an extensive investigation of the performance of floodwalls in the New Orleans area was undertaken by the U. S. Army Corps of Engineers and others. This investigation included detailed study of failures of cantilevered sheet-pile I-walls during the hurricane. An important lesson from this investigation was that gaps can form on the flood side of I-walls as the water rises, causing the I-wall to deflect. Once formed, these gaps filled with water, resulting in significantly higher loads on the walls. Gap formation was a key factor in several I-wall failures, most notably the breach at the 17th Street Canal and the two breaches at the London Avenue Canal as described in the paper by Duncan et al. (2008). Modeling such gaps correctly is clearly an important aspect of analyzing I-wall stability. The gap analysis is only one of many potential failure modes of an I-wall. Evaluation of an I-wall should address all potential failure modes in addition to the analysis outlined in this Appendix.

b. Observations made during field reconnaissance after Hurricane Katrina indicated that gaps formed at several I-wall sections caused by the canal water loads. Figure C-1 shows a gap behind the Michoud Canal I-wall. Although a gap formed behind the Michoud Canal wall, the wall did not fail. As shown in the figure, it is common for the exposed portions of the sheet pile to be encased in reinforced concrete.

Figure C-1. Gap on the canal side of the Michoud Canal I-wall in New Orleans after Hurricane Katrina

c. Because of the important effect of gaps on I-wall stability, study of gap formation became a central element of the IPET investigation. The IPET investigation included physical modeling (centrifuge tests), soil-structure interaction analyses, and limit equilibrium analyses. Brandon et al. (2008) summarizes the study of gap formation of the IPET. The studies showed the following:

(1) Formation of a gap will likely reduce the factor of safety for I-wall stability.

(2) Although the factor of safety decreases when a gap forms, it does not always decrease to a value less than 1.0.

(3) Because evaluation of soil stiffness is difficult, and because soil stiffness is highly variable, it is not possible to predict with a high degree of reliability whether or not a gap will form at a particular location behind an I-wall. The IPET investigation showed that gaps formed at some locations, but gaps did not form in adjacent, seemingly identical locations.

(4) Given the difficulty of predicting with confidence whether or not a gap will form, it should always be assumed that a gap has the potential to form when an I-wall is loaded by water above the top of the levee in which it is embedded.

d. These guidelines outline an analysis procedure for I-walls founded in both cohesive soils and in cohesionless soils overlain by cohesive soils. This appendix describes simple procedures for estimating the depths of gaps behind I-walls, for calculating the loads to which they are subjected, and for including them in stability analyses. Four example problems demonstrating how to perform a full and partial gap analysis are given in paragraphs C-8 through C-11.

e. The procedure for incorporating a gap into a stability analysis consists of three basic parts, which are described in this appendix:

(1) Determine the depth of the gap.

(2) Determine the resultant force acting on the wall. This will consist of water pressure if the gap extends to the tip of the wall, or a combination of water pressure and earth pressure if the bottom of the gap occurs above the tip of the sheet pile.

(3) Perform a limit equilibrium analysis. In the case of a gap extending to the tip of the sheet pile wall, this water pressure on the wall may be handled as a tension crack within the program. For gaps terminating prior to the tip of the sheet pile, manually inputting line loads or distributed loads will be required.

C-4. How Gaps Form in Uniform Cohesive Soils.

a. Gaps can form only in cohesive soils because they have the ability to sustain a gap. When an I-wall is loaded by water and deflects away from the soil on the flood side, the lateral stress in the soil on that side decreases. The lowest possible lateral stress in the soil is the minimum active earth pressure. Thus, if the minimum active earth pressure is lower than the water pressure, a gap can form and remain open. If the minimum active earth pressure is higher than the water pressure, a gap cannot remain open. This analysis assumes that the wall will displace or rotate sufficiently such that active earth pressures will be developed. Thus, a gap will extend to a depth where the active earth pressures and the water pressures are equal. Field conditions may vary from assumptions made resulting in shallower or deeper gap depths than computed following the procedure described in paragraph C-5. The selection of conservative undrained shear strengths will result in shallower gap depths, correspondingly higher lateral loads on the wall below the gap and lower computed factors of safety.

b. A mandatory first step in accounting for the presence of a gap behind the I-wall during the stability analysis is to determine the depth of the gap. The depth of the gap in comparison with the depth of the sheet-pile penetration will determine whether the stability analysis is for *full* or *partial gap* conditions. This will be explained in the following paragraphs.

C-5. Determining the Depth of the Gap. The gap depth can be estimated by two different methods. In the first method, which is applicable for cases where the embankment and the foundation are uniform and homogeneous, the depth can be computed directly using a formula. The second method applies to cases where the gap forms in layered cohesive soils. The second method is the more general of the two and will be used most often in these types of analyses.

These two procedures are described in the following paragraphs. An assumption applied in both procedures is that a horizontal ground surface exists when applying lateral earth pressure theory. Where sloping ground surface conditions exist the reduction in weight of the active wedge may be all or partially offset by the reduction in shear resistance due to the shorter failure surface.

a. Case 1: Gaps in Uniform Cohesive Soils.

(1) Equations C-1 and C-2 are expressions presented by Brandon et al. (2008) for computing the active earth pressure and water pressure at any depth for an I-wall constructed in a homogeneous levee and foundation, such as shown in Figure C-2.

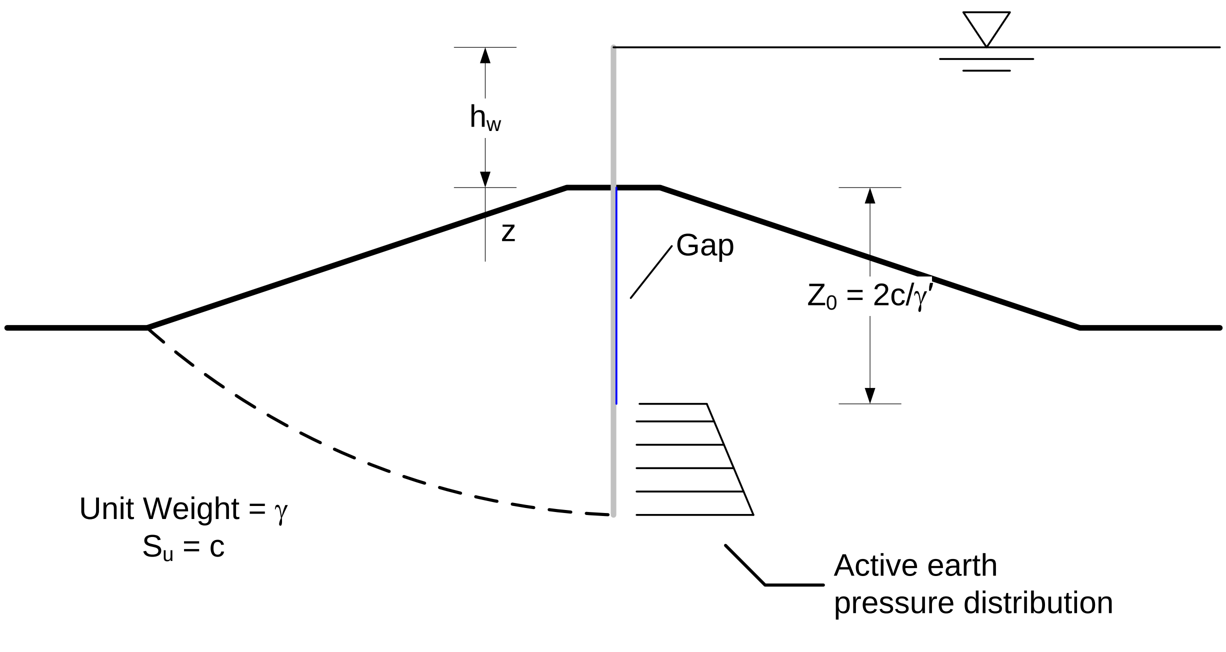

Figure C-2. Gap formation for I-wall for uniform embankment and foundation soil properties

(a) Active earth pressure

$$\sigma_{ha} = \sigma_{v0} - 2c = \gamma z + \gamma_w h_w - 2c \qquad \text{(C-1)}$$

where

σ_{ha} = active horizontal earth pressure

σ_{v0} = vertical earth pressure

c = undrained soil strength of uniform soil deposit

γ = unit weight of soil

z = depth below ground surface

γ_w = unit weight of water

h_w = height of water above flood-side ground surface

(b) Water pressure

$$u = \gamma_w(h_w + z) \tag{C-2}$$

where u is the hydrostatic water pressure in gap.

(2) By equating Equations C-1 and C-2 and solving for the depth, the depth of the gap z_o, where the water pressure and the active earth pressures are equal, can be determined as expressed in Equation C-3:

$$z_0 = \frac{2c}{\gamma'} \tag{C-3}$$

where γ' is the buoyant unit weight of soil.

(3) If the depth of the gap z_o exceeds the depth of the tip of the sheet pile, then a *full-gap* analysis should be performed. If the depth of the gap is less than the sheet pile tip depth, a *partial gap* analysis should be performed.

(4) For the full-gap analysis, water pressure is assumed to act on the sheet pile from the static water level (flood side water level) to the tip of the sheet pile. Example 1 (paragraph C-7) shows how to perform a full-gap analysis for an I-wall with uniform embankment and foundation soil properties.

(5) For the partial gap analysis, the failure mechanism presumes that active earth pressures are acting between the bottom of the gap and the tip of the sheet pile as shown in Figure C-2. Example 2 (paragraph C-9) shows how to perform a partial gap analysis for an I-wall with uniform embankment and foundation soil properties.

(6) Engineering judgment is necessary to determine if a given soil layer is of sufficient thickness and lateral extent to contribute substantially to the passive resistance at the tip of the sheet pile. In other words, if a sheet-pile tip penetrates into a firm layer for only a few feet, it may be prudent to neglect the presence of the firm layer in the analysis.

b. Case 2: Gaps in Layered Cohesive Soils.

(1) When the soil profile contains soil layers of differing unit weights and shear strengths, the possible depth of a gap can be determined as shown in Figures C-3 and C-4. The active horizontal earth pressure σ_{ha} at any depth z_i is calculated using the expression

$$\sigma_{ha} = \sum(\gamma_i z_i) + \gamma_w h_w - 2c_i \tag{C-4}$$

where

γ_i = unit weight of the i^{th} layer

z_i = thickness of the i^{th} layer

i = indicates the soil layer i

c_i = undrained shear strength of the i^{th} layer

(2) When the earth pressure is plotted along with the hydrostatic water pressure as a function of depth (or elevation), the depth of the gap can be determined as the location where the two are equal. For example, given the conditions shown in Figure C-3, the active horizontal earth pressures and the hydrostatic water pressure are computed using Equations C-4 and C-2 at the top and bottom of each layer. The values plotted in Figure C-4 show that the active horizontal earth pressure and the water pressure are equal at elevation -15.6 ft (4.75 m). Thus, a gap also would extend to the tip of the sheet pile (elevation -15 ft (4.5 m)).

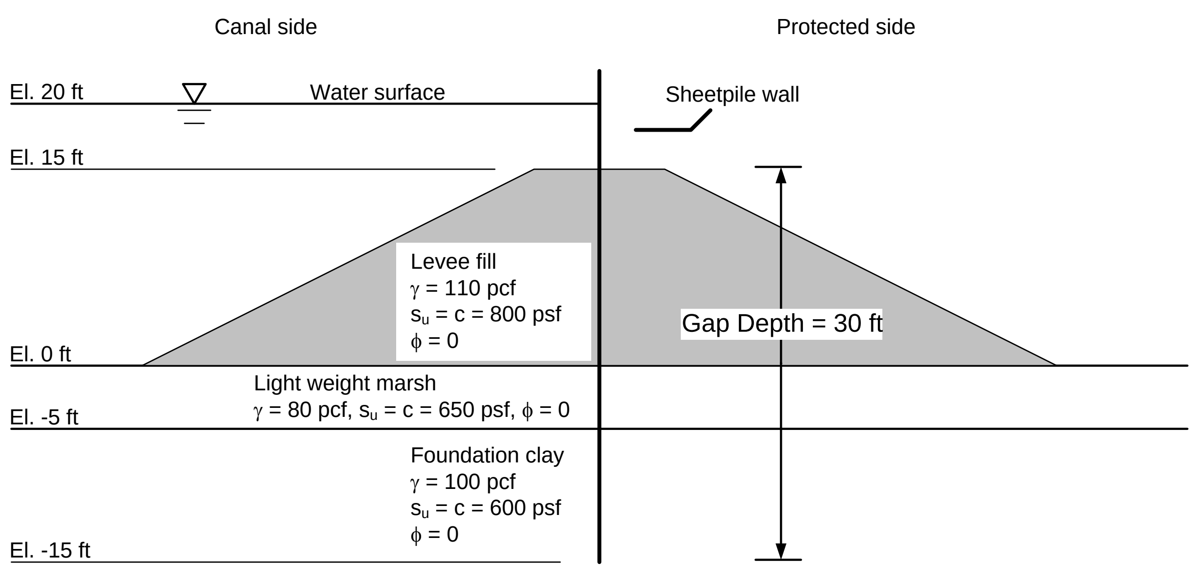

Figure C-3. I-wall in layered cohesive soils (note: to convert feet to meters, multiply by 0.3048; to convert pounds (mass) per cubic foot to kilograms per cubic meter, multiply by 16.02; to convert pounds (force) per square foot to kilopascals, multiply by 0.04788)

(3) In some situations, the calculations may indicate that the water pressure exceeds the active horizontal earth pressure for some distance below the bottom of the wall. For these cases, it would not be expected that the gap would extend below the bottom of the wall. Movement of the wall would reduce the earth pressure acting on it, but it would not be expected that the earth pressures in the soil below the wall would be similarly reduced. As a result, it seems logical that a gap would form and stay open only to the bottom of the wall. Because the hydrostatic

pressures are greater than the active earth pressures over the full depth of the gap, the formation of a gap increases the load on the wall, and makes the wall less stable.

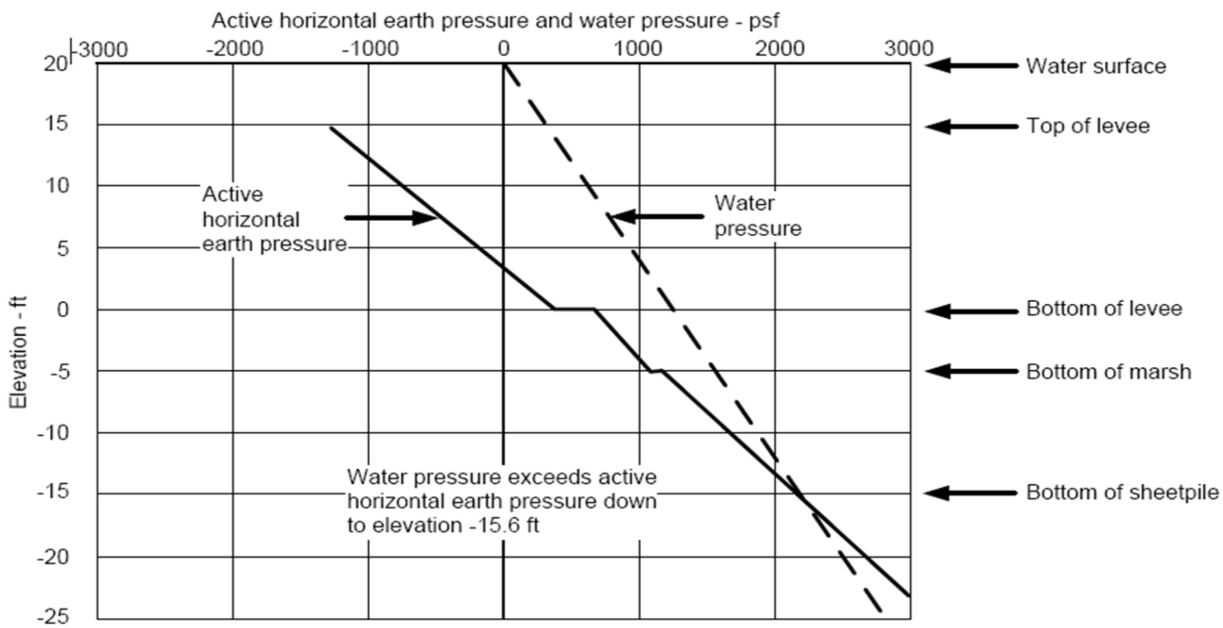

Figure C-4. Pressure distributions against sheet-pile I-wall of Figure C-3 (note: to convert feet to meters, multiply by 0.3048; to convert pounds (force) per square foot to kilopascals, multiply by 0.04788)

(4) Example 3 (paragraph C-10) shows how to perform a partial gap analysis for an I-wall with uniform embankment and layered foundation soil properties.

c. Case 3: Gaps in Layered Soils Where I-Wall Terminates in a Sand Layer.

(1) The calculated gap depth for an I-wall that penetrates into a cohesionless soil will never extend to the bottom of the wall. This is because the sand layer, having no cohesion, lacks the ability to sustain a vertical face in the submerged state. Figure C-5 illustrates a situation where the wall penetrates into a cohesionless layer. Below the top of the sand layer in Figure C-5, the total horizontal pressure is equal to the water pressure plus the effective horizontal earth pressure, and therefore cannot be less than the water pressure. The water pressure and total horizontal earth pressure for this case are shown in Figure C-6. It can be seen that the water pressure exceeds the active horizontal earth pressure down to the top of the sand layer. Therefore, since the sand is cohesionless and cannot sustain a gap, the bottom of the gap is at the same elevation as the top of the sand layer.

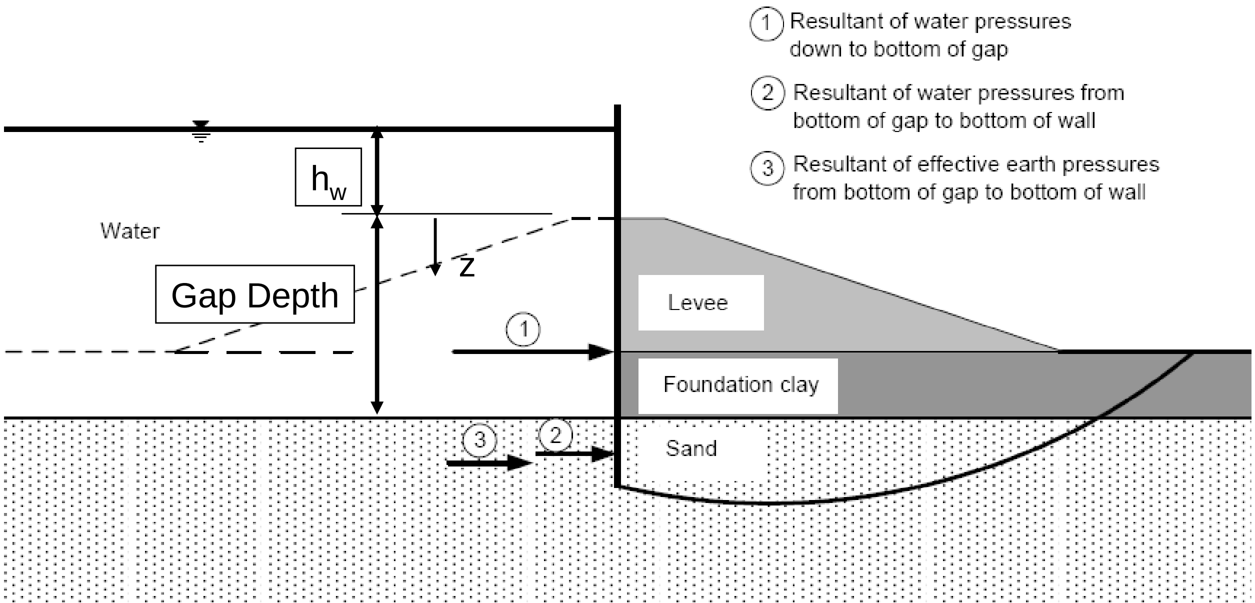

Figure C-5. Situation for I-wall penetrating sand layer showing gap depth and resultant forces acting on sheet-pile wall

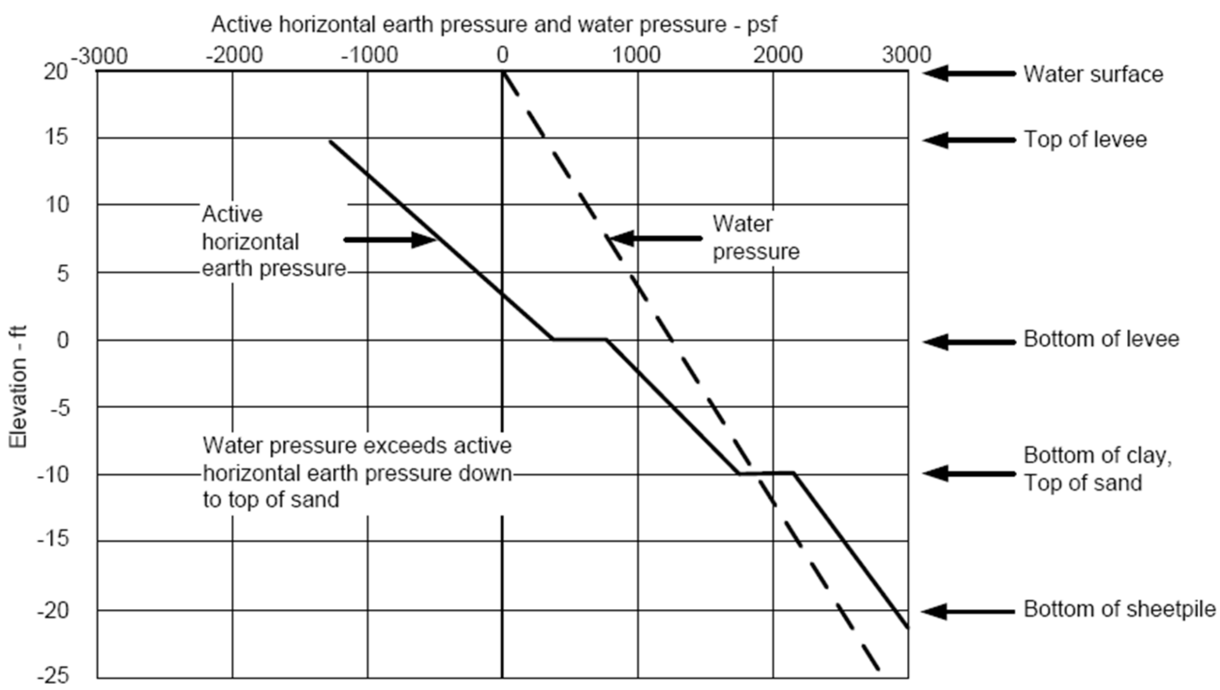

Figure C-6. Pressure distributions with sheet pile penetrating into a sand layer (note: to convert feet to meters, multiply by 0.3048; to convert pounds (force) per square foot to kilopascals, multiply by 0.04788)

(2) The total horizontal pressure σ_h at a given depth in the sand layer can be computed using Equation C-5:

$$\sigma_h = \sigma_v' K_a + u \tag{C-5}$$

where

σ'_v = effective vertical stress

K_a = active earth pressure coefficient = $\tan^2(45 - \phi/2)$

u = pore water pressure acting along the sheet pile in the sand layer

(3) It should be noted that the pore pressure along the wall within the sand is not hydrostatic because of head losses that occur in the sand. The pore pressures along the sheet pile should be calculated using a finite element seepage analysis, and these pore pressures should be used in calculating the effective vertical stress in Equation C-5. Example 4 (paragraph C-11) shows how to perform a partial gap analysis for an I-wall penetrating into a sand layer.

(4) If a silt layer is present in the strata, and effective stress or drained strength parameters are used, the same basic procedure outlined for sands is used. If the silt is assigned an effective stress cohesion c', the total horizontal pressure within the silt layer is calculated using Equation C-6:

$$\sigma_h = \sigma_v' K_a - 2c'\sqrt{K_a} + u \tag{C-6}$$

C-6. Modeling Gaps in Slope Stability Calculations.

a. Full-depth Gaps in Cohesive Soils.

(1) A full-gap analysis case exists when the depth of the computed gap equals or exceeds the penetration depth of the sheet-pile wall. When the gap extends fully to the bottom of the wall, an acceptable way to model the condition is by removing the soil on the flood side of the wall, and representing the water load as a distributed pressure as shown in Figure C-7. Alternatively, the water load can be represented as a concentrated force (line load) equal to the resultant of the triangular pressure distribution shown in Figure C-7.

(2) Some slope stability computer programs allow incorporation of a vertical tension crack, filled with water to some level. To model conditions with a full gap correctly, the software should automatically exclude the soil beyond the gap. An appropriate force due to the hydrostatic water pressures in the gap should be calculated by the computer program and applied to the vertical boundary. The tension crack method, if properly applied, will give the same factor of safety as the flood side soil removal method.

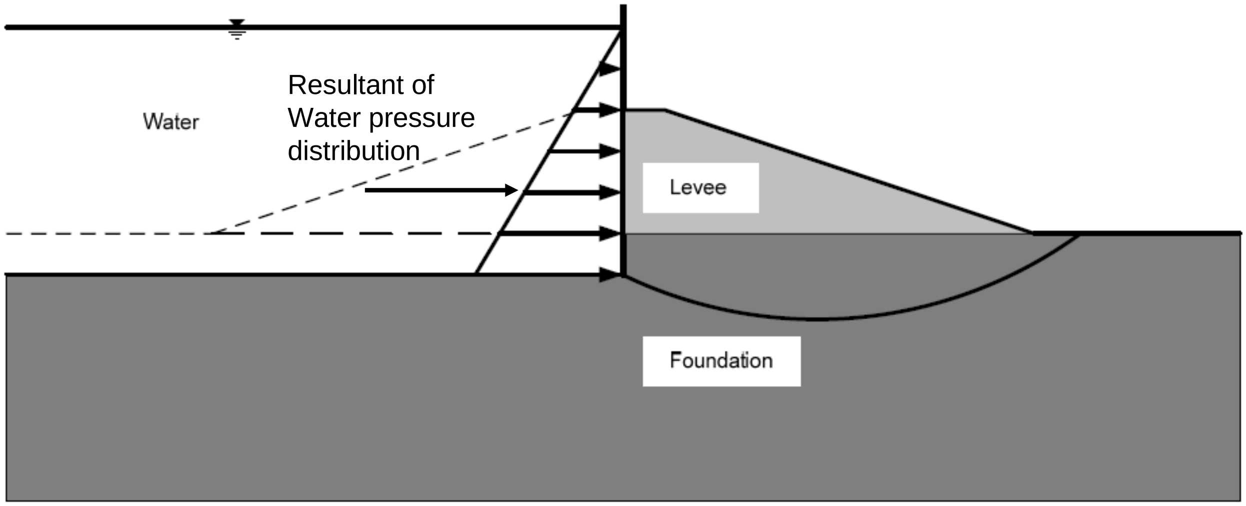

Figure C-7. Modeling a full-depth gap in a slope stability analysis

(3) In the course of the analyses that were performed by the IPET Geotechnical Team for the New Orleans flood control structures, it was found that three widely used computer programs (UTEXAS4, SLOPE/W, and Slide) did not properly model a water-filled crack when the water level in the crack was above the ground surface. All three software vendors have reportedly corrected the errors in the programs, but it is important that the user check to make sure that the change has been incorporated into the version of the program in use. It is strongly recommended that spreadsheet calculations be used to check results to ensure that computer programs are modeling the water-filled gap correctly. The forces acting on the slice that comprises the sheet-pile wall should be examined to ensure that the correct water pressure force is being applied. If the “soil removal” technique is used to model the gap, as in the examples in this appendix, the tension crack feature of the programs is not used.

(4) When a full-depth gap is analyzed, the use of the water-filled tension crack feature in the slope stability program should provide correct factors of safety for both gap and no-gap global failure conditions (slip surface below the sheet pile). If the soil removal technique is used, only the factor of safety for the gap analysis is correct. The global factor of safety for the no-gap analysis would not be correct owing to incorrect stresses on the flood side of the I-wall.

b. Partially Penetrating Gaps in Cohesive Soils.

(1) A partial gap analysis must be performed in those instances where the calculated depth of a gap does not extend to the bottom of the wall. This condition can occur when cohesive soil behind the wall has insufficient strength to keep the gap open, or when the wall is driven into cohesionless soil, which lacks the ability to sustain a gap. The partial gap analysis is performed in a manner similar to that used for full-depth gaps, except an additional force or forces need to be applied to the wall. In the case of a partial gap analysis in cohesive soil, the additional force is the resultant of the active earth pressure that acts between the bottom of the gap and the tip of the sheet pile.

(2) The analysis can be performed by the flood side soil removal method described in a above for full gaps. *Under no circumstances should the slip surface pass through the sheet pile.* The soil should be removed from the levee crest down to the tip of the sheet pile, and the water load should be applied as a distributed load (triangular pressure distribution) against the sheet pile from the water level elevation to the elevation of the sheet-pile tip. The values for active earth pressure are computed at the elevations of the bottom of the crack and the sheet-pile tip using Equation C-1. This results in a trapezoidal-shaped pressure distribution over the depth interval between the gap bottom and the sheet-pile tip. The resultant force and vertical centroid of this distribution are then computed.

(3) The slope stability analysis is performed by removing the soil down to the elevation of the sheet-pile tip (*not to the elevation of the bottom of the gap*). The forces or pressures against the sheet-pile wall are applied as shown in Figure C-8. A triangular water pressure distribution is applied along the sheet pile from the elevation of the water surface to the bottom of the gap. Depending on the features of the software package used, this may be applied as a pressure distribution, or the calculated resultant of the pressure distribution may be applied as a line load. Also, the trapezoidal earth pressure distribution is applied from the bottom of the gap to the tip of the sheet pile. Depending on the features of the program, this may be applied as a pressure distribution, or the resultant of the pressure distribution may be applied as a force (line load) at the correct point of application (vertical location of the centroid of the trapezoidal pressure distribution).

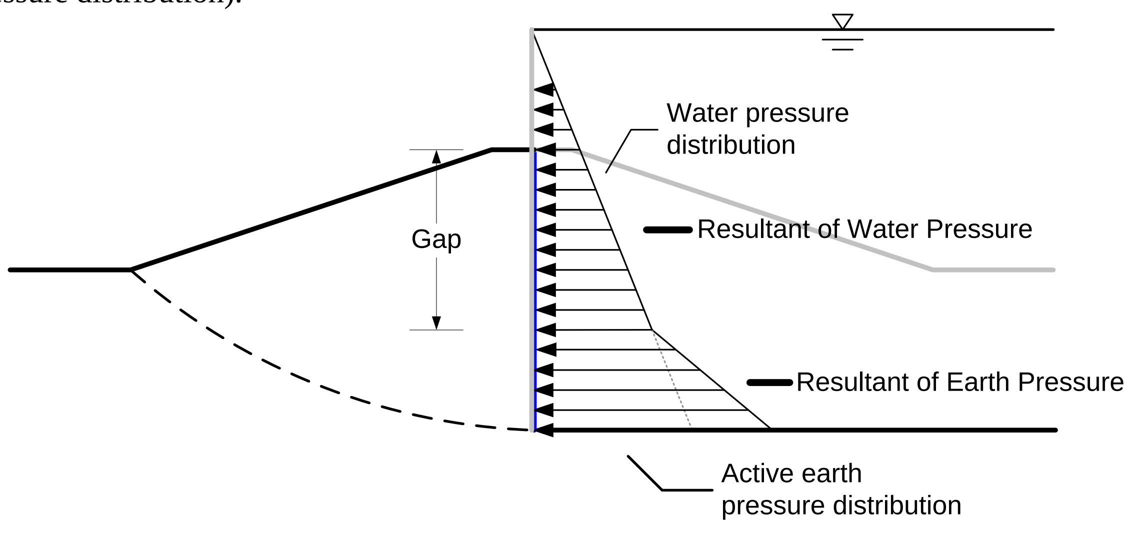

Figure C-8. Illustration of "soil removal" method for partial gap conditions for slope stability analysis

(4) An alternative method of performing the partial gap analysis in cohesive soils is to use the tension crack feature contained in most slope stability programs. A water-filled tension crack can be incorporated into the analysis extending to the tip of the sheet pile. The water elevation in the tension crack should be set as the canal water level, as done in the full-gap analysis. An additional force, representing the difference in the earth pressure and the water pressure from the bottom of the gap to the tip of the sheet pile, must be applied at the correct point of application. This method should result in the same factor of safety as the soil removal method described in

(3) above. The soil removal technique does not depend on knowledge of the specific tension crack features of the various slope stability programs, and its use is recommended.

(5) The critical slip surface should have an entry point at the tip of the sheet pile. *Under no circumstances should an analysis be made where the slip surface intersects the sheet pile at the bottom of the gap*. A two-step approach can be used to determine the factor of safety of the most critical slip surface. In the first step, a critical slip circle is determined from a circular search method. In the second step, a critical noncircular slip surface can be determined from an optimization procedure that is initiated with points along the critical slip circle from the first step.

c. Partially Penetrating Gaps in Cohesionless Soils.

(1) The partial gap analysis for sheet piles driven into a sand layer is similar to the analysis described in b above, with the addition of a couple of important details. First, the water pressure distribution in the sand layer along the sheet pile is not hydrostatic. Head losses occur in the sand layer as the water flows from the tip of the gap underneath the sheet pile, and this reduced water pressure should be accounted for in the analysis. Second, this reduced water pressure will also affect the vertical effective stress calculated along the wall, and thus the active earth pressures, and this should be accounted for as well.

(2) It would be expected that a partial gap stability analysis in a cohesionless soil would incorporate drained strength parameters in the sand and pore pressures calculated from a finite element seepage analysis. The formation of a gap would allow a hydraulic connection between the flood side water level and the sand layer and should be incorporated into the seepage analysis. The pore pressures acting on the portion of the sheet pile embedded in the sand can be determined based on the results of the finite element seepage analysis.[1] These pore pressures should be used in the calculation of the vertical effective stress in the sand and the resulting active earth pressures.

(3) Shown in Figure C-9 are the pressures that would be applied to a wall in the partial gap case for a cohesionless soil. The same basic techniques that are used for partial gaps in cohesive soils can be used for partial gaps in cohesionless soils. The soil on the flood side of the sheet pile can be removed from the cross section to a depth equal to the tip of the sheet pile (*not to the bottom of the gap*). A hydrostatic water pressure, calculated from the difference in elevation of the flood side water level and the bottom of the gap, is applied to the wall. A second water pressure (or resultant force) is applied to the portion of wall embedded in the sand. This water pressure force would be less than hydrostatic owing to the reduction in head as water flows through the sand. A third pressure distribution or resultant force would be applied to represent the active earth pressure acting on the portion of the wall embedded in sand. This active earth

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[1] In lieu of a finite element seepage analysis, an assumed phreatic surface may be used in the analysis. However, it would not be as accurate as methods accounting for the head loss in the sand. It can be either conservative or unconservative, depending on the assumed position of the phreatic surface.

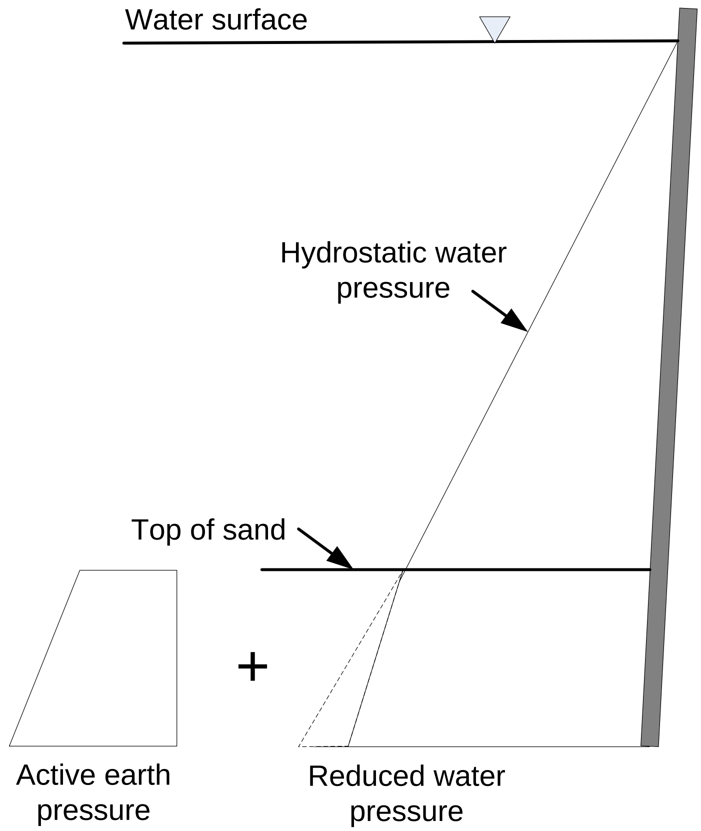


Figure C-9. Water pressure and earth pressure applied to an I-wall for a partial gap analysis in cohesionless soil

pressure distribution should be calculated using the reduced pore pressure distribution acting on the wall as opposed to a hydrostatic pore pressure distribution.

(4) The tension crack feature of slope stability software can also be used for partial gap analyses in cohesionless soils. The problem is modeled with a water-filled tension crack to the tip of the sheet pile. An additional force must be specified at the correct point of application representing the difference between the hydrostatic water pressure and the reduced water pressure (acting away from the wall) and the active earth pressure (acting toward the wall).

C-7. Summary. The depth of the gap that can be sustained in cohesive soils can be calculated based on the total horizontal stress and the hydrostatic pressures that act on the sheet pile. For cases where the gap does not extend to the slip surface, the wall below the bottom of the gap is loaded by total earth pressures where the soil below the bottom of the gap is cohesive, and by effective earth pressures and water pressures where the soil below the bottom of the gap is cohesionless.

Section II
Sample Problems

C-8. Example 1: Full-Depth Gap Analysis of I-wall in Uniform Soil Conditions.

a. Problem: Evaluate the stability of the I-wall shown in Figure C-10.

b. Step 1: Determine depth of gap z_0 from Equation C-3[1]:

$$z_0 = (2c)/\gamma' = (2 \times 600 \text{ psf})/(100 \text{ pcf} - 62.4 \text{ pcf}) = 31.9 \text{ ft}$$

Full-gap analysis of the I-wall is required because z_0 is greater than the penetration depth of 30 ft of the sheet pile below the ground surface.

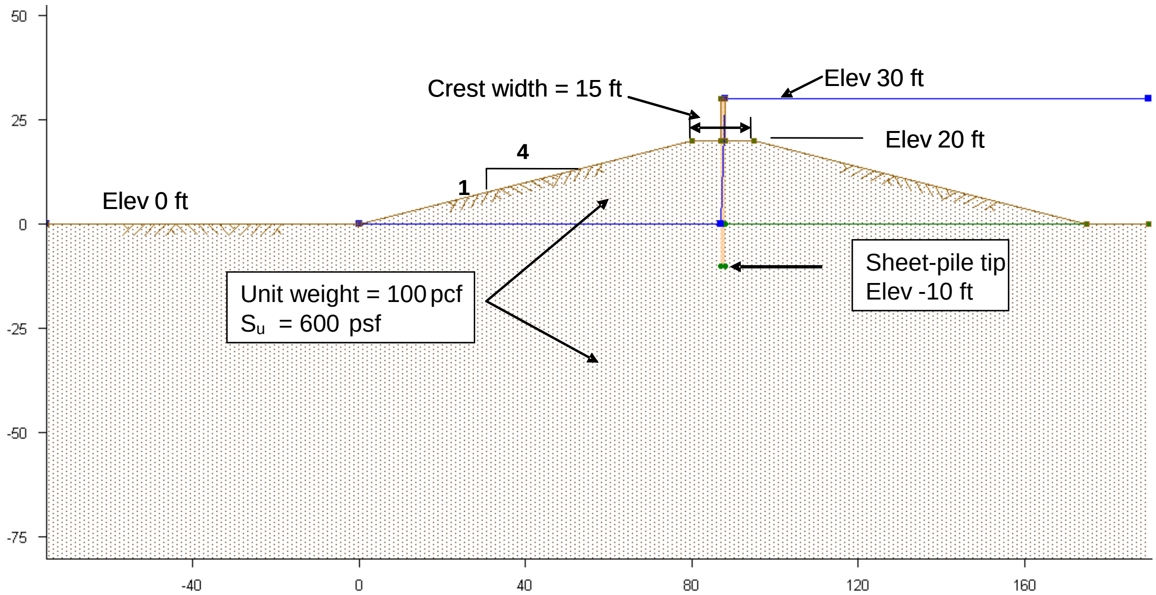


Figure C-10. Cross-section for Example 1

c. Step 2: Determine Forces Acting on Sheet-pile Wall.

(1) Since this is a full-gap analysis, only the forces from the water pressure acting on the wall over its full depth need to be considered.

(2) Water pressure can be treated as a distributed load acting on the face of the sheet-pile wall from the water surface at the top of the wall to the tip of the sheet pile at the bottom.

(3) The water pressure distribution is determined as follows:

[1] To convert pounds (force) per square foot to kilopascals, multiply by 0.04788. To convert pounds (mass) per cubic foot to kilograms per cubic meter, multiply by 16.02. To convert feet to meters, multiply by 0.3048.

(a) At water surface: Elevation 30 ft, water pressure = 0 psf.

(b) At sheet-pile tip: Elevation -10 ft, water pressure = (62.4 pcf × 40 ft) = 2,496 psf.

d. Step 3: Perform Stability Analysis.

(1) Search for critical surface using soil removal technique with surface passing through point on the flood side of sheet-pile tip.

(2) Factor of safety results:

(a) Circular search: 1.36

(b) Noncircular search: 1.29 (shown in Figure C-11)

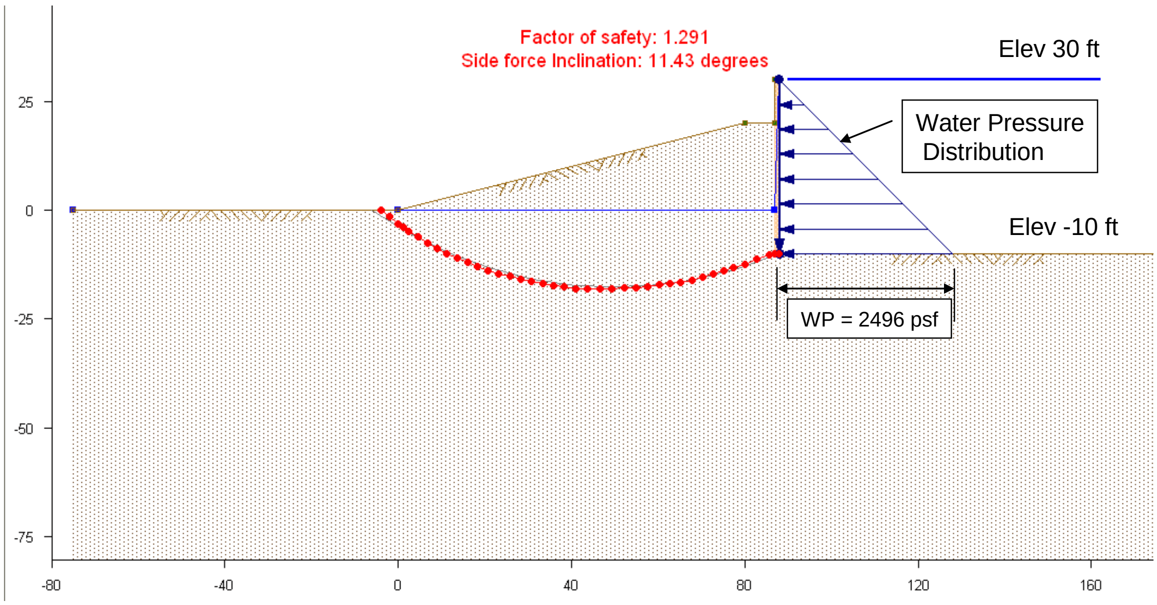


Figure C-11. Results of noncircular search using soil removal technique

C-9. <u>Example 2: Partial-Depth Gap Analysis of I-wall in Uniform Soil Conditions</u>.

a. Problem: Evaluate the stability of the I-wall shown in Figure C-12.

b. Step 1: Determine Depth of Gap from Equation C-3, repeated here:

$$z_o = (2c)/\gamma' = (2 \times 500 \text{ psf})/(100 \text{ pcf} - 62.4 \text{ pcf}) = 26.6 \text{ ft}$$

Partial gap analysis of the I-wall is required because z_0 is less than the penetration depth of 32 ft of the sheet pile below the ground surface.

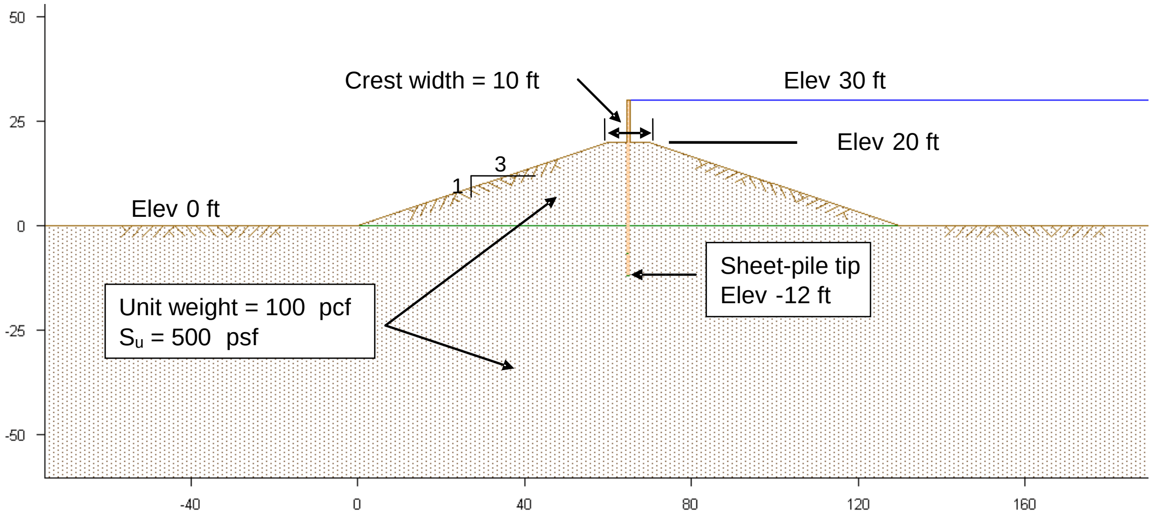


Figure C-12. Cross-section for Example 2

c. Step 2: Determine Forces Acting on Sheet-Pile Wall. Since this is a partial-depth gap analysis, the forces acting on the flood side of the sheet-pile wall will include both water pressures above the bottom of the gap and active earth pressures below the bottom of the gap:

(1) Water pressure distribution acting from water surface elevation (+30 ft) to the bottom of the gap (elevation -6.6 ft):

(a) Water pressure at water surface elevation = 0 psf.

(b) Water pressure at bottom of gap = 62.4 pcf × 36.6 ft = 2,284 psf.

(2) Active earth pressure distribution acting from the elevation of the bottom of the gap to the tip of the sheet pile is determined using Equation C-1 (repeated below) by computing the active earth pressures σ_{ha} at depths corresponding to the bottom of the gap (elevation -6.6 ft) and the tip of the sheet pile (elevation -12 ft):

$$\sigma_{ha} = \sigma_{v0} - 2c = \gamma z + \gamma_w h_w - 2c$$

(a) σ_{ha} at bottom of gap = 100 psf × 26.6 ft + 62.4 pcf × 10 ft – 2 × 500 psf = 2,284 psf.

(b) σ_{ha} at tip of sheet pile = 100 psf × 32.0 ft + 62.4 pcf × 10 ft – 2 × 500 psf = 2,824 psf.

d. Step 3: Perform Stability Analysis. The limit equilibrium analysis for this example was performed using the soil removal technique and the applied loads shown in Figure C-13. The results of this analysis are as follows:

(1) Search for critical surface using soil removal technique with surface passing through point on the flood side of sheet-pile tip.

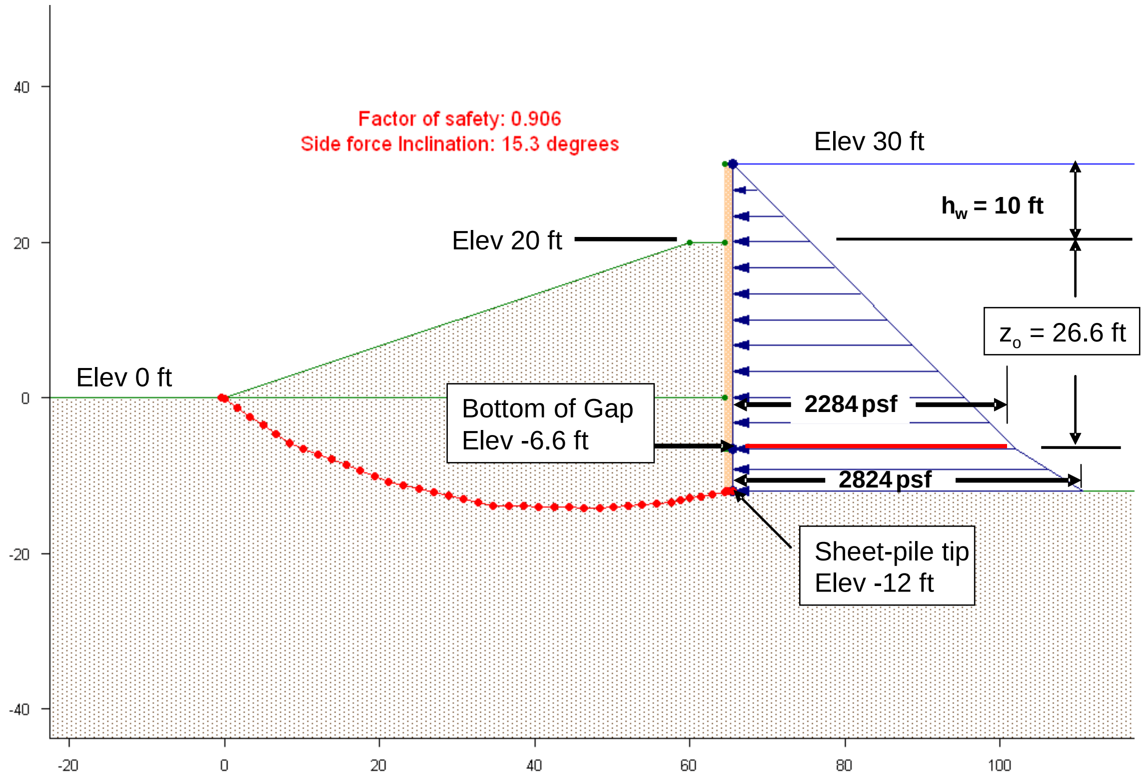


Figure C-13. Results of partial gap analysis for uniform soil conditions using soil removal technique

(2) Factor of safety results:

(a) Circular Search: 0.93.

(b) Noncircular search: 0.91 (shown in Figure C-13).

C-10. Example 3: Partial-Depth Gap Analysis of I-wall in Layered Cohesive Soils. Evaluate the stability of the I-wall shown in Figure C-14.

a. Step 1. Determine depth of gap from Equation C-3.

(1) The gap depth will be determined as the depth at which the water pressure and the active earth pressures are equal.

(2) Compute the water pressure and the active pressure at the depths (elevations) where there is an interface between material boundaries.

(3) Compute the water pressures using Equation C-2 (repeated below):

$$u = \gamma_w(h_w + z)$$

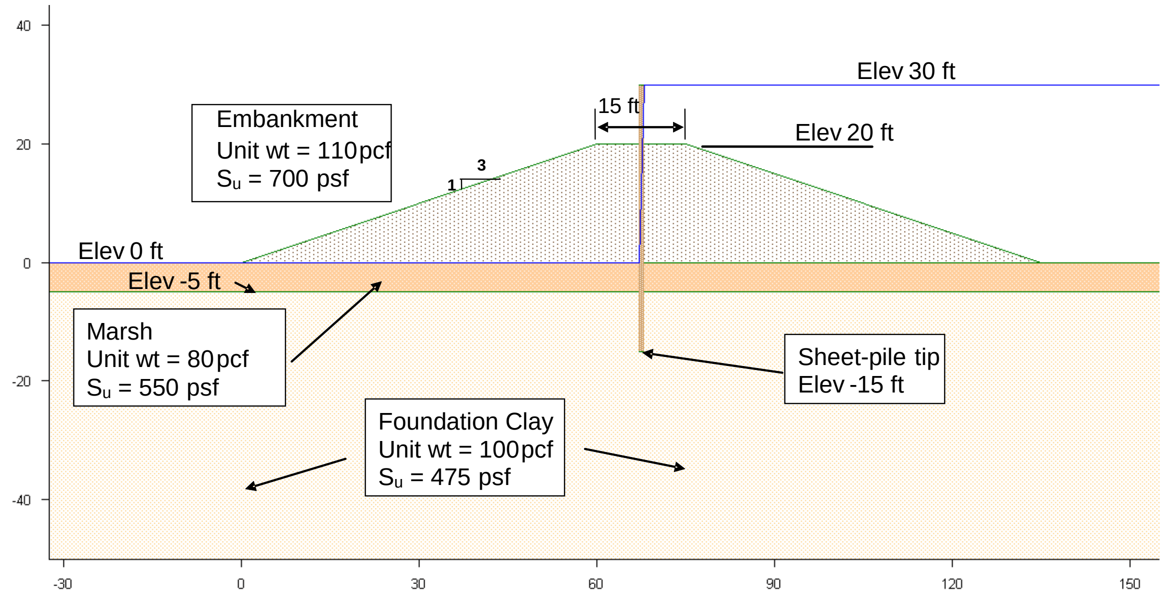


Figure C-14. Cross-section for Example 3

(4) Compute the active earth pressure using the top and bottom of the i^{th} layer using Equation C-4 (repeated below):

$$\sigma_{ha} = \sum(\gamma_i z_i) + \gamma_w h_w - 2c_i$$

(5) The pressure calculations can easily be performed in a spreadsheet. The results of these calculations are tabulated in Table C-1.

(6) Plot the water pressure and horizontal earth pressures as a function of elevation to determine the gap depth.

(7) Figure C-15 and Table C-1 show that the water pressure and horizontal active earth pressure are equal at elevation -5.0 ft where the depth of the gap is 25 ft below the levee crest.

(8) The sheet-pile extends between elevation 20 ft at the ground surface and elevation -15 ft at the tip, which equates to a penetration depth of 35 ft.

(9) A partial gap analysis is required because the depth of the gap, 25 ft, is less than the penetration depth of 35 ft of the sheet pile.

b. Step 2: Determine Forces Acting on Sheet-pile Wall.

(1) Since this is a partial-depth gap analysis, the forces acting on the flood side of the sheet-pile wall will include both water pressures above the bottom of the gap and total active earth pressures acting on the sheet pile from below the bottom of the gap to the tip of the sheet pile.

(2) The water pressure distribution is computed as follows:

Table C-1. Summary of Horizontal Active Earth Pressure and Water Pressure Calculations for Determination of the Gap Depth for Example Problem 3

Layer Number	Elevation ft	Description	Horizontal Active Earth Pressure Calculations $= \Sigma(\gamma_i z_i)$ psf	$\gamma_w h_w$ psf	$-2c_i$ psf	σ_{ha}, psf	Water pressure u, psf
1 Top	20	Levee fill	0	624	-1400	-776	624
1 Bot	0	Levee fill	2200	624	-1400	1424	1872
2 Top	0	Lightweight Marsh	2200	624	-1100	1724	1872
2 Bot	-5^+	Lightweight Marsh	2600	624	-1100	2124	2184
3 Top	-5^-	Foundation Clay	2600	624	-950	2274	2184
3 Bot	-15	Foundation Clay	3600	624	-950	3274	2808

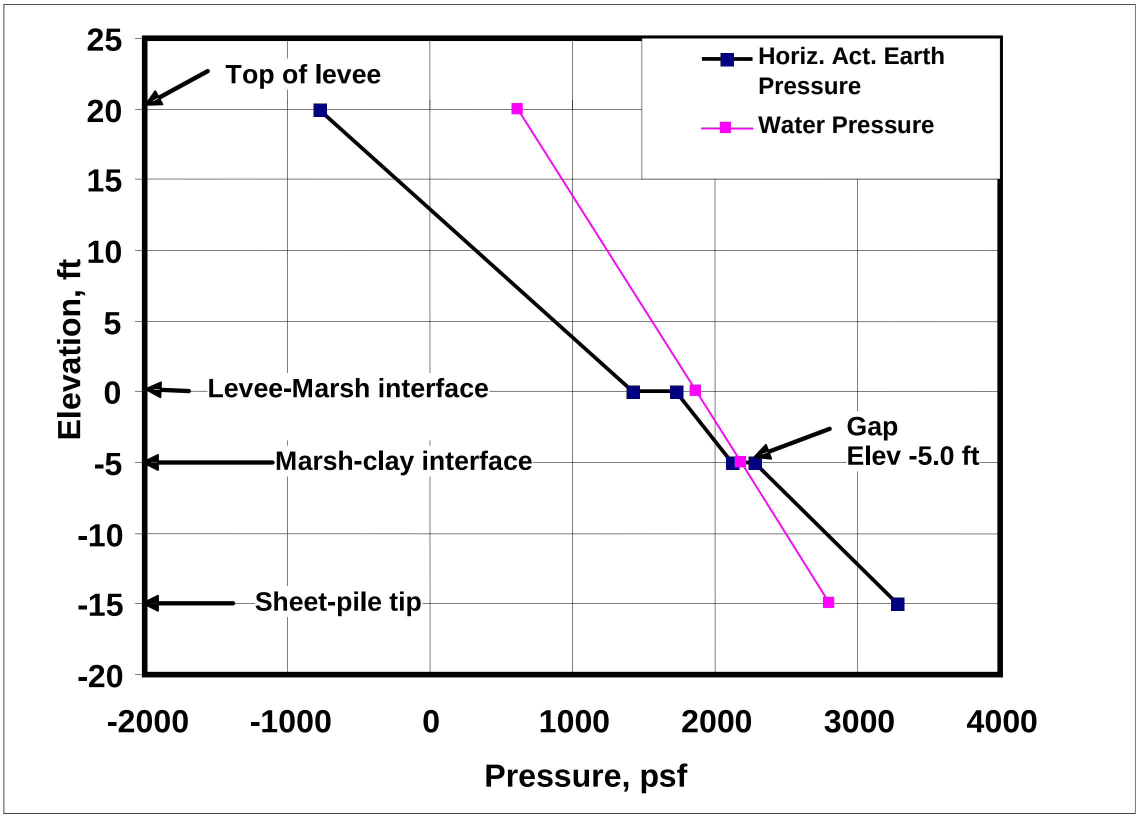


Figure C-15. Horizontal active earth pressure and water pressure profiles for determination of gap depth

(a) Water pressure distribution acting from the water surface elevation (+30 ft) to the bottom of the gap (elevation -5.0 ft):

- Water pressure at water surface elevation = 0 psf.

- Water pressure at bottom of gap = 62.4 pcf × 35.0 ft = 2,184 psf.

(b) The active earth pressure distribution acting from the elevation of the bottom of the gap to the tip of the sheet pile can be determined using the values in Table C-1. Since the gap depth occurs at the location of a layer interface (elevation -5 ft), the pressure against the wall just above the gap depth is represented by the water pressure at that elevation (2,184 psf); and the pressure against the wall just below the gap depth is represented by the total horizontal active earth pressure (2,274 psf). Since the bottom of the sheet-pile tip is in Layer 3, the foundation clay, the values of σ_{ha} at this elevation can be determined by adding the product of the vertical distance and the total density to the value of σ_{ha} at the top of the layer:

$$\sigma_{\mathrm{ha}} = \sigma_{ha\ @top\ of\ clay\ layer} + \gamma_{clay} \times (\mathrm{Elev}_{top\ of\ layer} - \mathrm{Elev}_{any\ point\ in\ layer}) \qquad \text{(C-7)}$$

- At bottom of gap (top of Layer 3): σ_{ha} = 2,274 psf.

- At tip of sheet pile: σ_{ha} = 2,274 psf + 100 pcf [-5ft - (-15 ft)] = 3,274 psf.

c. Step 3. Perform Stability Analysis. The limit equilibrium analysis for this example was performed using the soil removal technique and the applied loads shown in Figure C-16. The results of this analysis are listed below:

(1) Search for critical surface using soil removal technique with surface passing through point on the flood side of sheet-pile tip.

(2) Factor of safety results:

(a) Circular Search: 0.89.

(b) Noncircular search: 0.87 (shown in Figure C-16).

C-11. <u>Example 4: Partial-Depth Gap Analysis of I-wall Penetrating a Sand Layer</u>. Evaluate the stability of the I-wall shown in Figure C-17.

a. Step 1: Determine depth of gap from Equation C-3:

$$z_o = (2c)/\gamma' = (2 \times 800\ \text{psf})/(110\ \text{pcf} - 62.4\ \text{pcf}) = 33.6\ \text{ft}$$

(1) The computed depth of the gap z_o is sufficient to reach the sand layer because the vertical distance from the top of the levee is only 15 ft.

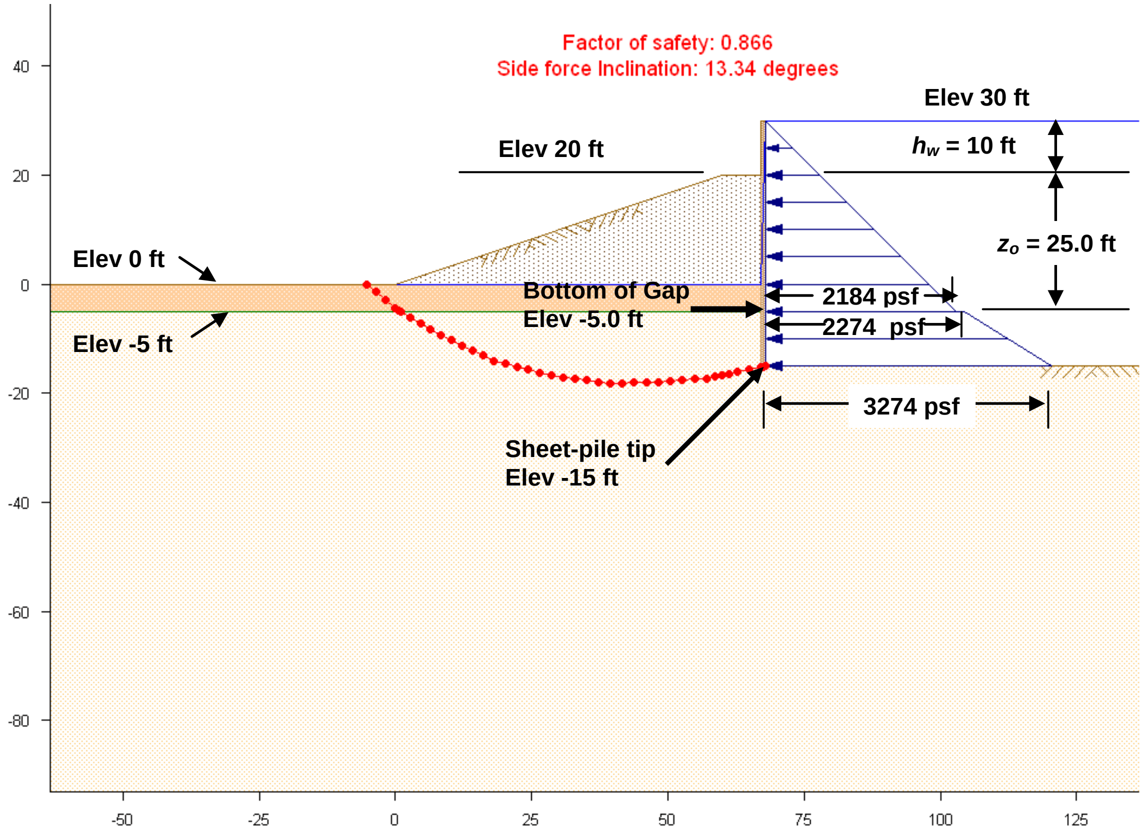


Figure C-16. Results of noncircular search for partial gap analysis using soil removal technique

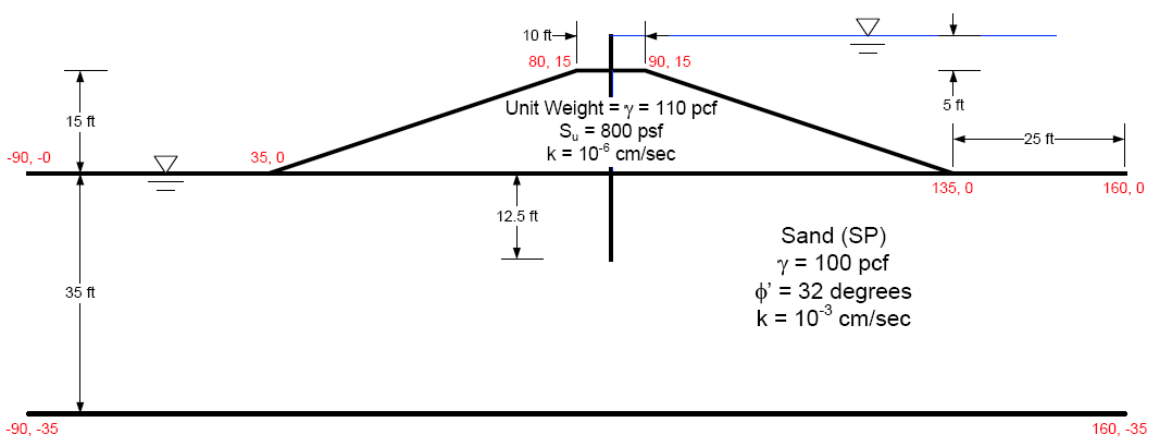


Figure C-17. Cross-section for Example 4

(2) Therefore, a partial-gap analysis of the I-wall is required.

b. Step 2. Perform steady-state seepage analysis.

(1) A seepage analysis is needed for two main purposes:

(a) Determine the pore-water pressures in the sand layer. These are imported into the slope stability so that the shear strength of the sand can be calculated on the basis of effective stress.

(b) Determine the water pressure and effective stress distributions against the sheet-pile wall so that the resultants of these forces acting on the wall can be calculated for the stability analysis.

(2) The seepage analysis should be performed with the appropriate boundary conditions for the problem in question. However, additional boundary conditions need to be included to account for the pathway on the flood side of the sheet pile that the gap provides for the flow of water under a constant total head set equal to the water level flood side of the wall.

(3) The pore-water distribution acting against the wall is shown in Figure C-18. The pore-water field in the soils is shown in Figure C-19.

c. Step 3: Determination of forces acting against the sheet-pile wall.

(1) The water pressure distribution and horizontal effective earth pressures acting against the wall are plotted in Figure C-18.

(2) The effective active earth pressures were determined using Equation C-8:

$$\sigma'_h = \sigma'_v K_a \qquad \text{(C-8)}$$

where $K_a = \tan^2 (45 - \varphi/2) = \tan^2 (45 - 32^o/2) = 0.307$

(3) The resultant forces and their locations where they act were numerically evaluated in a spreadsheet.

(4) The results of the spreadsheet calculations for both the water pressure distribution and the effective horizontal active earth pressures are shown in Figure C-18.

d. Step 4: Perform Stability Analysis. The results of the slope stability analysis are shown in Figure C-19, which shows the following:

(1) The pore-water contours from the seepage analysis.

(2) The boundary conditions.

(3) The line loads representing the resultant water pressures and the effective horizontal active earth pressures acting on the wall.

(4) The factor of safety is 1.62 for the critical circle shown in the figure.

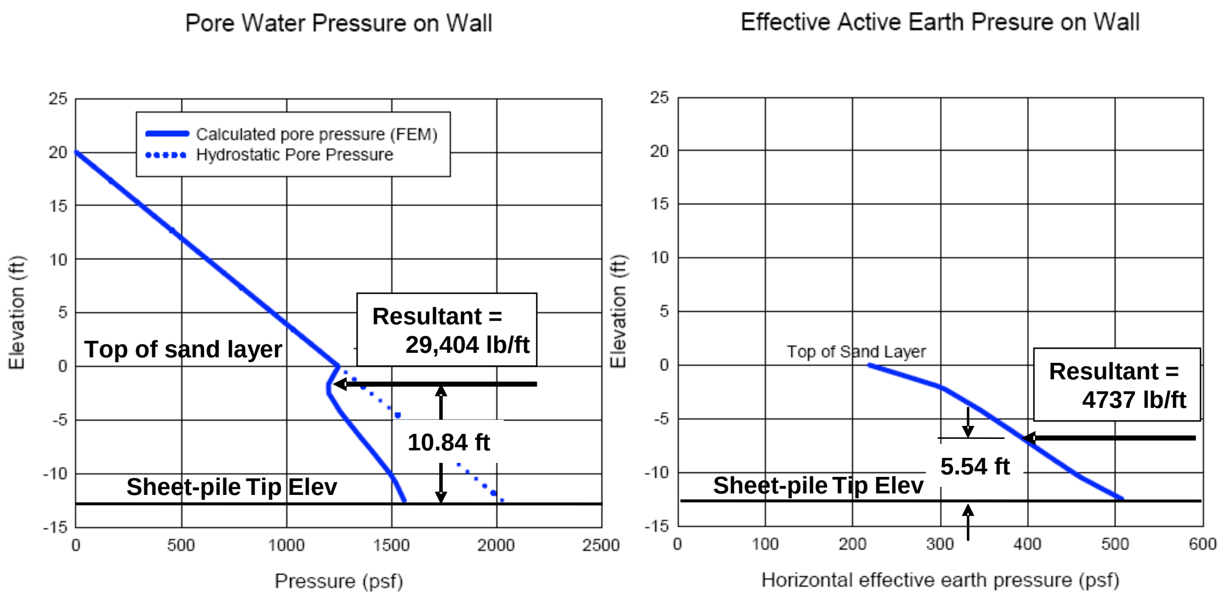


Figure C-18. Pore-water pressure and effective active earth pressures acting on the sheet-pile wall

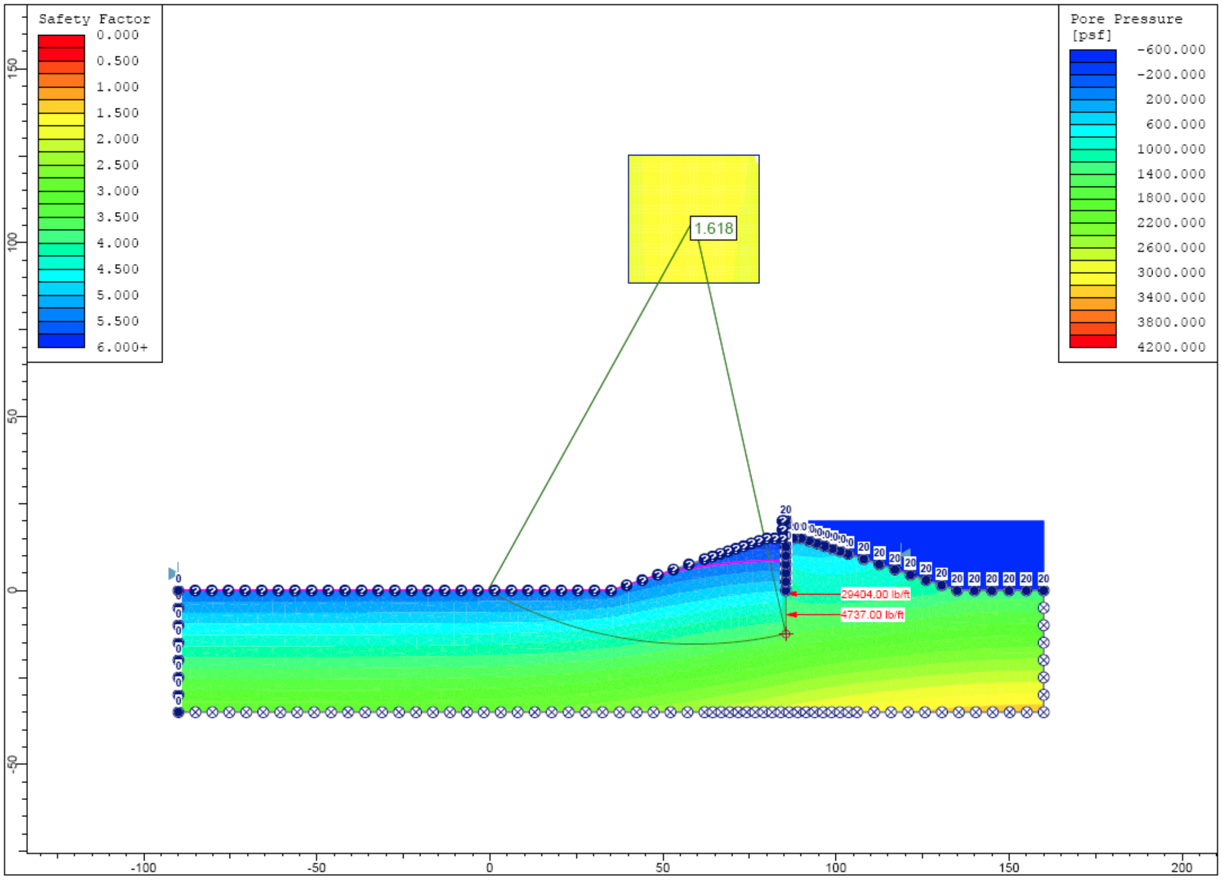


Figure C-19. Results of slope stability analysis

APPENDIX D
COMPUTATION OF ROTATIONAL FACTOR OF SAFETY FOR I-WALLS ON LEVEES

D-1. General. This appendix provides an approach for computing a correct factor of safety (FS) for rotational stability for I-walls on levees using CWALSHT (U.S. Army Engineer Research and Development Center 2007). The procedure is described step by step using an example of an I-wall section from the London Avenue outfall canal in New Orleans, LA. Factors of safety computed by soil-structure interaction (SSI) analysis and by the Global Stability analysis are compared.

D-2. Basic Procedure.

a. CWALSHT computes passive pressures using equations that are based on wedge solutions or trial wedge searches. Fast Lagrangian Analysis of Continua (FLAC) analyses show that a failure surface for a levee-founded I-wall may not be a wedge that develops through higher strength levee material. Instead it may be a generalized slip surface that could involve sliding in the foundation beneath the levee (i.e., the passive resistance in a rotational stability analysis may be limited by the critical block resistance rather than the passive wedge). In this way CWALSHT is not always capable of finding the critical failure mechanism when calculating the passive resistance on the protected side of the wall. This procedure adjusts the passive pressures in CWALSHT to compute factors of safety more accurately. The least passive resistance, calculated by CWALSHT or the critical block resistance found using slope stability software, is used for calculating rotational stability in this approach.

b. The basic steps of the procedure are as follows:

(1) Determine the elevation at which a slip surface level may be critical for passive pressure force. In some sections the soil stratigraphy may make this obvious, but in others several elevations may need to be investigated.

(2) Run CWALSHT and copy the left-side passive pressures from the output into a spreadsheet. Also record the FS.

(3) Compute the total passive pressure force from the ground surface to the selected slip surface elevation from the CWALSHT output (the integral of the passive pressure distribution).

(4) Compute the minimum passive pressure force from the top of levee ground surface to the slip surface level for the FS corresponding to the CWALSHT analysis using a slope stability program. It is expedient in this process to develop a plot of minimum passive pressure force versus FS. Various surcharge loads are applied within the gap (removing flood-side soil in this example) from the water surface on the wall to the elevation of the selected slip surface at the wall. The minimum FS is then computed for each load to create the plot.

(5) Compute the difference in the passive pressure force from CWALSHT and the minimum passive pressure forces corresponding to the same FS.

(6) Apply the difference in passive pressure forces as a pressure or line load in CWALSHT. The location of the force or shape of the pressure is determined by judgment but in many cases can be represented by a uniform pressure from the ground surface to the selected slip surface level.

(7) Run CWALSHT and record the FS. Check that the transition point in the net pressure diagram is below the slip surface down to which pressures are computed. If the transition point is above the slip surface level, then the slip surface should be moved up to the elevation of the transition point.

(8) Redo steps (2) through (7) until the FS in step (7) matches, or is close to, the previous iteration. This is the FS for rotational stability.

D-3. Example Section. The analysis section taken from the London Avenue outfall canal in New Orleans, shown in Figure D-1, includes clay embankment (el +2.5[1]), underlain by marsh material (el -6.2 to -11.5), underlain by a silty sand stratum (el -11.5 to -20), underlain by a clean sand stratum (el -20 to -43.5) that mantles a lower clay deposit. The thickness of the entire marsh layer was based on its thickness at the levee toe. The flood-side berm at the wall face is at el +2.6 whereas the land-side berm is at el +2.3. The floodwall has a top elevation of about +12.9 consisting of a concrete cap and CZ-101 sheet pile to a tip elevation of -21.5 ft. Piezometer data used in this model was obtained during the London Avenue Canal load test performed in 2007.

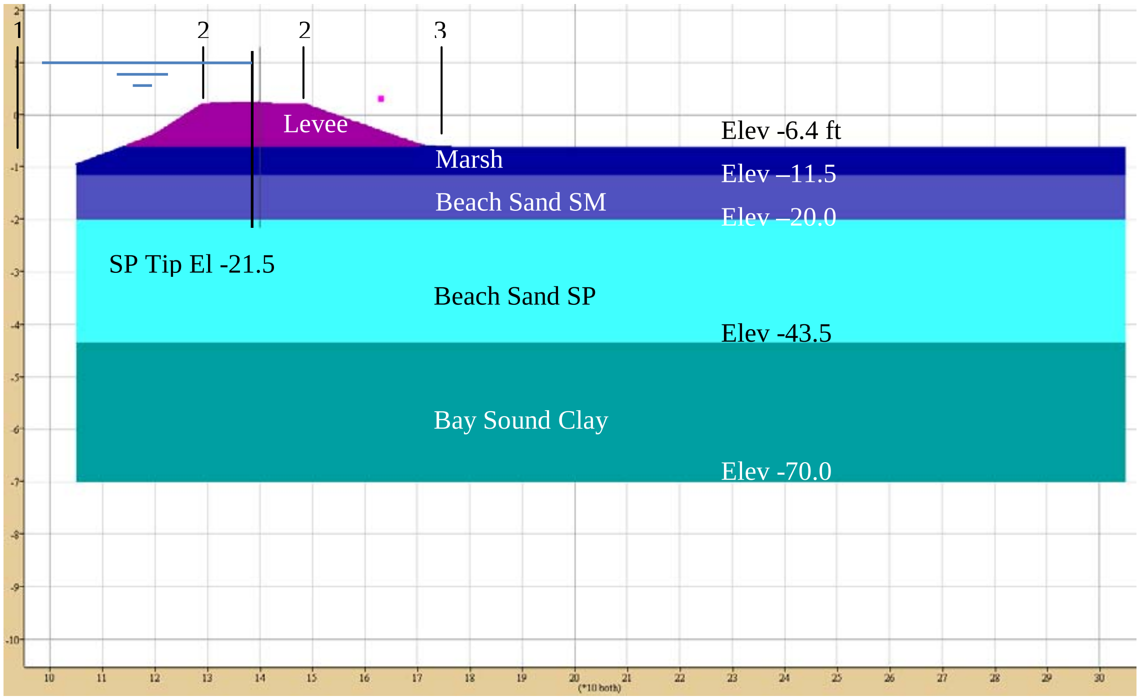


Figure D-1. Analysis section. The S_u values of the marsh material vary between verticals 1 to 2 and 2 to 3

[1] All elevations (el) cited in this appendix are in feet referred to the North American Vertical Datum of 1988 (NAVD 88). To convert feet to meters, multiply by 0.3048.

D-4. Computation of Correct Passive Pressure Forces.

a. Slope stability software (Slope/W in this example) can be used to establish a minimum passive pressure force versus FS plot in accordance with Step 4 of the procedure (paragraph D-2b). To create this plot, a horizontal surcharge load was applied within the gap (removing flood-side soil) from the water surface on the wall to the elevation of the slip surface at the wall (Figure D-2). The unit weight of this surcharge load was varied, and the resulting FS was computed. The load against the wall was found from the slice information to create the plot shown in Figure D-3. The initial slip surface point was fixed vertically at el -11.2, just above the marsh sand interface at el -11.5, for generating data shown in Figure D-3. This point was initially allowed to search vertically but was fixed at el -11.2 based on the location of slip surfaces (the slip surfaces all dipped to around the marsh sand interface as seen in Figure D-2) to obtain consistent results.

b. For the CWALSHT analyses the reduction in passive capacity was applied from the top of levee to el -11.2, approximately the bottom of the slip surfaces, according to Figure D-3.

D-5. CWALSHT Analyses Using the Corrected Passive Pressure.

a. A plot of the input for the CWALSHT of the example section with water at el 10 is shown in Figure D-4. As stated in the overall procedure, the wall is analyzed with an estimated FS; the passive pressures are copied to Excel; integrated to el -11.2; and then subtracted from the passive force from the curve in Figure D-3 at the FS computed in CWALSHT. The difference is input into CWALSHT as a uniform pressure from the ground surface to el -11.2 as shown in Figure D-5.

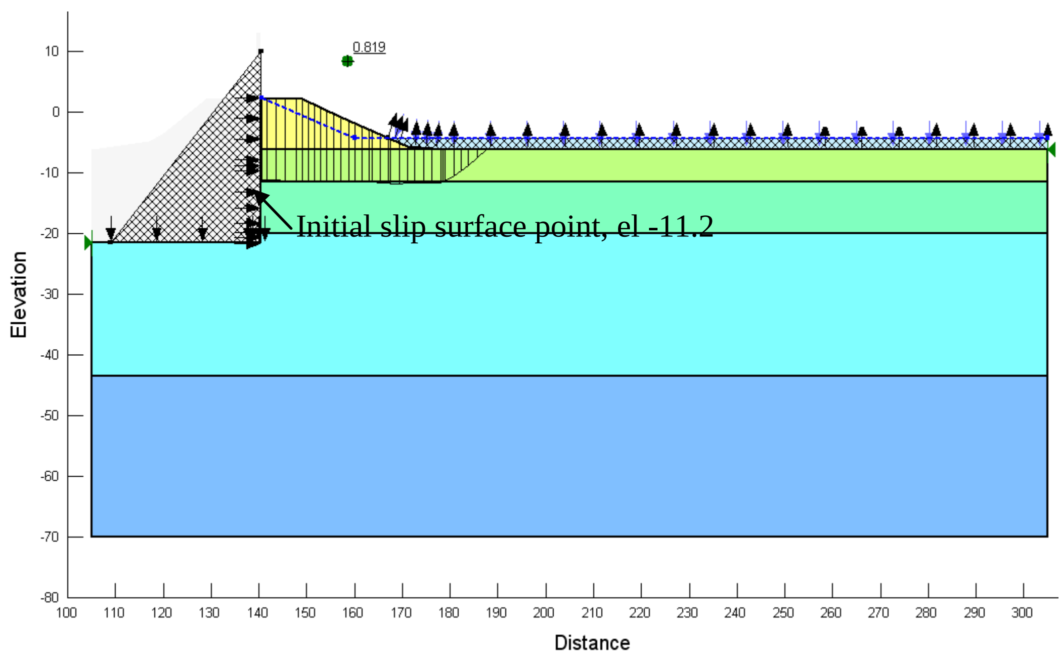


Figure D-2. Slope/W model used to find the passive resistance FS for an actuating load (Note: FS = 0.82 for a surcharge unit weight of 62.4 pcf (999.6 kg/m^3) when the sheet pile and the water load above the top of levee are neglected)

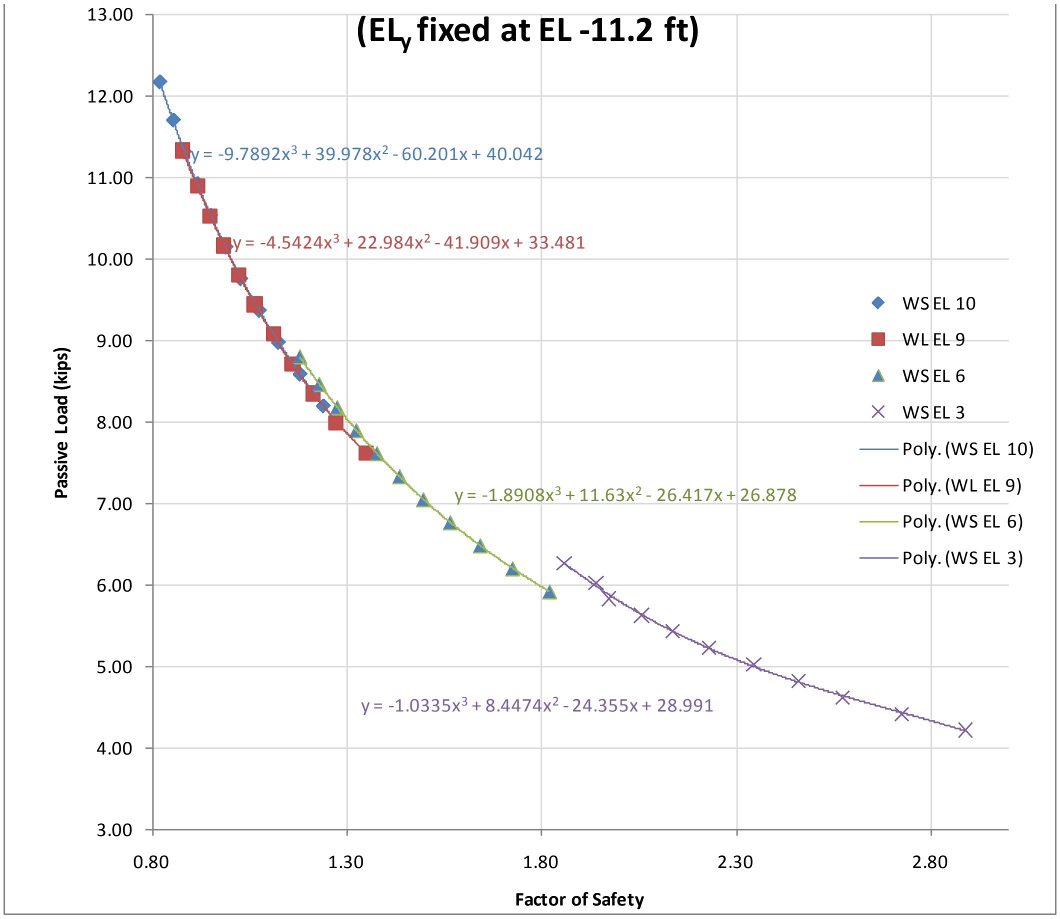


Figure D-3. Slope/W passive resistance for a fixed initial point
of the slip surface at el -11.2

b. The data shown in d below was copied from the CWALSHT output file to Excel. Shown is the CWALSHT run after iterating the FS and applied load so that the CWALSHT FS was the same as the FS used to compute the pressure difference applied as a pressure as shown in Figure D-5.

c. The left-side passive pressures are integrated from el 2.3 to el -11.2. This is equal to 15,268 lb (6,925 kg). Using the equation in Figure D-3 for a water level of 10, the passive pressure from the CSLIDE model for an FS of 1.06 is 9,489 lb (4,300 kg). The difference of 5,779 lb (2621 kg) is divided by the height from the ground surface to the top of sand at el -11.2 for a uniform horizontal load of 428 lb/ft (59.2 kg/m) as shown in Figure D-5. Since the FS from CWALSHT matches the FS used in the prior run to compute the difference in pressure, the correct FS is 1.06. The FS computed by CWALSHT without the correction is 1.78.

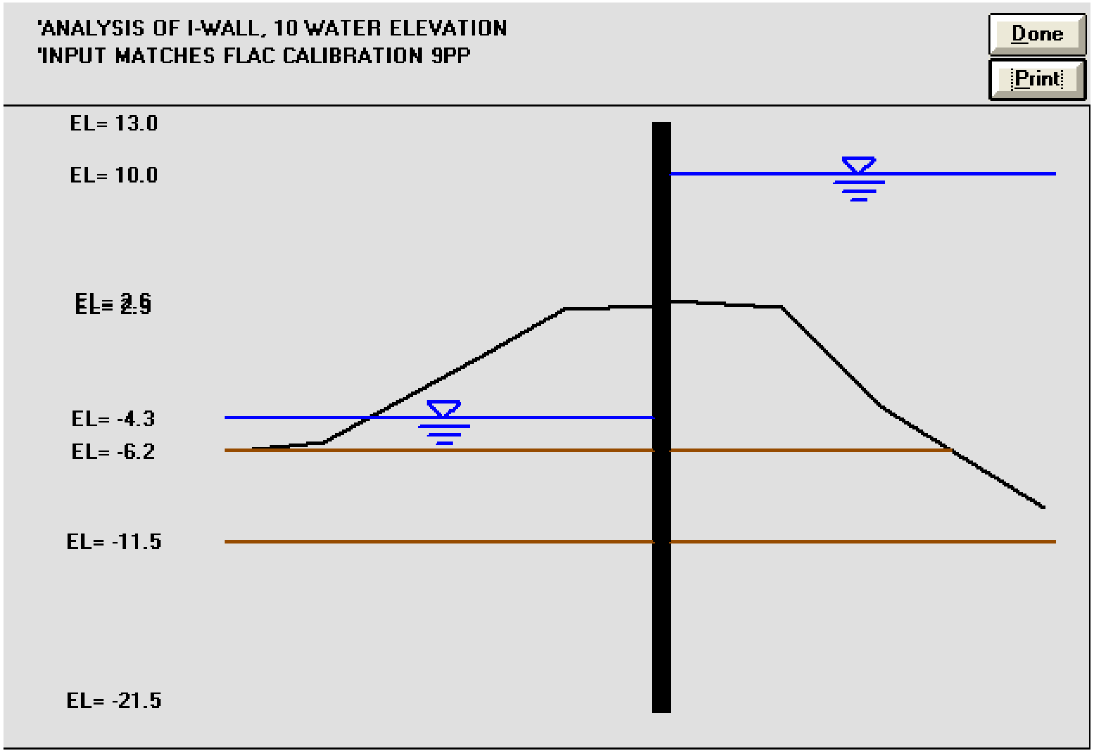


Figure D-4. Input for the CWALSHT of the example section with water at el 10

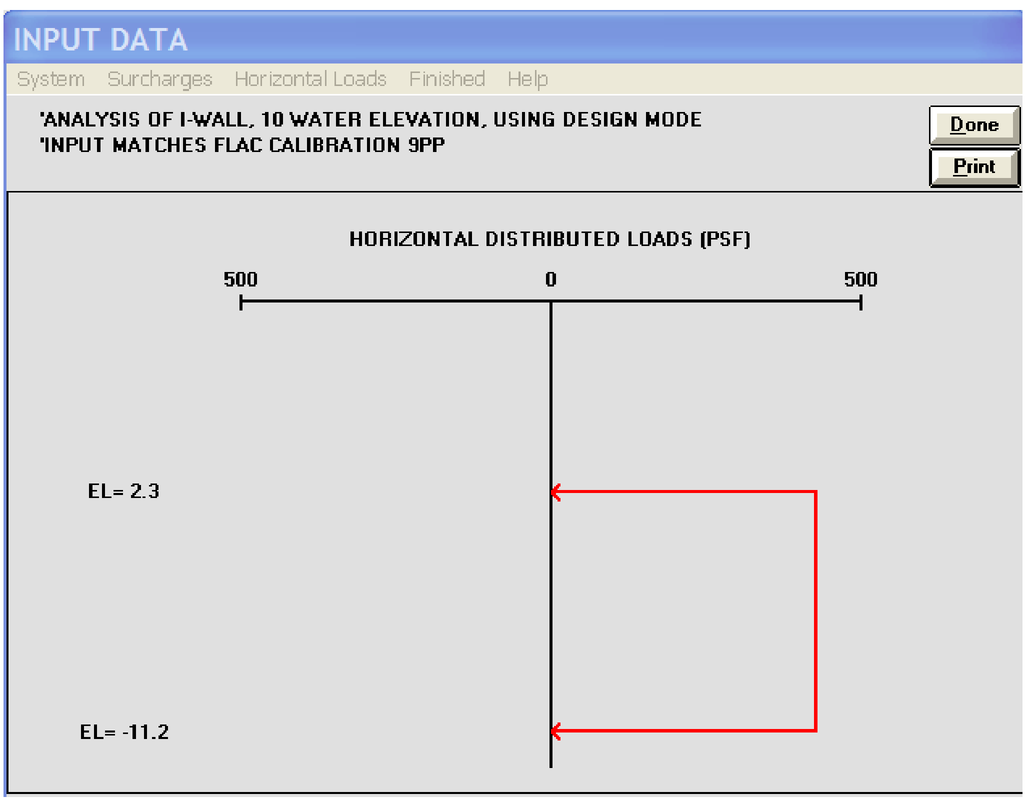


Figure D-5. Uniform pressure from the ground surface to el -11.2

d. CWALSHT Output File to Excel

```
 I.--HEADING
'ANALYSIS OF I-WALL, 10 WATER ELEVATION, USING DESIGN MODE
'INPUT MATCHES FLAC CALIBRATION 9PP
'PASSIVE PRESSURE ADJUSTED TO MATCH SLOPE/W

II.--CONTROL
   CANTILEVER WALL DESIGN
   FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
   FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.06
```

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES		------------->	
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
13.00	0.	0.	0.	0.	0.
12.00	0.	0.	0.	0.	0.
11.00	0.	0.	0.	0.	0.
10.00	0.	0.	0.	0.	0.
9.00	62.	0.	0.	0.	0.
8.00	125.	0.	0.	0.	0.
7.00	187.	0.	0.	0.	0.
6.00	250.	0.	0.	0.	0.
5.00	312.	0.	0.	0.	0.
4.00	374.	0.	0.	0.	0.
3.00	437.	0.	0.	0.	0.
2.60+	462.	0.	0.	0.	0.
2.60-	462.	0.	0.	0.	849.
2.30+	480.	0.	0.	0.	833.
2.30-	480.	849.	0.	0.	833.
2.00	499.	863.	0.	0.	846.
1.30	543.	937.	0.	0.	877.
1.00	562.	969.	0.	0.	891.
0.54	590.	1018.	0.	0.	912.
0.00	624.	1076.	0.	0.	936.
-1.00	686.	1182.	0.	0.	981.
-2.00	749.	1289.	0.	0.	1026.
-3.00	811.	1396.	0.	0.	1070.
-4.00	874.	1502.	0.	0.	1115.
-4.30	892.	1528.	0.	0.	1129.
-5.00	892.	1565.	0.	0.	1160.
-6.00	892.	1610.	0.	0.	1205.
-6.20	892.	1370.	0.	0.	1145.
-7.00	892.	822.	0.	0.	424.
-8.00	892.	840.	0.	0.	442.
-9.00	892.	857.	0.	0.	459.
-10.00	892.	875.	0.	0.	477.
-11.00	892.	908.	0.	0.	495.
-11.20	892.	896.	0.	0.	498.
-11.50	892.	1116.	0.	0.	593.
-12.00	892.	2687.	0.	0.	1225.
-13.00	892.	2718.	0.	0.	1382.
-14.00	892.	2874.	0.	0.	1359.
-15.00	892.	2990.	0.	0.	1324.
-15.27	892.	2992.	3.	0.	1386.
-16.00	892.	2999.	9.	0.	1551.
-17.00	892.	2942.	148.	0.	1655.
-18.00	892.	2947.	271.	0.	1826.
-19.00	892.	2907.	278.	0.	2187.
-20.00	892.	2893.	286.	0.	2448.

```
-21.00        892.     3094.      288.        0.      2656.
-21.43        892.     3335.      298.        0.      2857.
-23.00        892.     3570.      297.        0.      3073.
```

D-6. FLAC Factor of Safety Analyses.

a. FLAC analyses were performed to compute the FS for differing water levels for the example section. Since FLAC is a complete SSI analysis, the benefit in using this tool is the ability to identify potential failure modes without the restriction of having to assume one mode over another (i.e., rotational versus global stability). It is possible that the most critical mode of failure may consist of both modes such as the wall rotating along with translation.

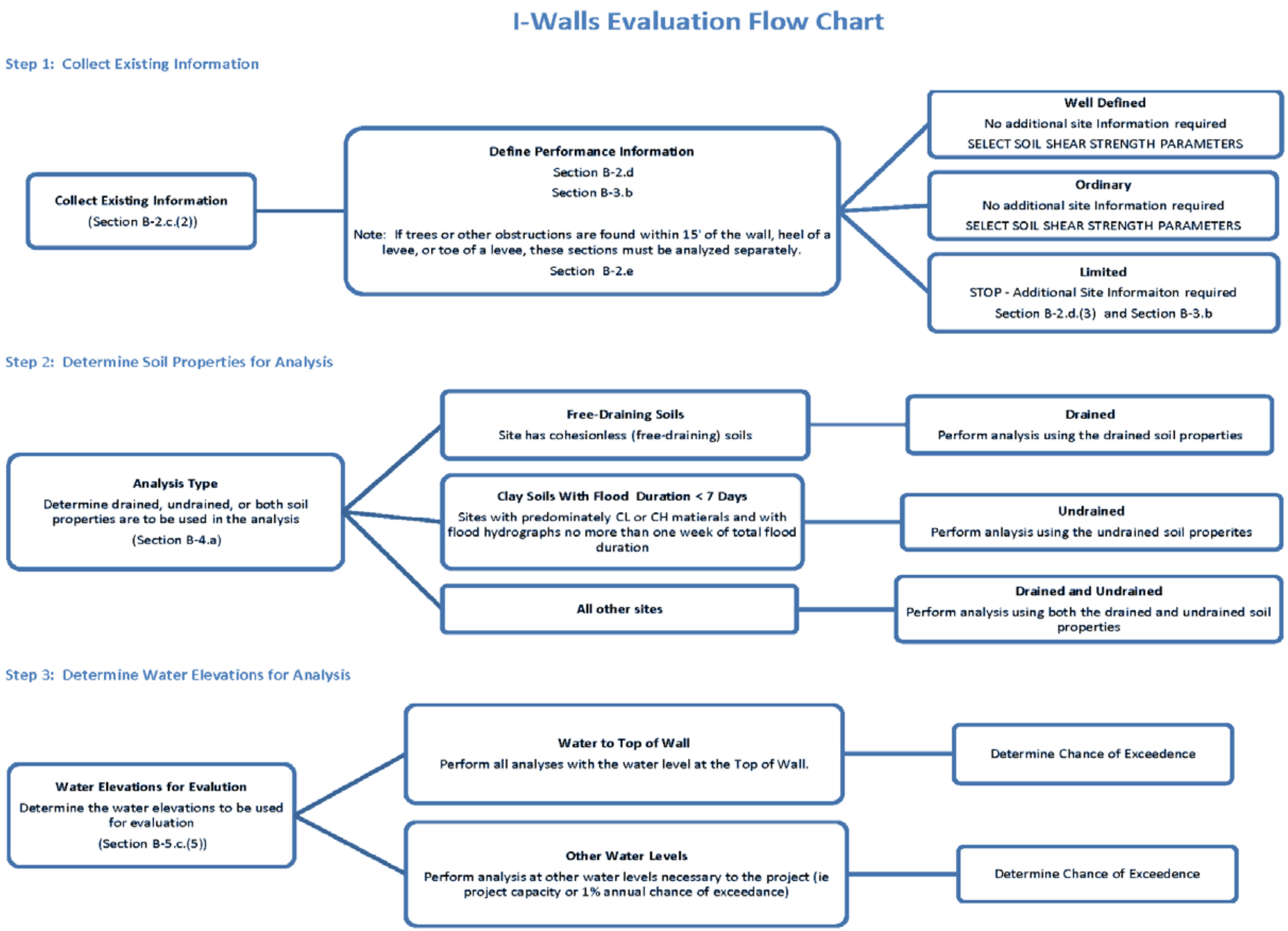
b. The FS found using FLAC indicates that the governing mode of failure is rotation about a point above the tip of the sheet pile for water levels exceeding about el 8. For water levels lower than about el 4, the governing failure mode is slope stability of the levee on the protected side of the wall. At water levels between about el 5 and el 7, the failure mode appears to be transitioning between global stability and rotation.

D-7. Summary of Rotational Factors of Safety. Table D-1 lists factors of safety that were computed for various water elevations in the canal using CWALSHT with the sweep search method and with this procedure. The corrected analysis shows good agreement at higher water levels where the FLAC model indicated that the failure mode was from rotation only, as shown in the shaded cells.

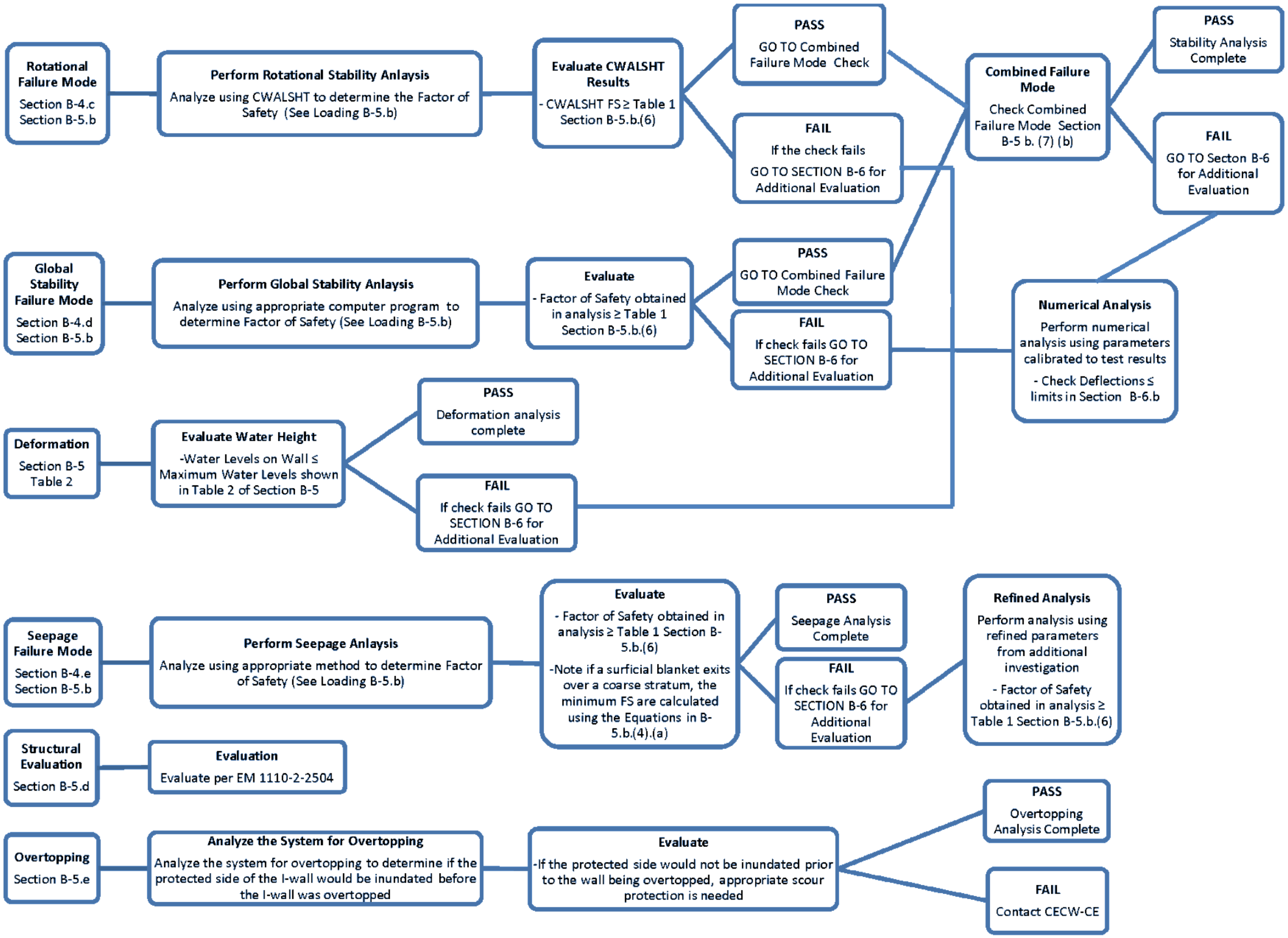
Table D-1. Summary of FLAC and CWALSHT Factors of Safety

	FLAC		CWALSHT	
Water Elevation	Deflection in. (mm)	FS_{fl}	Uncorrected	Corrected
3	0.19 (4.82)	1.92	50+	3.25
4	0.39 (9.91)	1.86	50+	2.6
5	0.65 (16.5)	1.73	50+	2.1
6	1.01 (25.7)	1.61	50+	1.84
7	1.81 (46.0)	1.48	8.50	1.6
8	4.23 (107)	1.33	3.30	1.36
9	6.62 (168)	1.21	2.29	1.2
10	11.0 (278)	1.05	1.78	1.06

APPENDIX E
I-WALLS EVALUATION FLOWCHART



Step 4: Analyze Wall (Rotation, Global Stability, Seepage, Structural, Overtopping, Deflection)



GLOSSARY

c	Cohesive strength; undrained soil strength of uniform soil deposit
c'	Effective stress cohesion
c_i	Undrained shear strength of the i^{th} layer
FS_g	Factor of safety relative to seepage gradient
h_o	Excess head (above hydrostatic) at toe
h_w	Height of water above flood-side ground surface
i	Subscript indicating the soil layer i
I_{cr}	Critical exit gradient = γ'/γ_w
I_e	Exit gradient
K_a	Active pressure coefficient
K_p	Passive pressure coefficient
S_u	Undrained shear strength
u	Hydrostatic water pressure in gap; pore water pressure acting along the sheet pile in the sand layer
z	Depth below ground surface
z_0	Depth of gap
z_{bL}	Land-side blanket thickness
z_i	Thickness of the i^{th} layer
γ	Unit weight of soil
γ'	Effective soil unit weight
γ_i	Unit weight of the i^{th} layer
γ_w	Unit weight of water

σ_h	Total horizontal pressure
σ_{ha}	Active horizontal earth pressure
σ'_v	effective vertical stress
σ_{v0}	Vertical earth pressure
φ	Internal friction angle