Draft Final Report – Volume 2

to the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM (NCHRP)

on Project 12-70

Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments

Recommended Specifications, Commentaries, and Example Problems

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Section 1 – Introduction

Volume 2 of this Draft Final Report *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankment* provides: (1) recommended Specifications and Commentaries for the seismic analysis and design of retaining walls, slopes and embankments, and buried structures; and (2) example problems demonstrating the use of the proposed Specifications and Commentaries. This volume was prepared as part of the National Cooperative Highway Research Program (NCHRP) 12-70 Project. Volume 1 of the Project Report summarizes background information that was used to develop the Specifications, Commentaries, and Example Problems contained in this volume. The background information in Volume 1 includes a compendium of work summarized in interim Project reports dated January of 2005, March of 2006, and November 2006, and June / July 2007.

1.1 Background

The NCHRP 12-70 Project involved the development of seismic design guidelines in three areas: retaining walls, slopes and embankments, and buried structures. The retaining walls considered in this Project are freestanding walls that do not form part of a bridge structure (e.g., abutment walls are not included). Slopes and embankments can be either natural or fill slopes associated with construction of a new embankment and cuts in existing sloping areas. Buried structures refer to drainage structures and small pedestrian tunnels but not vehicular tunnels.

The Project's overall objective was to develop analysis and design methods and to prepare LRFD specifications and example problems for the design of retaining walls, slopes and embankments, and buried structures. This overall objective was intended to address short-comings in AASHTO *LRFD Bridge Design Specifications* being used when the Project was initiated in 2004. In some cases the Project objective addressed the absence of a recommended design methodology in the AASHTO Specifications.

A number of design limitations had been identified within each area of evaluation as summarized below:

- **Retaining Walls:** Common practice, including the latest edition of the AASHTO *LRFD Bridge Design* involves use of the Mononobe-Okabe (M-O) equations for estimating seismic active and passive earth pressures. This procedure is found to give unreasonably high levels of earth pressures when some combinations of high ground acceleration and steep backslopes above the retaining wall occur. The M-O equations also are not derived for soil conditions typically encountered during the design of many freestanding walls, and there is general lack of clarity on what seismic coefficient to use in the M-O equations when assessing the various performance modes (e.g., external and internal stability). Additionally, it is not clear that the M-O equations are applicable for walls that are restrained from movement, such as anchored retaining walls.

- **Slopes and Embankments:** The AASHTO LRFD *Bridge Design Specifications* provides no specific guidance for the design of slopes and embankments under gravity or seismic loading. The evaluation of seismic slope stability is often a key component of the earthquake hazards assessment, either when the roadway involves cuts and fills or when global stability poses a risk to a bridge or retaining structure forming part of the transportation corridor.
- **Buried Structures:** Section 12 of the AASHTO *LRFD Bridge Design Specifications* provides guidance on the design of culverts and drainage pipes for static loads, but provides no methods for considering seismic loads or seismic-induced ground movement. These buried structures could be damaged by either transient ground displacement (TGD) or permanent ground displacement (PGD) during an earthquake. While many buried structures do not warrant a seismic design, for those situations where the buried structure could lead to damage to the roadway, some standard guidance is needed.

1.2 LRFD Design Methodology

The work carried out for the NCHRP 12-70 Project attempted to be consistent with the philosophy and format of the AASHTO *LRFD Bridge Design Specifications* and the seismic provisions for highway bridges. In this philosophy "Bridges shall be designed for specified limit states to achieve the objectives of constructibility, safety, and serviceability, with due regard to issues of inspectibility, economy, and aesthetics…." In the LRFD procedure, margins of safety are incorporated through load (γ_p) factors and performance (or resistance, ϕ_r) factors.

The basic requirement for a project designed in accordance with the LRFD philosophy was to ensure that factored capacity exceeded factored load as defined by the following equation for various limit states (or acceptable performance):

$$\phi_r R_n \geq \sum \gamma_i Q_i \tag{1-1}$$

Where

ϕ_r = performance factor

R_n = nominal resistance

γ_p = load factor for load component i

Q_i = load effect due to load component i

For the static (or gravity) design case the appropriate load and resistance factors had been developed for many structures to yield a consistent margin of safety in the designed structure. Ideally, this same logic needed to be followed for seismic loading to retaining walls, slopes and embankments, and buried structures. However, the approach for defining a consistent margin of safety during seismic loading was more difficult to define for the following reasons:

- The load factors and load cases (i.e., on the right-hand-side of the above equation) needed to be consistent with those recommended by the NCHRP Project 20-07 *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* (NCHRP, 2006). The NCHRP 20-07 Project was establishing the appropriate earthquake loading return period – subject to the approval of the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS T-3) and eventually the AASHTO voting members. These recommendations would result in very large loads associated with a seismic event at a specific site, but the likelihood of the load occurring was relatively small (i.e., 7 percent probability in 75 years). Under this situation use of a load factor on the seismic load was believed to be overly conservative.

- From a resistance factor standpoint, designers would use either a force- or displacement-based approach. For a force-based approach there is an implied understanding that making conservative assumptions on soil parameters results in a conservative design. However, this would be unconservative for displacement-based approaches and could be unconservative for some force-based approaches. The Project Team decided to recommend best-estimate soil properties for most cases when evaluating the ratio of seismic capacity (C) to demand (D). By using best-estimate soil properties, the designer would have (1) a better understanding of the actual reserve (i.e., margin of safety) in the system, and (2) if a displacement-based approach was taken, the displacements would not be overestimated. This latter reason would help designers avoid recommending expensive mitigation methods for projects where the best-estimates of displacement would be tolerable.

The thrust of the work involved three activities: (1) identifying the limit states to be considered during the earthquake load case; (2) defining the expected performance of the designed system for each of the limit states; and (3) outlining the design analysis procedure and capacity criteria. The limit-state evaluation identified three areas of consideration for retaining wall design, which was considered the most critical component of the project:

- The first involved the evaluation of the global stability of the overall site. This evaluation is essentially an assessment of slope stability with the retaining wall included. Generally the assessment of global stability involves a failure surface that passes below the deepest extent of the retaining wall. For semi-gravity walls the failure surface is relative shallow, but for nongravity cantilever walls the sliding surface can be very deep. For some sites this includes evaluation of the potential for and consequences of liquefaction.
- The next area dealt with the design of the foundation system for external stability to ensure that the size of the foundation and the implied geotechnical (i.e., overall soil) capacity were sufficient. This evaluation includes sliding, overturning, and bearing checks, with the structure normally assumed to be rigid.
- Finally, the internal structural stability was evaluated to ensure that structural components would function properly under the increased dynamic load from the earthquake. For retaining walls these structural components range from inextensible and extensible reinforcement (e.g., steel strips, welded wire, geogrids, or geotextiles) in a mechanically stabilized earth (MSE) wall to the reinforcement in the stem of a semi-gravity wall.

The limit states for the design of slopes and embankments and for buried structures were more limited than for retaining walls. For slopes and embankments, either a limit equilibrium stability analysis or acceptable displacements were of interest. These analyses were consistent with the global stability considered for the retaining wall. For buried structures forces and deformations developing in the pipe or culvert during ground shaking were of primary interest. These analyses addressed internal stability only.

When preparing the Specifications and Commentaries for the retaining walls, slopes and embankments, and buried structures, it was assumed that the starting point for seismic design would be the provisions given in the current AASHTO *LRFD Bridge Design Specifications* for gravity and live loads. The objective of the designer would be to check the static design for seismic loads. If the static design did not meet capacity-to-demand (C/D) requirements for seismic loads, then the static design would be modified and the check on seismic performance would be conducted again.

A key consideration with this approach was the performance expectations during the design seismic event. Criteria for internal capacity of retaining structures with respect to shear and moment are relatively well established. However, the requirements for global and external stability are less well established. Specifically, some movement of the structure or slope may be tolerable at some locations, as long as the movement does not lead to unacceptable damage to the retaining structure or to facilities located in or near the moving earth. The decision on performance expectations needs to be made by the Owner with the designer providing a realistic description of the performance that is being expected.

1.3 Organization of Volume 2 Report

Volume 2 is organized into two parts following this introductory section. Part 1 provides the proposed Specifications and Commentaries, and Part 2 includes the example problems. The intent of Volume 2 is to be a stand-alone document. The example problems in Part 2 should be sufficiently self explanatory when used with the Specifications and Commentaries that it is unnecessary to refer to Volume 1 of the NCHRP 12-70 Project Report.

1.3.1 Part 1 – Specifications and Commentaries

The Specifications and Commentaries consist of three sections:

- **Section X: Retaining Walls –** This section provides proposed specifications for retaining walls. Six types of retaining walls are addressed in the specifications section: (1) rigid and semi-rigid gravity walls, (2) nongravity cantilever walls, (3) anchored walls, (4) MSE walls, (5) prefabricated modular walls, and (6) soil nail walls. With the exception of the soil nail walls, the design of each of these wall types for gravity loads was covered within the AASHTO *LRFD Bridge Design Specifications* being used at the time of the Project. In the case of soil nail walls a methodology is outlined for design based on allowable stress methods using computer software commonly used by Department of Transportation (DOT) staff and their consultants for the design of soil nail walls. This use of allowable stress design should be considered an interim approach until AASHTO Specifications are developed for this wall type.

  Section X includes two appendices.

  - **Appendix A_X –** This appendix presents a strategy that the Owner could use to decide on the amount of permanent movement that will be acceptable for a specific retaining structure. A flow chart showing the proposed methodology for establishing the magnitude of permanent displacement is included as Figure A_X -1. A flowchart showing the overall design process for retaining walls (Figure A_X -2) is also included in this appendix.

- **Appendix B_X –** This appendix includes a series of charts that can be used to estimate seismic active and passive earth pressures for sites that are characterized by some apparent cohesion in the soil (i.e., not a clean, cohesionless soil). These charts were developed and presented because many natural slopes include some amount of fines, and this fines content has a significant effect on the seismic earth pressure – leading to lower active pressure values and higher passive pressure values than would be estimated if the cohesive content is not included. This effect is particularly important when estimating seismic passive pressures.

- **Section Y: Slopes and Embankments –** This section provides proposed specifications and commentaries for the seismic design of slopes and embankments. The specifications cover natural slopes and engineered fills. A methodology for addressing sites with liquefaction potential is included in the specifications. AASHTO *LRFD Bridge Design Specifications* being used at the time of the NCHRP 12-70 Project did not provide specific guidance on methods to use when evaluating the stability of slopes under gravity and live loads. In this case the Specifications and Commentaries use the "standard of geotechnical practice" as the starting point for design. This section includes one appendix.

  - **Appendix A_Y –** This appendix contains two flowcharts: (1) one showing a strategy for deciding on acceptable displacements (Figure A_Y-1) and (2) a flowchart showing the overall design process for slopes and embankments (Figure A_Y -2).

- **Section Z: Buried Structures –** This section covers the seismic design of drainage structures and small pedestrian tunnels. This discussion focuses on design for transient ground displacements (TGD) and includes brief mention of the design requirements for permanent ground displacement (PGD). Generally, the ability of the drainage structure or small pedestrian tunnel to withstand permanent ground displacement depends on the amount of permanent ground movement which will occur during the seismic event. Procedures given in Section Y can be used to estimate these displacements. Drainage structures or small pedestrian tunnels will generally move with the ground, and if movements exceed more than few inches, the movements could damage the drainage structure, culvert, or small tunnel. This section includes one appendix.

  - **Appendix A_Z –** This appendix summarizes a strategy that the Owner could use for deciding whether seismic design is required. The appendix includes a flowchart showing the overall design process for buried structures (Figure A_Z -1).

1.3.2 Part 2 – Example Problems

Part 2 contains the example problems. These are organized as follows:

- Retaining Walls
  - Gravity and semi-gravity walls
  - Nongravity cantilever walls
  - Anchored Walls

- MSE Walls
- Soil Nail Walls

- Slopes and Embankments
  - Natural Slopes
  - Fill Slopes
- Buried Structures
  - Oval Structures
  - Box Structures

1.4 Use of Recommended Specifications and Commentaries

The specifications, commentaries, and example problems in this volume of the Draft Final Report were prepared based on literature reviews and evaluations that were performed during the NCHRP 12-70 Project. A number of uncertainties regarding the seismic design of retaining walls, slopes and embankments, and buried structures were identified as the reviews and evaluations were performed and as approaches were developed and tested. In some cases these uncertainties simply could not be adequately investigated within the budget and schedule for the Project, and engineering judgment and experience had to be used during the preparation of the specifications and commentaries. The following subsection provides comments on the general use of the specifications and commentaries. This discussion is followed by specific topics that will require further evaluation to address current uncertainties.

1.4.1 General Use

The approaches identified in these specifications and commentaries have been tested on a limited number of example problems. Additional trial applications will be required to confirm that the recommended approaches are resulting in reasonable design recommendations. In some cases the recommended approaches can be tested against field observations or controlled laboratory experiments, such as by conducting model tests with the centrifuge. However, in many cases engineering judgment will have to be used to decide whether the design seems to make sense. Individuals experienced in the actual design and construction of retaining walls, slopes and embankments, and buried structures will need to be involved in these evaluations.

The reader also should be aware that it is unlikely that these specifications and commentaries will be adopted by AASHTO in their current form. Before implementing all or portions of these recommended specifications and commentaries, AASHTO subcommittee(s) will review and only adopt those sections that are judged by the committee(s) as being suitable for use in the AASHTO *LRFD Bridge Design Specifications*. For this reason some of the recommended specifications and commentaries may be changed substantially or be replaced entirely by different specification language and even specification approaches. Those individual attempting to use these specifications and commentaries must realize this limitation. Until some or all of the recommended specifications and commentaries have been officially adopted by AASHTO, the recommendations in this volume of the Draft Final Report must be treated as the approach recommended by the NCHRP 12-70 Project team. Alternate approaches may be possible and even preferred.

1.4.2 Specific Topics for Further Evaluation

The following specific topics were identified during the preparation of these specifications, commentaries, and example problems as warranting further evaluation and in some cases will require further research. In some cases these topics result in uncertainties in the seismic design; in other cases, the topic represents a departure from the current method being used by AASHTO. Users should keep these specific topics in mind if they attempt to use all or part of the specifications and commentaries. As AASHTO committee members consider these specifications and commentaries, they may want to discuss whether further investigation of the topic is needed or whether an alternate approach could be taken that completely avoids the issue.

1.4.2.1 Ground Motions and Soil Properties

The first series of topics applies to ground motions and soil properties used for the seismic design of retaining walls, slopes and embankment, and buried structures.

- Further testing of the screening methods in Articles X.4.1, Y.5.1, and Z.1 is required. Likewise, recommendations for the wave scattering (incoherence) factor given in Article X.4 may require further evaluation.
- Simplified equations for estimating permanent displacement (Article X.4.5) would be desirable. The current equations are not necessarily easy to adapt into a spreadsheet solution that allows the yield acceleration to be identified for a combination of allowable displacement and peak ground acceleration.
- The amount of apparent cohesion that should be used in design is based on estimated fines content (Article X.5). This requires further evaluation, particularly in terms of the effects of cyclic loading on this contribution.
- The potential effects of shear banding in cohesionless soils on the development of seismic active earth pressures needs further review. This mechanism could limit the magnitude of seismic active earth pressures by controlling the failure plane from which seismic pressures develop.
- Liquefied soil properties are currently defined for level-ground conditions, with the assumption that these properties will also apply for sloping ground. This assumption is known to be inaccurate in some situations. Simplified methods that account for sloping ground effects on liquefaction need to be re-evaluated and confirmed.

1.4.2.2 Retaining Walls

The next series of topics deals with retaining walls. In many cases the topic is applicable to several retaining wall types.

- The limit equilibrium method and the simplified Newmark displacement approach are recognized as simple representations of the response of gravity and semi-gravity retaining walls during seismic loading (Article X.7.3). A simplified displacement-based approach that uses springs to represent the foundation-soil interface for retaining structures is needed, particularly for more accurately evaluating deformations and stresses in the retaining wall. This approach might be analogous to the use of computer programs such as L-Pile or BMCOL for the design of pile foundations subjected to lateral loads.

- For semi-gravity retaining walls the contribution of the soil above the heel of the wall on internal stresses and deformations in the wall needs further evaluation (Article X.7.3.3). The recommended specifications assume that no additional inertial forces from this soil mass occur in the evaluation of wall bending moments and shear forces.
- The influence of the distribution of the seismic coefficient near the top of tall anchored walls needs further evaluation (Article X.9.2.1). For most cases this is not considered a problem, but there could be combinations of soil conditions, slope geometry, and wall stiffness where the distribution of the seismic coefficient could influence bending moments and anchor forces.
- Simplified equations for the determination of seismic passive pressure are needed (Article X.8.2.2). The wedge procedure suggested in Appendix B_X offers an excellent approach, but may be difficult to implement for some users. Ideally, the Caltrans' program CT-FLEX would be made available.
- The distribution of seismic loads to anchors is assumed to be the same as static loading (Article X.9.2.2). This needs further evaluation through numerical modeling, laboratory (centrifuge) experimentation, and field testing.
- The amount of reinforced mass used in the external stability analyses in MSE walls is limited to 0.7 times the wall height (Article X.10.3.2). This length is greater than required by AASHTO, but less than would occur if the entire reinforced zone were included, particularly in the case of tall walls in highly seismic areas where steep backslopes occur above the wall. Numerical or centrifuge studies will be required to evaluate this issue.
- The recommended specifications and commentaries remove the (1.45 – A)A factor currently given in AASHTO (Article X.10.2.2). If this change is adopted as recommended, computer software such as MSEW and ReSSA will need to be modified.
- The distribution of seismic forces within the reinforced zone of MSE walls, as required for internal stability evaluations (Article X.10.3.3), needs to be re-evaluated. Methods recommended in the specifications and commentaries differ from current AASHTO procedures.
- The soil nail wall specifications need to be made consistent with AASHTO *LRFD Bridge Design Specifications* for gravity load, once the specifications for static design of soil nail walls are prepared and adopted by AASHTO.
- The effects of seismic loads on the state of stress behind retaining walls following a large seismic event are also largely unknown, and therefore warrants more review. For example, if relatively large permanent displacements occur for a semi-gravity wall, it is unclear whether the wall will regain its original capacity to resist seismic loads or would

undergo even larger displacements during a similar earthquake in the future. This question could affect the post-earthquake mitigation strategy to adopt.

1.4.2.3 Slopes and Embankments

The next three topics deal with the seismic design of slopes and embankments.

- Methods for evaluating the strength of liquefied soils (Article Y.4) are needed, similar to the comment made for retaining walls. Likewise, the amount of cohesion to allow, particularly if associated with capillarity requires further consideration.
- A recommendation is made to reduce the strength parameters by a factor of 0.9 if the magnitude of the earthquake exceeds 7.5. Common practice in some areas is to apply the 0.9 factor regardless of earthquake magnitude. A consensus approach is required in this areas.
- Further evaluations of the screening levels in Article Y.5.1, including levels for liquefaction, are needed. It is possible that the screening criteria for central and eastern United States might be different than western United States. In this case a velocity-based criteria might be more suitable, similar to the method used for buried structure design.

1.4.2.4 Buried Structures

The final topics deal with buried structures.

- Further evaluations are need to verify that simplified methods for TGD are appropriate for a range of soil types, buried structure types (rigid and flexible), and buried structure geometries.
- Further guidance on simplified approaches to use for TGD with semi-slip should be considered.

1.5 Limitations

The opinions and conclusions expressed or implied in this Draft Final Report are those of the Project Team. They are not necessarily those of the Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or the individual states participating in the National Cooperative Highway Research Program.

Inasmuch as these specifications and commentaries are dependent on the level of ground shaking, the soil conditions at the site, and the methods used for gravity loading design, it is the responsibility of the user to decide whether the methods recommended in these specifications are appropriate and meaningful.

Part 1
Specifications and Commentaries
June 2008

SECTION X: RETAINING WALLS

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X.1 SCOPE

This section provides requirements for the seismic design of retaining walls used in highway construction to stabilize fills or cut slopes but not connected to or forming part of a bridge. These retaining walls, referred to in this section of the Specifications as freestanding retaining walls, shall be designed to withstand seismic loads and displacements associated with the design seismic event in accordance with the requirements of this section of the Specifications.

Before conducting seismic analyses and design evaluations described in this section of the Specifications, freestanding retaining walls shall be designed to satisfy all static service and strength limit requirements specified in Sections 3, 10, and 11 of the AASHTO *LRFD Bridge Design Specifications*.

C.X.1

Freestanding retaining walls stabilize fill soils or cut slopes by providing lateral support or reinforcement. The design process for these retaining walls involves sizing the wall to meet gravity load and live load requirements and then providing a check to confirm that seismic performance is within acceptable limits. The seismic checks include levels of shear and bending stresses in the structural components of the wall, as well as checks on the external and global stability of the wall. If seismic loads result in unacceptable performance in terms of either excessive stresses or displacements, the geometry or detailing of the wall needs to be modified until acceptable performance occurs.

The seismic analysis is conducted by geotechnical and structural engineers. The initial phase of the design is conducted by the geotechnical engineer. This work involves defining

- The soil resistance to loading, referred to as the capacity (C). The capacity depends on soil factors (e.g., shear strength of the soils) and the geometry of the foundation.

- The seismic earth pressure loading condition, referred to as the demand (D). In addition to the seismic earth pressure, the seismic demand can also include the seismic inertial forces and seismic hydrodynamic forces for some types of retaining walls.

The capacity (C) and the demand (D) are used by the geotechnical and the structural engineer to confirm that the retaining wall meets (1) global stability, (2) external stability (i.e., sliding, overturning, and bearing), and (3) internal capacity with respect to shear and moment (also referred to as internal stability in this Section) of structural components. In these analyses

- The global stability involves traditional slope stability evaluations where soil failure occurs below the retaining structure.
- The external stability considers the structure as a rigid body and evaluates sliding, overturning, and bearing for the imposed seismic earth pressures; and
- The internal stability deals with the shear and moment capacity and deformations of the structural system.

Normally, the geotechnical engineer will evaluate global stability, the geotechnical or structural engineer will evaluate external stability, and the structural engineer will determine internal stability. For some wall types, such as anchored walls and mechanically stabilize earth (MSE) walls, the geotechnical engineer will also handle all or portions of the internal stability evaluation. Each designer must determine the optimum approach for addressing these wall design needs.

There is a hierarchy to consider when approaching the seismic design of freestanding retaining walls. From reconnaissance surveys following past earthquakes, retaining walls that suffered structural distress were typically classified as having unacceptable damage, even though these structures did not collapse. However, retaining walls that maintained their structural integrity but underwent measurable permanent displacement and rotation were often classified as having acceptable damage. Emergency repairs could usually be made very quickly to any damage resulting from permanent displacement and rotation, but structural damage usually took much more time and capital investment.

This experience suggests that permanent movement (i.e., sliding and rotation) is acceptable as long as there is no associated structural failure and as long as the displacement does not result in the failure of critical utilities or involve other secondary considerations. Appendix A_X provides a discussion of factors that the Owner should consider when determining acceptable displacements.

From this point of view, the internal stability issue has a much higher hierarchy in the design process – requiring that the designer assure a higher margin of performance for internal stability than for global and external stability. In establishing the seismic design specifications within the scope of this document, specifications have been written to reflect the above experience and concepts.

X.2 DEFINITIONS

A classification system for retaining walls is shown in Figure X.1-1, (FHWA, 1996). Walls are classified according to construction method (i.e., fill construction or cut construction) and basic mechanisms of lateral load support (i.e., externally stabilized or internally stabilized). Fill wall construction refers to a wall system in which the wall is constructed from the base of the wall to the top (i.e., “bottom-up” construction). Cut wall construction refers to a wall in which the wall is constructed from the top of the wall to the base (i.e., “top-down” construction).

It is important to recognize that the “cut” and “fill” designations refer to how the wall is constructed, not necessarily the nature of the earthwork (i.e., cut or fill) associated with the project. For example, a fill wall, such as a prefabricated modular wall, may be used to retain earth for a major highway cut. Externally stabilized wall systems utilize an external structural wall, against which stabilizing forces are mobilized. Internally stabilized wall systems employ reinforcement which extends within and beyond the potential failure mass.

The following wall types are addressed in these Specifications:

- *Rigid Gravity (mass concrete) and Semi-Gravity (standard concrete cantilever) Walls*—These walls (often termed conventional retaining walls) derive their capacity through a combination of the dead weight of the wall and structural resistance.
- *Nongravity Cantilevered Wall*—These walls derive resistance through shear and bending stiffness and embedment of vertical structural elements. These walls can

include pile-supported retaining walls.

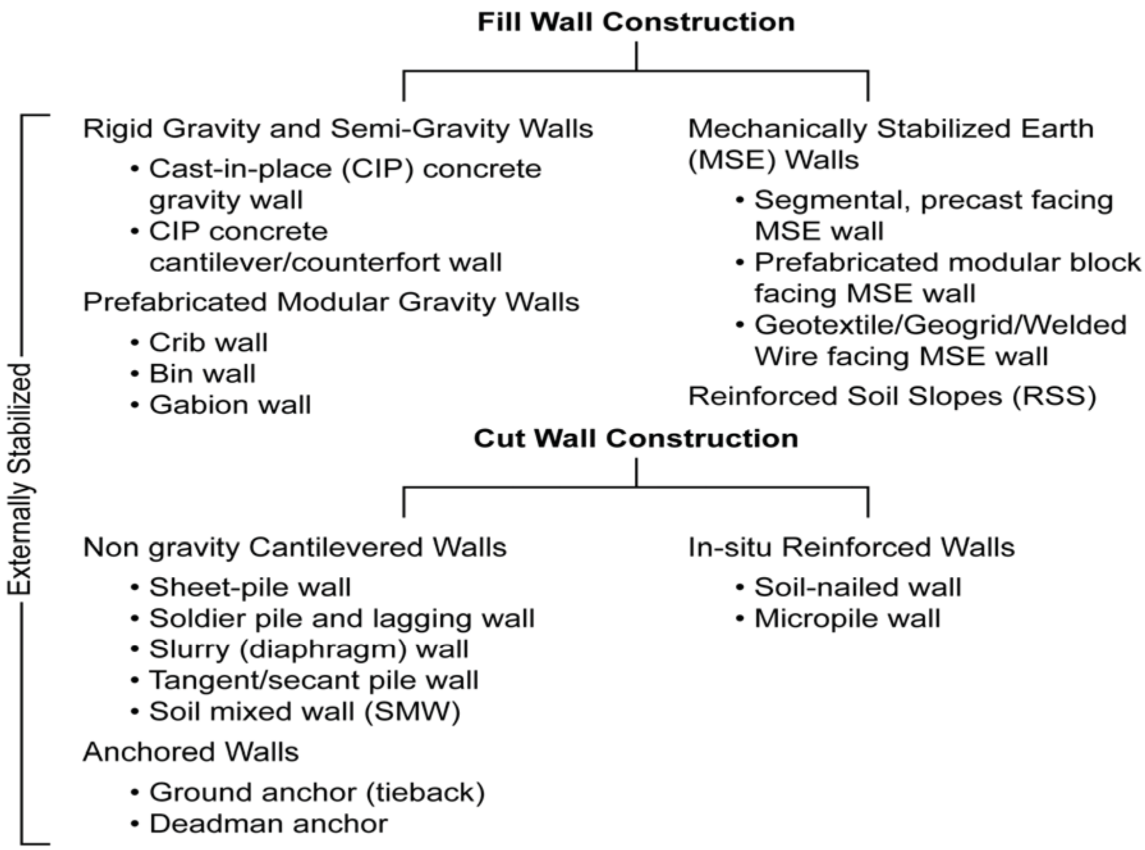


Figure X.1-1 Classification System for Retaining Walls (after FHWA, 1997)

- *Anchored Walls*—These walls derive resistance in a manner similar to a nongravity cantilevered wall, but substantial additional support is obtained through the use of anchors. Anchors may be prestressed tie-backs (ground anchors) which extend from the wall face back to a grouted zone, or they can be deadman anchors which extend from the wall face back to a mechanical anchorage such as a steel sheet pile or concrete block.
- *Mechanically Stabilized Earth (MSE) Walls*—These walls employ either metallic (strip, grid, or wire mesh) or polymer (strip, grid, or sheet) reinforcement in the backfill soil. The metallic or polymer reinforcement resists lateral load through interface shear and passive resistance between the soil and the reinforcement. The reinforcement is connected to a vertical or near-vertical facing.
- *Reinforced Soil Slopes (RSS)*—These systems employ tensile reinforcement in the backfill soil in a manner similar to MSE walls. The inclination of the slope face is typically less than 70 degrees. The reinforcement extends to the slope face and is connected to a facing, where present.
- *Prefabricated Modular Walls*—These walls employ interlocking soil-filled or rock-filled concrete, timber, or steel modules that resist lateral load by acting as a gravity

wall.

- *Soil Nail Walls*—These walls employ metallic bars that are drilled and grouted, or driven into the retained soil mass to develop resistance at each level.

X.3 NOTATION

X.3.1 General

A = area (ft.^{2})
A_e = area in contact during liftoff of footing (ft.^{2})
A_s = peak seismic ground acceleration coefficient modified by zero-period site factor (i.e., $A_s = F_{pga}$ PGA) (dim.)
C = capacity (kip or kip/ft.)
c = soil cohesion (psf.)
D = demand (kip or kip/ft.)
d = displacement (in.)
F_{pga} = site factor for PGA (dim.)
F_a = site factor for short-period spectral acceleration (dim.)
F_v = site factor for spectral acceleration at 1 second (dim.)
g = gravitational acceleration
H = height of retaining wall (ft.)
i = backfill slope angle (degrees)
K_{AE} = seismic active earth pressure coefficient (dim.)
K_{PE} = seismic passive earth pressure coefficient (dim.)
k_h = horizontal seismic coefficient (dim.)
k_{max} = peak seismic coefficient = F_{pga} PGA = A_s
k_{av} = average seismic coefficient after adjustments for wave scattering effects = $\alpha\, k_{max}$ (dim.)
k_y = yield acceleration coefficient for displacement analysis (dim.)
L_{ei} = effective resistance length for MSE wall reinforcing (ft.)
N = Standard Penetration Test (SPT) blowcount (blows/ft.)
N_{avgh} = average SPT resistance for the top 100 ft of soil profile (blows/ft.)
P = load (kip)
P_{AE} = seismic active earth pressure (kip/ft)
P_{PE} = seismic passive earth pressure (kip/ft.)
PGA = peak ground acceleration coefficient on rock (Site Class B) (dim.)
PGV = peak ground velocity at ground surface (in/sec)

Q_i = load effect due to load component i (kip)
R_n = nominal resistance (kip)
R_{ult} = sliding resistance (kip)
S_a = spectral acceleration coefficient
S_s = spectral acceleration coefficient at 0.2 seconds
S_1 = spectral acceleration coefficient at 1 second
S_u = undrained shear strength (psf)
T = period (sec.)
T_p = fundamental period of bridge (sec.)
V_s = shear wave velocity (ft/sec.)

α = fill height reduction factor (dim.)
β = spectrum shape factor = F_vS_1/k_{max} (dim.)
δ = interface friction between wall and soil or foundation base and soil
γ = soil unit weight (kip/ft.^{3})
γ_p = load factor (dim.)
ϕ_r = resistance factor (dim.)
ϕ = soil friction factor (degrees)
θ = arc tan $(k_h/(1 – k_v)$ (degrees)

X.4 SEISMIC LOADS AND LOAD FACTORS

X.4.1 General

The seismic loads for freestanding retaining wall design shall be computed on the basis of the seismic ground motions and adjustment methods described in this section of the Specifications, unless approved or directed otherwise by the Owner.

For sites that are not susceptible to liquefaction, a seismic analysis of a freestanding retaining wall is not required if the site-adjusted peak ground acceleration coefficient (F_{pga} PGA) for the site is less than the values listed in the following table:

C.X.4.1

The method used to compute seismic loads is based on ground motion maps developed by AASHTO. These maps provide estimates of the PGA for both a reference soft rock condition (Site Class B), as well as other site conditions.

The levels of peak ground acceleration the ground surface in some areas will be low enough that a check on seismic loading is not required. For these locations sufficient margin exists within the design for gravity loads that the seismic loads can be accommodated without special provisions.

The level of peak ground acceleration which seismic analyses are not required

Slope Angle Above Wall	F_{pga} PGA
Flat	0.3
3H:1V[1]	0.2
2H:1V	0.1

Sites not requiring a seismic analysis shall meet seismic detailing requirements consistent with requirements in the AASHTO *LRFD Bridge Design Specifications* for the Seismic Zone.

depends on the soil conditions at the site and the geometry of the slope above the wall. Screening values of site-adjusted peak ground acceleration coefficient given in this article assume that the retaining walls are not supported on soils that are liquefiable. If liquefiable soils occur, special studies are required to evaluate the whether liquefaction will occur and if it does, the most suitable retaining wall.

The slope angle used in screening refers to the average angle of the slope above the retaining wall. If the slope is characterized by a non-uniform slope, the average angle of the slope from the face of the wall to one wall height behind the wall should be used for determining the average slope value. Linear interpolation can be used when determining the need for a seismic analysis for slopes between those given in the table.

For locations where a seismic analysis is not required, structures should still meet minimum structural detailing requirements for the seismic zone to assure acceptable performance if seismic ground shaking occurs.

X.4.2 Design Acceleration Values at Ground Surface

The acceleration value used in the design of freestanding retaining walls shall be defined on the basis of the AASHTO ground motion maps and CD adopted by AASHTO in 2007 unless site-specific methods are used to determine the peak acceleration value. The use of site-specific methods for determining ground motions is subject to the approval of the Owner.

The acceleration values determined from the AASHTO maps shall be adjusted for local site effects using the site classes and short period spectral acceleration values given in Table X.4-1. If permissible by the Owner, site-specific dynamic response methods can also be used to estimate the effects of local

C.X.4.2

The AASHTO ground motion maps and CD were developed by U.S. Geological Survey (USGS) for AASHTO to revise the seismic hazard level used in the design of bridges and related structures, including freestanding retaining walls. The revision changes the design hazard level from 10% in 50 years to a hazard level of 7% in 75 years. This revised hazard level corresponds to an approximate earthquake return period of 975 years.

Hazards maps have been prepared for the revised earthquake return period. The basis for the revised maps is provided in the NCHRP 20-07 Report titled *Recommended LRFD Guidelines for the*

1 Horizontal (H) to Vertical (V)

site conditions on the acceleration value used for design.

A load factor γ_p = 1.0 shall be used with the site-adjusted PGA values to compute seismic loads for the Extreme Event I limit state, except as noted in the following sections in these Specifications.

Seismic Design of Highway Bridges prepared by TRC/Imbsen and Associates Inc. (NCHRP, 2006a).

The 2008 AASHTO maps provide PGA and response spectral acceleration coefficients at short and 1 second periods (S_s and S_1) for soft rock (Site Class B) site conditions. The design response spectrum is computed for structural design using a three-point method, as shown in Figure C.X.4-1.

For sites that are not Class B, spectral ordinates are modified by multiplying the reference spectral ordinates for soft rock (i.e., PGA, S_s, and S_1 from the AASHTO maps) by site factors F_{pga}, F_a, and F_v. The site factors are defined in Table X.4-1. The intent of the F_a and F_v factors is to account for modifications to the seismic ground motions that occur between the rock reference condition (Site Class B) and the average soil conditions at a site. The site class is determined by the average shear wave velocity (V_s) over a depth of 100 feet.

The AASHTO maps are available with a ground motion software tool packaged on a CD-ROM for installation on a PC using a Windows-based operating system. The CD facilitates interpretation of the AASHTO maps by allowing the user to calculate the mapped spectral response accelerations as described below:

- **Site Class B (soft rock):** PGA, S_s, and S_1 are determined using the AASHTO CD for the latitude-longitude or zip code of the site. Resulting accelerations are for a reference soft rock (Site Class B) condition.

- **Site Classes A, C, D, and E (hard rock or soil):** PGA, S_s, and S_1 for Site Class B are modified by multiplying the site factors (F_{pga}, F_a and F_v) to define accelerations at sites not characterized by soft rock (Site

Class B).

For some sites or projects the Owner may decide that the AASHTO maps are not appropriate, either because new seismic hazard information is available for a site or a longer return period (i.e., lower frequency of occurrence than 7% in 75 years) is desired for the particular structure. In this case site-specific seismic hazard analyses can be performed to develop site-specific information about the potential for ground shaking at a site.

These site-specific seismic hazard analyses can be based on either deterministic or probabilistic procedures. The decision to use alternate methods of determining the seismic hazard level for a site must be determined in consultation and agreement with the Owner. The proposed AASHTO Guide Specifications for LRFD Seismic Bridge Design (2008) limit the results of site-specific hazard analyses to 2/3rds of the spectrum obtained from the AASHTO maps.

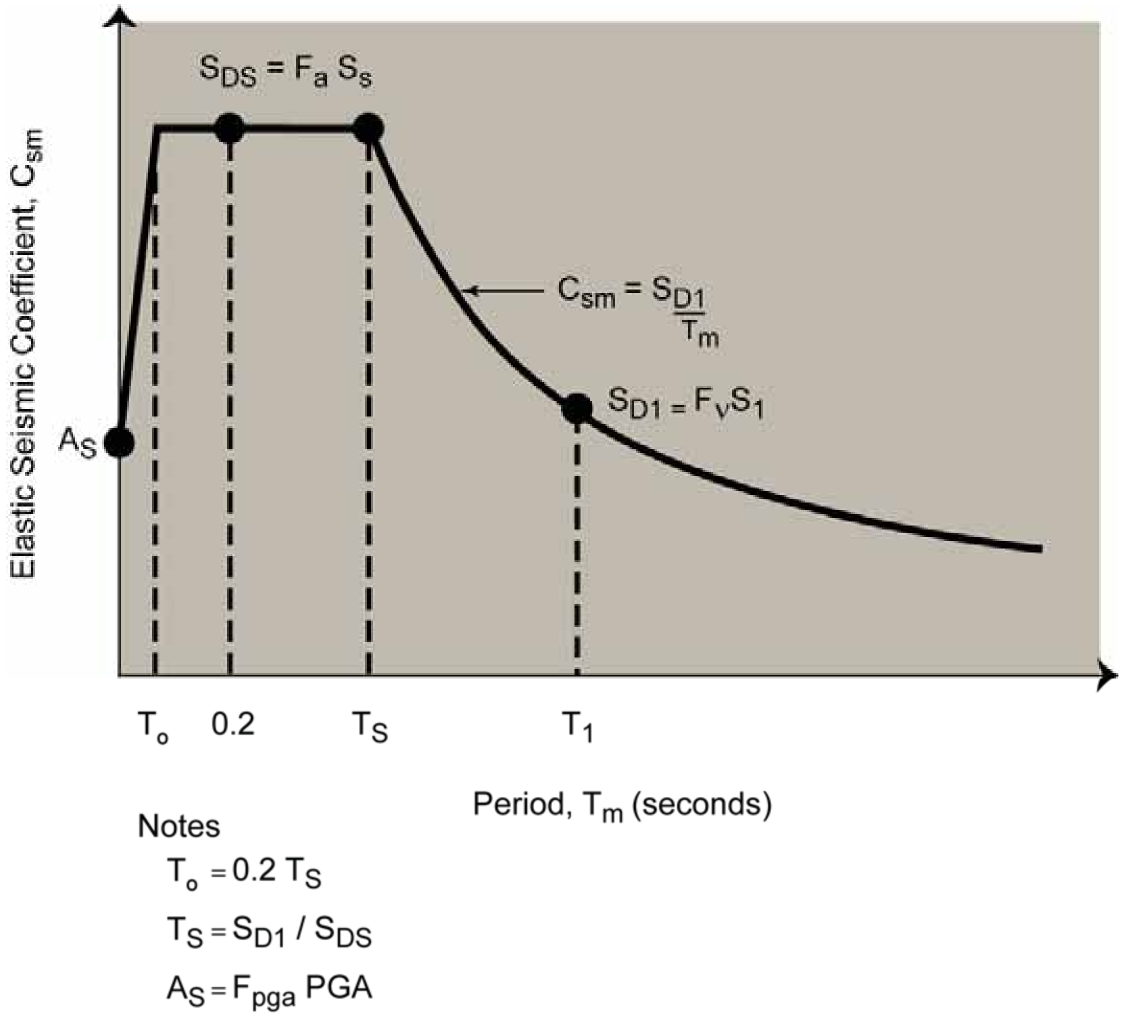


Figure C.X.4-1 Design Response Spectrum Construction Using Three-Point Method

Table X.4-1: Site Classification Categories A-E and Site Coefficients F_{pga}, F_a, and F_v

a) Site Classification Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $V_s > 5{,}000$ ft/sec.
B	Rock with 2,500 ft/sec $< V_s \le 5{,}000$ ft/sec
C	Very dense soil and soil rock with 1,200 ft/sec $< V_s \le 2{,}500$ ft/sec, or with either $N_{avg} > 50$ blows/ft or $S_u \ge 2.0$ ksf
D	Stiff soil with 600 ft/sec $\le V_s \le 1{,}200$ ft/sec, or with either $15 \le N_{avg} \le 50$ blows/ft, or $1.0 \le S_u \le 2.0$ ksf

Site Class	Soil Type and Profile
E	Soil profile with V_s < 600 ft/sec or with either N_{avg} < 15 blows/ft or S_u < 1.0 ksf, or any profile with more than 10 ft of soft clay with PI > 20, m/c > 40% and S_u < 0.5 ksf.
F	Soils requiring site-specific evaluations, such as • Peats or highly organic clays (H > 10 ft or peat or highly organic clays where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H > 120 ft)
Exception:	Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where

V_s = Average shear wave velocity for the upper 100 ft of the soil profile

N_{avg} = Average Standard Penetration Test (SPT) blowcount in blow/ft (ASTM D 1586) for the upper 100 ft of the soil profile

S_u = Average undrained shear strength in ksf (ASTM D 2166 or ASTM D 2850) for the upper 100 ft of the soil profile

PI = Plasticity index (ASTM D 4318)

m/c = Moisture content (ASTM D 2216)

b) Values of Site Factor (F_{pga}) at Zero Period on Acceleration Spectrum

Site Class	Peak Ground Acceleration Coefficient (PGA)[a]				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[b]	*	*	*	*	*

Notes:

a) Use straight line interpolation for intermediate values of PGA.
b) Site-specific geotechnical investigation and dynamic site response analyses should be performed for all sites in Site Class F following the current AASHTO *LRFD Bridge Design Specifications*.

c) Values of Site Factor (F_a) for Short-Period Range of Acceleration Spectrum

Site Class	**Spectral Acceleration Coefficient at Period 0.2 sec (S_s)**[a]				
	$S_s \leq 0.25$	**$S_s = 0.50$**	**$S_s = 0.75$**	**$S_s = 1.00$**	**$S_s \geq 1.25$**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[b]	*	*	*	*	*

Notes:

a) Use straight line interpolation for intermediate values of S_s.
b) Site-specific geotechnical investigation and dynamic site response analyses should be performed for all sites in Site Class F..

d) Values of Site Factor (F_v) for Long-Period Range of Acceleration Spectrum

Site Class	**Spectral Acceleration Coefficient at Period 1.0 sec (S_1)**[a]				
	$S_1 \leq 0.25$	**$S_1 = 0.50$**	**$S_1 = 0.75$**	**$S_1 = 1.00$**	**$S_1 \geq 1.25$**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[b]	*	*	*	*	*

Notes:

a) Use straight line interpolation for intermediate values of S_1.
b) Site-specific geotechnical investigation and dynamic site response analyses should be performed for all sites in Site Class F.

X.4.3 Maximum Seismic Coefficients for Design

The maximum seismic coefficient (k_{max}) for computation of seismic lateral wall loads shall be determined on the basis of the PGA at the ground surface (i.e., $k_{max} = F_{pga}$ PGA),

C.X.4.3

The definition of k_{max} is identical to A_s used in the current AASHTO *LRFD Bridge Design Specifications*. Different terminology is adopted within these except for walls founded on Category A soil (hard rock) where k_{max} shall be based on 1.2 times the site-adjusted peak ground acceleration coefficient (i.e., k_{max} = 1.2 F_{pga} PGA). If permitted by the Owner, wall-height adjustment factors are allowed for walls greater than 20 feet in height. For wall heights greater than 70 feet, special seismic design studies shall be performed.

proposed Specifications to be consistent with historic use of “k” in the evaluation of seismic earth pressures.

The designer can conservatively k_{max} for design; however, various studies have shown that the ground motions in the mass of soil behind the wall will often be lower than the k_{max} at the ground surface, particularly for taller walls. The following discussions outline the adjustment that can be used by the designer to account for this effect and the rationale for the adjustment.

Height-Dependent Adjustments for Wall Heights from 20 to 70 feet

For values of H greater than 20 feet but less than 70 feet, the seismic coefficient used to compute lateral loads acting on a freestanding retaining wall may be modified to account for the effects of spatially varying ground motions behind the wall, using the following equation:

$$k_{av} = \alpha\, k_{max} \qquad \text{(C.X.4.3-1)}$$

where

k_{max} = F_{pga} PGA

α = fill height reduction factor

For Site Category C, D, and E

$$\alpha = 1 + 0.01H\,[(0.5\beta) - 1] \qquad \text{C.X.4.3-2)}$$

where

H = fill height (feet)

β = $F_v S_1 / k_{max}$

For Site Category A and B (hard and soft rock foundation soils), the values of α given by Equation C.X.4.3-1 is increased by a factor of 1.2.

Height-Dependent Adjustments for Wall Heights > 70 feet

For wall heights greater than 70 feet, special seismic design studies involving the use of numerical models should be conducted. These special studies are required in view of the potential consequences of failure of these very tall walls, as well as limitations in the simplified wave scattering methodology.

Basis for Height Adjustment Factor

The basis for the height-dependent reduction factor described above is related to the response of the soil mass behind the retaining wall. Common practice in selecting the seismic coefficient for retaining wall design has been to assume rigid body soil response in the backfill behind a retaining wall. In this approach the maximum seismic coefficient (k_{max}) is assumed equal to the F_{pga} PGA when evaluating lateral forces acting on an active pressure failure zone. Whereas this assumption may be reasonable for wall heights less than about 20 feet, for higher walls, the magnitude of accelerations in soils behind the wall will vary spatially as shown schematically in Figure C.X.4-2.

The nature and variation of the incoherent ground motions is complex and will be influenced by the dynamic response of the wall-soil system to the input earthquake ground motions. In addition to wall height the acceleration distribution will depend on factors such as the frequency characteristics of the input ground motions, the stiffness contrast between backfill and foundation soils, and wall slope. From a design standpoint, the net effect of the spatially varying ground motions can be represented by an averaging process over a potential active pressure zone, leading to a time history of average acceleration and hence a maximum average acceleration or seismic coefficient as shown in Figure C.X.4-2.

To evaluate this averaging process, the results of a series of analytical studies are documented in the NCHRP 12-70 Report (NCHRP, 2008). An evaluation of these results forms the basis for the simplified equations C.X.4.3-1 and C.X.4.3-2. The analytical studies included wave scattering analyses assuming elastic soil media using different wall heights and slopes and a range of earthquake time histories. The acceleration time histories simulated spectral shapes representative of Western United States (WUS) and Central and Eastern United States (CEUS) sites and reflected different earthquake magnitudes and site conditions.

Additional height-dependent, one-dimensional SHAKE (Schnaebel et al., 1972) analyses were also conducted to evaluate the influence of nonlinear soil behavior and stiffness contrasts between backfill and foundation soils. These studies were also calibrated against finite element studies for MSE walls documented by Segrestin and Bastick (1988), which form the basis for the average maximum acceleration equation (a function of A_s) given in the current AASHTO *LRFD Bridge Design Specifications* for MSE walls.

The results of these studies demonstrate that the ratio of the maximum average seismic coefficient (k_{av}) to k_{max} (the α factor) is primarily dependent on the wall height and the shape of the acceleration spectra (the β factor). The acceleration level has a lesser effect. It was also found that equations C.X.4.3-1 and C.X.4.3-2 could be applied to slopes.

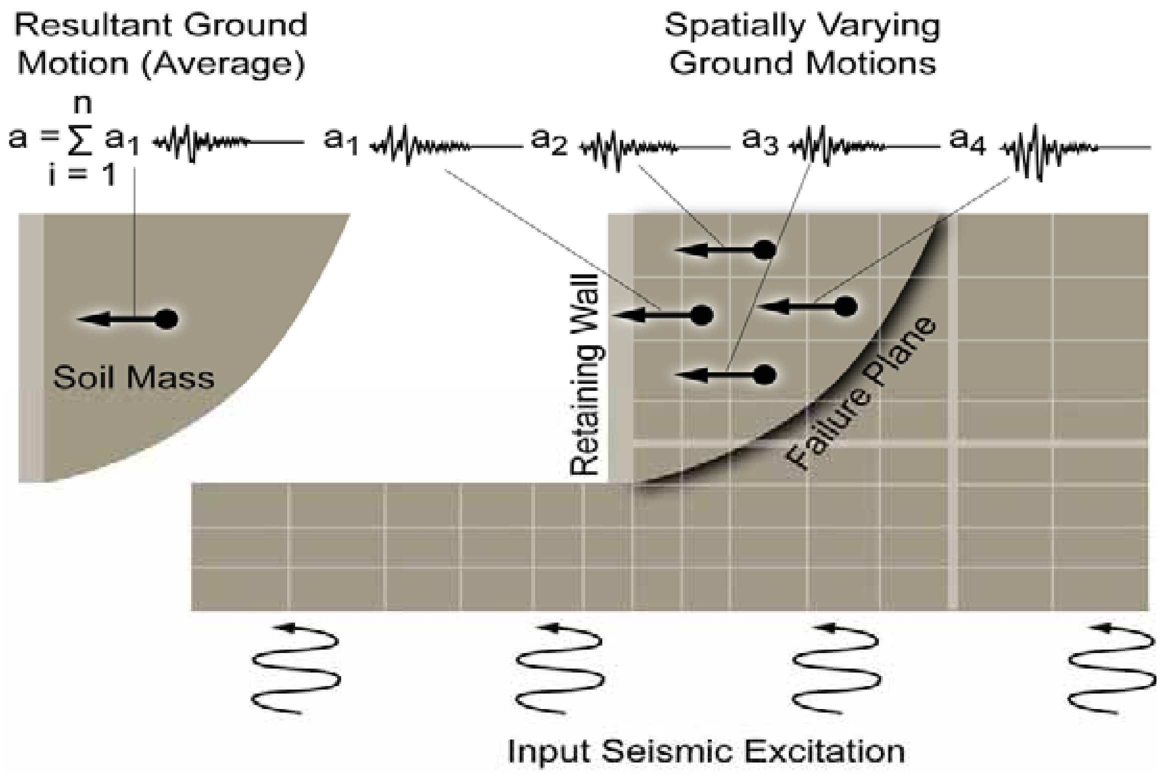


Figure C.X.4-2 Average Seismic Coefficient Concept

X.4.4 Displacement-Related Seismic Coefficient Reduction for Gravity Walls

Where limited permanent displacement of a freestanding retaining wall is allowed by the Owner, a 50% reduction in the maximum seismic coefficient (k_{max}) shall be permitted when determining the seismic coefficient used to compute seismic lateral earth pressure wall loads for gravity walls (i.e., rigid and semi-rigid gravity walls, MSE walls, modular block walls, and soil nail walls).

Use of the 50% reduction for other retaining walls (e.g., nongravity cantilever and anchored) shall be permitted if analyses demonstrate that the displacements associated with a 50% reduction do not result in (1) yield of structural members making up the wall, such with a pile-supported wall, or (2) overloading of lateral support systems, such as the ground anchors.

Where reductions for wave scattering

C.X.4.4

If the maximum average seismic coefficient k_{max} is used for gravity wall design, the size of wall structures may be excessive if design is based on limit equilibrium principles with zero wall displacement. The concept of designing for a small tolerable horizontal displacement is described in Appendix Section 11.1.1.2 of the current AASHTO *LRFD Bridge Design Specifications*. The concept is based on the Newmark sliding block analogy (Newmark, 1965), where incremental wall sliding displacements occur when horizontal accelerations exceed a yield acceleration k_y corresponding to an acceleration level for a sliding factor of safety of 1.0.

The Newmark sliding block concept (Newmark, 1965) was originally developed to evaluate seismic slope effects are permitted by the Owner, the 50% reduction in the maximum seismic coefficient shall be allowed after adjustments for wave scattering.

stability in terms of earthquake-induced slope displacement as opposed to a factor of safety against yield under peak slope accelerations. The concept is illustrated in Figure C.X.4-3, where a double integration procedure on accelerations exceeding the yield acceleration of the slope leads to an accumulated downslope displacement.

The concept of allowing gravity walls to slide during earthquake loading and displacement-based design (i.e., assuming a Newmark sliding block analysis to compute displacements when accelerations exceed the horizontal limiting equilibrium, yield acceleration for the wall-backfill system) was introduced by Richards and Elms (1979). Based on this concept, Elms and Martin (1979) suggested that a design acceleration coefficient of 0.5 PGA would be adequate for limit equilibrium pseudo-static design, provided allowance be made for a horizontal wall displacement in inches of 10 times the PGA. This concept was adopted by AASHTO in 1992, and is reflected in the current AASHTO *LRFD Bridge Design Specifications*. [The PGA term in Elms and Martin is equivalent to the F_{pga} PGA or k_{max} in these proposed Specifications.]

Use of a seismic coefficient of 0.5 times the k_{max} for computation of earth pressure loads is also adopted in these Specifications as further discussed in Article X.7. Recent work completed as part of the NCHRP 12-70 Project (as discussed in Article X.4.5) concluded that the amount of permanent ground displacement associated with the Newmark method is, however, less than previously used. In most cases the amount of movement associated with 0.5 k_{max} will be less than 1 to 2 inches.

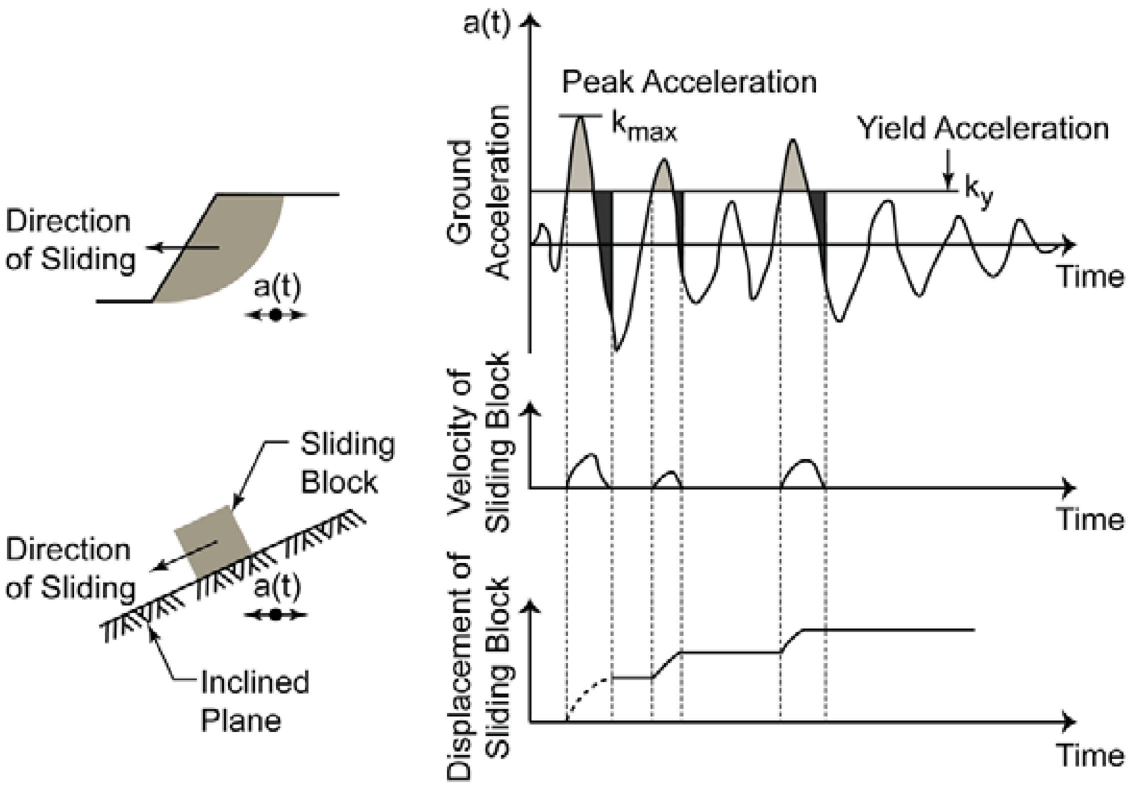


Figure C.X.4-3 Newmark Sliding Block Concept

X.4.5 Newmark Displacement Estimates

When computing permanent displacement of the freestanding retaining walls, the proposed method of computing seismic deformations shall be reviewed with the Owner to confirm that the proposed method is acceptable to the Owner. A maximum acceptable level of permanent ground displacement shall be established based on the Owner's minimum performance expectation for the retaining wall. Appendix A_x to this section provides a strategy for Owner decision-making on the amount of acceptable displacements.

C.X.4.5

Various methods can be used to estimate permanent displacements of gravity retaining structures for walls that can move without damaging either adjacent facilities or components of the wall. These methods range from simple equations or charts based on the Newmark method to using numerical modeling. For many situations simple equations or charts will be sufficient; however, as the complexity of the site or the wall-soil system increases, more rigorous numerical modeling methods become advantageous.

Current AASHTO Equation for Displacement Estimates

The current AASHTO approach is further discussed in the NCHRP 12-70 Report (NCHRP, 2008) where it is noted that the current AASHTO displacement equation (in inches) is given as:

$$d = 0.087(PGV)^2/k_{max}g\ (k_y/k_{max})^{-4} \quad \text{(C.X.4.5-1)}$$

Revised AASHTO Equation for Displacement Estimates

Additional sliding block displacement analyses were conducted as part of the NCHRP 12-70 Project using an extensive database of earthquake records. The objective of these analyses was to establish updated relationships between wall displacement (d) and the following three terms: the ratio k_y/k_{max}, k_{max} , and PGV. Based on regression analyses, the following simplified relationships were established and are recommended for design purposes:

For all sites except CEUS rock sites (Categories A and B), the displacement (in inches) can be estimated by the following equation:

$$\log(d) = -1.51 - 0.74 \log(k_y/k_{max}) + 3.27 \log(1-k_y/k_{max}) - 0.80 \log(k_{max}) + 1.59 \log(PGV) \quad \text{(C.X.4.5-2)}$$

For CEUS rock sites (Categories A and B), displacement (in inches) can be estimated by:

$$\log(d) = -1.31- 0.93 \log(k_y/k_{max}) + 4.52 \log(1-k_y/k_{max}) - 0.46 \log(k_{max}) + 1.12 \log(PGV) \quad \text{(C.X.4.5-3)}$$

Figures C.X.4-4 shows a comparison between the displacements estimated using the old and new equations. Note that the above displacement equations represent mean values, and can be multiplied by 2 to obtain an 84 percent confidence level.

Similar displacement equations to those recommended in Equations C.X.4.5-2 and C.X.4.5-3 were developed by Martin and Qiu (1994) from a more limited database of earthquake records, and were described in the NCHRP 12-49 Project (NCHRP, 2003). The recommended equations give displacements slightly greater than the Martin and Qiu (1994) correlations.

PGV Equation

In Equations C.X.4.5-2 and C.X.4.5-3 it is necessary to estimate the peak ground velocity (PGV) and the yield acceleration (k_y). Values of PGV can be determined using the following correlation between PGV and spectral ordinates at one second (S_1) for Site Class B.

$$PGV\ (in/sec) = 55\ F_v\ S_1 \quad \text{(C.X.4.5-4}$$

where S_1 is the spectral acceleration at 1 second and F_v is the Site Class adjustment for Site Class B.

The development of the PGV-S_1 correlation is based on a simplification after regression analyses conducted on an extensive earthquake database established from recorded and synthetic accelerograms representative of both rock and soil conditions for the WUS and CEUS. The study is described in the NCHRP 12-70 Report (NCHRP, 2008). It was found that earthquake magnitude need not be explicitly included in the correlation, as its influence on PGV is captured by its influence on the value of S_1. The equation is based on the mean plus one standard deviation from the simplification of the regression analysis. (1.46 x the median) for conservatism. Based on Equation C.X.4.5-4, illustrative values of PGV for Site Class B (i.e., F_v = 1.0) are as follows:

$S_1 = 0.1$: PGV = 5.5 in/sec

$S_1 = 0.5$: PGV = 27.5 in/sec

Yield Acceleration (k_y)

Values of the yield acceleration (k_y) can be established by computing the seismic coefficient for global stability that results in a C/D ratio of 1.0 (i.e., FS = 1.0). A conventional slope stability program is normally used to determine the yield acceleration. For these analyses the total stress (undrained) strength parameters of the soil should usually be used in the stability analysis, as discussed in the next section.

Alternate Methods of Estimating Permanent Displacement

The revised Newmark equations given above present a simplified method of estimating the displacements that will occur if the C/D ratio for a limiting equilibrium stability analysis is less than 1.0. Alternate methods of estimating permanent displacements can also be used. The most common of the alternatives involves using the computer programs FLAC (Itasca, 2007) or PLAXIS (PLAXIS BV, 2007). Both software packages allow seismic time history analysis of a 2-dimensional model of the soil cross-section. Such models require considerable expertise in the set-up and interpretation of model results, particularly relative to the selection of strength parameters consistent with seismic loading. For this reason use of this alternate approach should be adopted only with the Owner's concurrence.

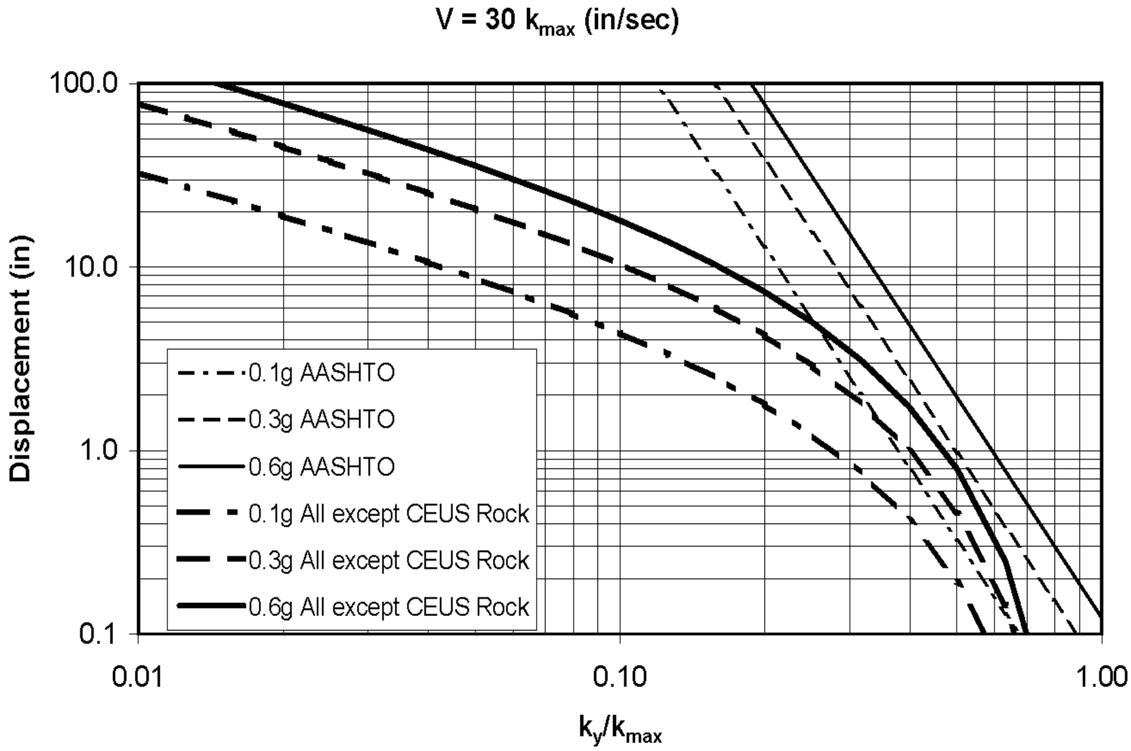


Figure CX.4-4 Comparison between AASHTO (2004) and Recommended Displacement Equation C.X.4.5-2 for PGV = 30 k_{max} (in/sec)

X.5 SOIL PROPERTIES

For competent soils that do not undergo strength degradation under seismic loading, static strength parameters shall be used for seismic design.

- For cohesive soils total stress strength parameters based on undrained tests shall be used during the seismic analysis.
- For clean cohesionless soils, the effective stress friction angle of the soil shall be used.

For saturated, sensitive cohesive soils or saturated cohesionless soils, the potential for earthquake-induced strength degradation shall be considered.

The static design of retaining walls in Section 11 of the AASHTO *LRFD Bridge Design Specifications* is based on the use of effective stress strength parameters. For conservatism the effective cohesion intercept is normally neglected and only the effective friction angle is used. The use of effective stress strength parameters is appropriate for design of retaining walls for long-term, gravity loads. However, for transient seismic loading, total stress parameters are more appropriate for cohesive soils.

Selection of Strength Parameters

Seismic stability analyses for retaining walls require the determination of strength parameters (c and ϕ) for either or both compacted fill and natural soils. In the case of fill wall construction, specifications for wall construction usually require backfill materials to be cohesionless and free draining materials (i.e., amount of soil passing the No. 200 sieve less than 5 to 10%). For these soils cohesion (c) is assumed to be zero, and the effective (drained) friction angle (ϕ') should be used to characterize the soil strength parameters. This strength can be obtained by conducting effective stress, or drained, laboratory strength tests or through the use of empirical correlations to field measurements, such as the Standard Penetration Test (SPT) blowcount or the cone penetration test (CPT) end resistance.

For wall construction involving cuts in natural ground, a high likelihood of encountering soils with cohesive content exists. The undrained (total stress) strength parameters should be used to characterize these soils for seismic loading analyses. The undrained strength can be determined on the basis of total stress strength parameters by in situ testing (e.g., vane shear tests), or through empirical correlations to results of CPT soundings.

In some geographic areas the availability and cost of clean granular backfill soil is becoming a significant construction issue, and backfill soils with a cohesion component due to fines content are increasingly being used. Gravity walls which involve the use of these "dirty" granular backfill soils may also require determination of total stress strength parameters for evaluation of wall design requirements.

Additional information regarding the characterization of soil strength by field and laboratory testing methods is provided in Section 10 of the current AASHTO *LRFD Bridge Design Specifications* and within SCEC (2002).

Contributions from Soil Capillarity – Cohesionless Soils

In many situations it may be appropriate to include the effects of apparent cohesion from soil capillarity in the assignment of strength properties for the seismic loading analyses. This contribution will occur in many relatively clean sands or silts above the water table. The magnitude of apparent cohesion is difficult to establish without conducting special field and laboratory tests, as discussed, for example, by Fredlund and Rahardjo (1993). For this reason the following conservative guidelines are suggested, in the absence of specific testing that demonstrates higher values of apparent cohesion from capillarity.

Percent Passing No. 200 Sieve (%)	Maximum Allowed Apparent Cohesion from Capillarity (psf)
5 - 15	50
15 - 25	100
25 - 50	200

Note that for backfill materials characterized by large particle sizes (e.g., gravels, quarry spalls, or larger particles) the effects of apparent cohesion from capillarity stresses should be ignored. Silts and sands permanently located below the water table, or where fluctuations in water table occur, also should not include apparent cohesion from capillarity. In these locations either capillarity will not develop or cannot be reliably included in the analysis.

Influence of Slope Geometry on Soil Parameter Evaluation

The presence of cohesive soils often leads to steep cut face during construction – with angles of 1H:1V or steeper. For these conditions the use of simple Coulomb theory for seismic active pressure computations as shown in Figure X.5-1 becomes problematic, as critical failure surfaces leading to maximum seismic active pressures that develop. For this situation, total stress strength parameters (c and ϕ) for the natural ground need to be established. If the fill immediately behind the wall is a clean granular backfill (with little or not cohesion), then the seismic design must consider strengths from both drained (clean granular backfill) and undrained (natural soil) when evaluating wall stability.

<u>*Liquefiable Soils*</u>

For soils that are susceptible to liquefaction, the most common method of determining the strength of liquefied soil (often referred to as the residual strength) involves use of Standard Penetration Test (SPT) correlations to liquefied strength or similar Cone Penetrometer Test (CPT) correlations. These correlations are documented by Seed and Harder (1990), Olson and Stark (2002), and Idriss and Boulanger (2007). In view of the various factors that affect the strength of liquefied soil, it is important to establish potential variations in the liquefied strength, and then use this variation during the retaining wall analysis.

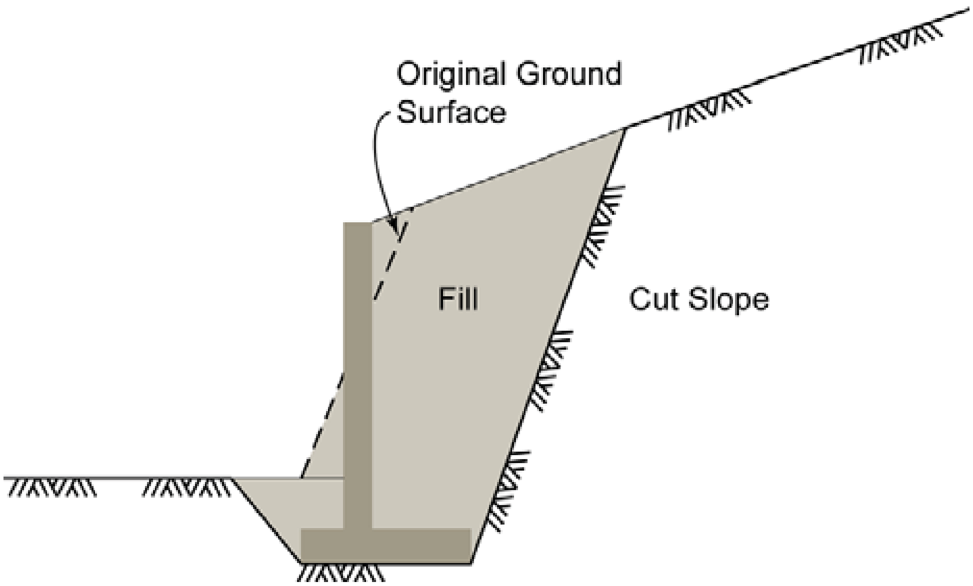


Figure C.X.5-1 Typical Cross-Section of Semi-Gravity Retaining Wall next to Steep Cut Slope

X.6 LIMIT STATES AND RESISTANCE FACTORS

X.6.1 General

Retaining walls located in seismically active areas shall apply load and resistance factors consistent with Extreme Event I:

- Load factors shall be 1.0 for loading conditions defined in Table 3.4.1
- Resistance factors shall be 1.0, unless defined otherwise in this section of the Specifications.

The load factor for live loads in Extreme Event I (per AASHTO *LRFD Bridge Design Specifications* Section 3) shall be determined on a project-specific basis, except where the retaining wall supports a heavily traveled roadway. For this case live loads shall be included in seismic design, and the load factor (γ_p) for live load shall be at least equal to 0.5.

C.X.6.1

The use of a load factor of 1.0 is consistent with standard practice in earthquake design. This load case is an extreme event with a low likelihood of occurrence. The addition of a load factor would be equivalent to adding conservatism to an event that is already expected only once every 1,000 years.

In many cases the live load for a free-standing retaining wall during seismic loading will be taken as 0, under the assumption that the occurrence of the design earthquake and the live load will be a very unlikely event. However, in situations where the live load results from traffic on a heavily traveled roadway, the live load should be included in seismic design.

A resistance factor of 1.0 is recommended for the extreme event limit state in view of the unlikely occurrence of the loading associated with the design earthquake. The choice of 1.0 is influenced by the following factors:

- For competent soils that are not expected to degrade in strength during seismic loading (e.g., due to partial or total liquefaction of saturated cohesionless soils or to strength reduction of sensitive clays), the use of static strengths as recommended for seismic loading is usually conservative, as rate-of-loading effects tend to increase soil strength for transient loading.

- Earthquake loads are transient in nature, and hence if soil yield occurs, the net effect is an accumulated small deformation as opposed to potential foundation failure under static loading. For this situation it is also assumed that stability checks show that global stability is adequate.

- By not reducing the capacity by a resistance factor, the designer has a better understanding of the capacity-to-demand (C/D) ratio, and can decide whether the proposed design provides adequate protection to the public.

Using a resistance factor of 1.0 for soil assumes ductile behavior. While this is a correct assumption for many soils, it is inappropriate for brittle soils where there is a significant post-peak strength loss. For such conditions special studies will be required to determine the appropriate combination of resistance factor and soil strength.

X.7 GRAVITY AND SEMI-GRAVITY WALLS

X.7.1 General

Gravity and semi-gravity walls shall be designed for seismic loading except where

C.X.7.1

The seismic design of a gravity or semi-gravity retaining wall can be an conditions given in Article X.4.1 are satisfied or where allowed otherwise by the Owner.

Before conducting the seismic evaluation, the gravity and semi-gravity wall shall be designed to meet all gravity and live load requirements in accordance with the provisions in Sections 3, 5, 6, and 11 of the AASHTO *LRFD Bridge Design Specifications*.

important component of the transportation network, particularly where

- failure of the wall would limit use of a critical lifeline transportation route
- failure could pose a significant risk to the safety of the public, or
- failure of the wall could damage another transportation facility that must stay in operation following the design seismic event

As part of the seismic design process, the designer must determine an acceptable level of performance of the wall consistent with the Owner's design philosophy.

Gravity and semi-gravity walls are often located in areas where grade separations are required or where the topography of the roadway alignment requires cuts or fills within existing hillsides. The desired performance of such walls during a design seismic event can range from allowing damage or even failure of the wall to requiring damage-free, post-earthquake conditions. In many cases a well-designed gravity or semi-gravity wall could slide several inches and perhaps even a foot or more, as well as tilt several degrees, without affecting the function of the wall.

A number of factors go into the determination of the wall performance criteria for a rigid or semi-rigid gravity retaining wall. These include

- Wall location and function
- Wall type
- Wall geometry
- Types of soils
- Implications of wall movement

Appendix A_x identifies a strategy that can be followed by the Owner when establishing performance levels for a rigid or semi-rigid retaining wall.

The starting point for the seismic design of the rigid and semi-rigid retaining walls is an acceptable static design, meeting the requirements of the current AASHTO *LRFD Bridge Design Specifications*. Once the static design has been completed, the wall is checked for global, external, and internal stability under seismic loading.

X.7.2 Methods of Analysis

During seismic loading, gravity and semi-gravity retaining walls shall resist the forces due to seismic earth pressures without excessive sliding or rotation of the structures or structural failure of the wall. Either limit equilibrium or displacement methods of analysis as described herein shall be used to establish that performance is acceptable during the design earthquake.

C.X.7.2

The seismic evaluation of rigid and semi-rigid retaining walls involves a comparison between the capacity of the foundation system (i.e., soil and structure) to resist seismic loads and the force demands resulting from seismic loads. The capacity of the foundation system to resist loads results from shearing resistance at the base of the structure, the passive soil resistance at the toe of the foundation, and the flexural rigidity of the structural system. The demand on the foundation system includes the seismic active earth pressure behind the wall and the inertial response of the structure. While no explicit determination of displacement is made, the value of k_{max} may be chosen taking into consideration the level of acceptable displacement, at least in a general sense.

The comparison of capacity-to-demand (C/D ratio) is normally made by evaluating sliding along the base of the foundation, overturning of the structure, and a bearing failure mechanism. These mechanisms are treated independently. This treatment represents a simplification of the actual response of the foundation. For most designs this simplification will be adequate. For those designs where the geometry of the structure or soil is complex, it may be necessary to conduct more rigorous computer analyses to understand the performance of the rigid or semi-rigid retaining wall during seismic loading.

If limit equilibrium methods are used to evaluate the C/D ratio, seismic active and passive earth pressures must be determined. Successful design is normally assumed to occur if the C/D is greater than 1.0. No explicit determination of displacement is made with this approach. It is assumed that displacements are large enough to mobilize active earth pressures but not necessarily enough to mobilize passive pressures.

The alternate approach to wall design involves displacement-based methods. A displacement-based approach carries the analysis one step further by including a quantitative estimate of displacement as part of the earth pressure estimate. This approach has inherent advantages in terms of providing the Owner with an understanding of the potential movements associated with a design event. To obtain meaningful estimates of displacements, both the load and resistance factors are set equal to 1.0 for displacement-based analyses.

The displacement-based approach can be conducted using the simplified methods described in Article X.4. An alternate approach involving the use of 2-dimensional finite element or finite difference computer programs is also acceptable. As noted in Article X.4, considerable skill and experience are required when using numerical methods, particular when seismic loading is involved. Most often these numerical methods are suitable for special studies. Before using this alternate approach for seismic design of gravity or semi-gravity walls, detailed discussions should take place with the Owner. The intent of these discussions should be to review the proposed methods of analysis, the assumptions that will be made regarding soil parameters, and the likely reliability of the results. With this approach the Owner also must identify performance expectations for the wall under the imposed seismic loads.

X.7.2.1 Seismic Active and Passive Earth Pressure Determination

Seismic active and passive earth pressures for rigid gravity and semi-gravity retaining walls shall be determined following the methods described in this Article. Site conditions, soil and retaining wall geometry, and the earthquake ground motion determined for the site shall be considered when selecting the most appropriate method to use.

C.X.7.2.1

The suitability of the method used to determine active and passive earth pressures should be determined after a review of features making up the static design, such as backfill soils and slope above the retaining wall. These conditions, along with the ground motion for a site, will determine the appropriate method for estimating seismic active and passive pressures.

Past editions of the AASHTO Specifications relied on the Mononobe-Okabe (M-O) equations for determination of seismic active and passive earth pressure. However, these equations apply to very specific situations.

- The equation for seismic active earth pressure is applicable for uniform, cohesionless backfill soils, and these conditions may not exist. For combinations of high ground motion and steep slopes above the wall, the M-O equation also gives unrealistically high estimates of active earth pressure. For this reason either a wedge equilibrium or a generalized limit equilibrium approach is given as a more suitable alternative.

- For seismic passive pressures the M-O equation has a similar limitation relative to soil conditions, but also relies on Coulomb theory and this approach can overestimate the seismic passive earth pressures.

For this reason the M-O equation for passive earth pressures is not used, and rather a log spiral approach is given.

X.7.2.1.1 M-O Active Earth Pressure Equation

The M-O equation given below shall be an acceptable method for determination of seismic active earth pressures only where (1) the material behind the wall is a uniform, cohesionless soil within a zone defined by a 3H:1V wedge from the heel of the wall and (2) the combination of peak ground acceleration and backslope angle do not exceed the limits shown in Figure X.7-2.

$$P_{AE} = 0.5\,\gamma\,H^2\,K_{AE} \qquad \text{(X.7-1)}$$

where the terms are defined in Figure X.7-1.

The seismic coefficient (k_h) in the M-O active pressure equation shall be defined using the methods given in Article X.4, where a displacement-based reduction of 50% in k_{max} is permitted. The height of wall shall be taken as the distance from the heel of the retaining structure to the ground surface directly above the heel. The equivalent pressure representing the total static and seismic active force (P_{AE}) shall be distributed uniformly over the wall height when used for internal and external stability evaluations.

C.X.7.2.1

The M-O equation for seismic active earth pressure is based on the Coulomb earth pressure principle for a homogeneous, cohesionless backfill. The M-O equation for seismic active earth pressure has been shown to provide a reasonable estimate of the total seismic active earth pressure when the backfill is homogenous. The distribution of total active earth pressure can vary depending on such factors as the mode and magnitude of wall movement, wall friction angle, and seismic acceleration levels.

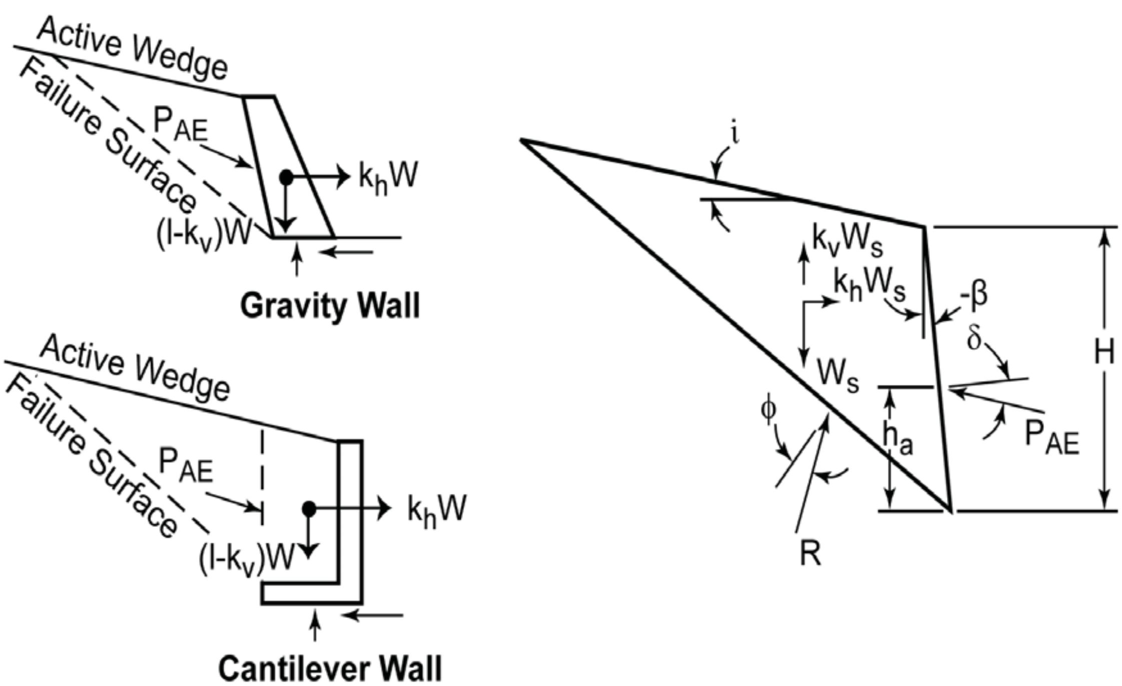
The assumption of a uniform pressure distribution is a practical compromise given the variability of experimental and analytical information reported in the literature, and provides a degree of design conservatism. Past practice of separating static and seismic components and using an inverted trapezoid for the seismic component is inappropriate when different load and resistance factors are used for static and seismic design.

Limitations of the M-O equation are discussed in NCHRP (2008), as well as many other reports and articles. The following additional factors must be considered when using the M-O equation for determining seismic active earth pressures.

<u>Seismic Coefficient</u>

The seismic coefficient (k_h) in Equation C.X.7.2-1 is the site-adjusted peak ground surface acceleration identified in Article X.4 (i.e., k_{max}) after adjustments for (1) wave scattering effects and (2) limited amounts (e.g., 1 to

2 inches) of permanent deformation as determined appropriate. Where the wall height is greater than 20 feet, k_{av} can be used in place of k_{max} to introduced wave scattering effects. The allowance for displacement assumes that provisions are made to assure that internal stability of the wall (moment and shear capacity) is satisfied under the seismic active earth pressure (see Article X.7.3.3). In this equation the vertical acceleration coefficient (k_v) is normally assumed to be zero for design.



$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)} \text{x} \left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^{-2} \quad \text{C.X.7.2-1}$$

Where

γ = unit weight of soil (ksf)

H = height of wall (ft)

ϕ = friction angle of soil (^{o})

θ = arc tan ($k_h/(1 - k_v)$) (^{o})

δ = angle of friction between soil and wall (^{o})

k_h = horizontal acceleration coefficient (dim.) = k_{max} or k_{av} if the wall height is greater than 20 ft

k_v = vertical acceleration coefficient (dim.)

i = backfill slope angle (^{o})

β = slope of wall to the vertical, negative as shown (^{o})

Figure X.7-1 Mononobe-Okabe

Slope of Failure Plane

The M-O equation is based on the assumption that the soil within the active soil wedge during seismic loading is a homogeneous, cohesionless material. For many situations gravity and semi-gravity walls are constructed by cutting into an existing slope where the soil properties differ from the backfill that is used behind the retaining wall. In situations where soil conditions are not homogeneous and the failure surface is flatter than the native slope (as shown in Figure X.7-2), seismic active earth pressures computed for the M-O equation using the backfill properties may no longer be valid, particularly if there is a significant difference in properties between the native and backfill soils.

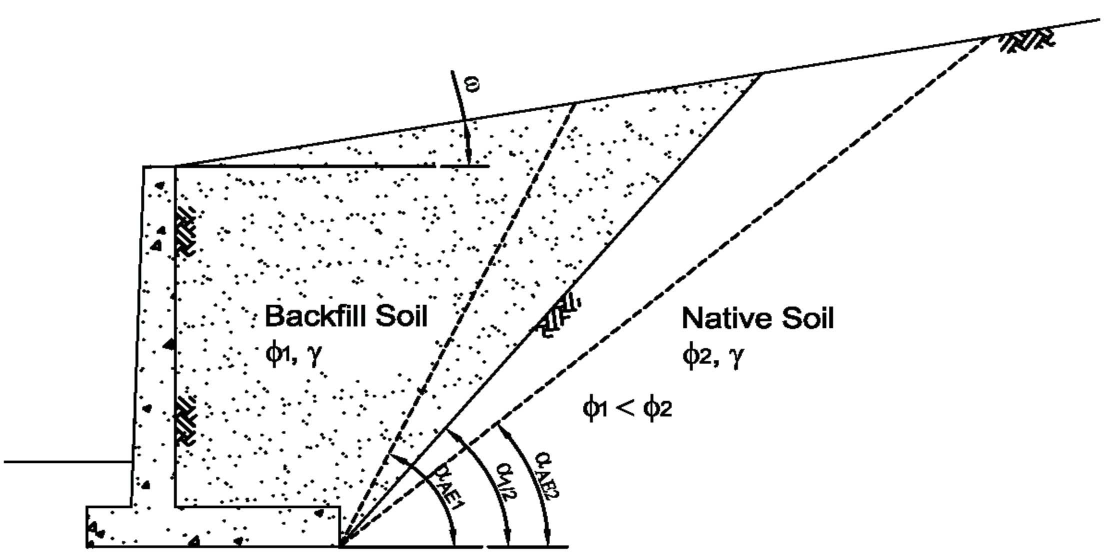


Figure X.7-2 Application of M-O Method for Non-Homogeneous Soil

In most cases, the cut into the native soil will be stable, in which case the active pressure corresponding to the cut angle will govern. Results of analyses conducted for the NCHRP 12-70 Project (NCHR, 2008) showed that the angle of the failure surface in the M-O equation changes with the peak ground surface acceleration and the angle of the backslope above the wall. The following table provides simplified guidance based on the NCHRP 12-70 evaluations.

Slope Angle Above Retaining Wall	Slope of Active Wedge during Seismic Loading
Flat	1.5H:1V
2H:1V	1.75:1V
3H:1V	2H:1V

In these guidelines, as long as the native or natural slope is flatter than 1.5H:1V for a flat condition above the wall, the M-O equation applies (assuming the backfill material is cohesionless, as is normally the case). If the native slope is steeper than the guideline and the native material differs from the backfill, the M-O equation should not be used. Similarly, for a 2H:1V or 3H:1V slope above the wall, the native slope must be flatter than 1.75H:1V or 2H:1V, respectively, to use the M-O equation.

For more complex cases involving non-uniform backslope profiles and backfill/cut slope soils, numerical procedures using the same principles of the M-O method may be used to compute seismic active earth pressures, such as the classical Culmann or wedge equilibrium methods, where a wedge-shaped failure surface is assumed as for the M-O method. The Caltrans computer program CT-FLEX (Shamsabadi, 2006), as described in Appendix B_X, was developed to handle these more complex cases using a wedge-equilibrium approach.

Mixed c-ϕ Soil Conditions

Most natural cohesionless soils have some fines content that contributes cohesion, particularly for short-term loading conditions. Similarly, cohesionless backfills are rarely fully saturated, and partial saturation provides for some apparent cohesion, even for most clean sands. In addition, it appears to be common practice in some states, to allow use of backfill soils with 30% or more fines content (possibly containing some clay fraction), particularly for MSE walls. Hence the likelihood in these cases of some cohesion is very high. The effects of cohesion, whether actual or apparent, are an important issue to be considered in practical design problems.

The M-O equation has been extended to c-ϕ soils by Prakash and Saran (1966), where solutions were obtained for cases including the effect of tension cracks and wall adhesion. Similar solutions have also been discussed by Richards and Shi (1994) and by Chen and Liu (1990). As part of the NCHRP 12-70 Project, a set of design curves were developed to show the effects of cohesion on seismic active earth pressure. These curves are included in Appendix B_X.

Results of the cohesion analysis show a significant reduction in the seismic active pressure for small values of cohesion. From a design perspective, this means that even a small amount of cohesion in the soil could reduce the demand required for retaining wall design.

From a design perspective, the uncertainties in the amount of cohesion or apparent cohesion make it difficult to explicitly incorporate the contributions of cohesion in many situations, particularly in cases where clean backfill materials are being used, regardless of the potential benefits of partial saturation. Realizing these uncertainties, the following guidelines are suggested.

- Where cohesive soils are being used for backfill or where native soils have a clear cohesive strength component, then the designer should give consideration to incorporating some effects of cohesion in the determination of the seismic coefficient. The best method of quantifying the amount of cohesion is to conduct triaxial strength tests on the soil that will be used for backfill.

- If the cohesion in the soil behind the wall results primarily from capillarity stresses, then the maximum apparent cohesion should be limited to values listed in Section X.5. Note that for high fines content soils, effective stress strength parameters are recommended for long-term static design in the AASHTO Specifications, whereas total stress strength parameters are recommended for short-term seismic loading. Consequently, the concept of using incremental earth pressures (i.e., seismic minus static) for seismic design is inappropriate for such soils.

Groundwater or Submerged Considerations

The groundwater within the active wedge or submerged conditions (e.g., as in the case of a retaining structure in a harbor or next to a lake or river) can influence the magnitude of the seismic active earth pressure. The time-averaged mean groundwater elevation is used when assessing groundwater effects.

If the soil within the wedge is fully saturated, then the total unit weight (γ_t) should usually be used in Equation X.7-1) to estimate the earth pressure, under the assumption that the soil and water move as a unit during seismic loading. This situation will apply for soils that are not free draining.

If the backfill material is a very open granular material, such as quarry spalls, it is possible that the water does not move with the soil during seismic loading. In this case the effective unit weight should be used in the pressure determination, and an additional force component due hydrodynamic effects should be added to the wall pressure. Various methods are available to estimate the hydrodynamic pressure (see Kramer, 1996). Generally, these methods involve a form of the Westergaard solution.

If soil is located below the water table, apparent cohesion from capillarity should not be used. If there is some percentage of fine-grained soil within the backfill, the cohesive component of strength is acceptable as long as the cohesion can be demonstrated through the conduct of appropriate undrained (total stress) strength tests.

X.7.2.1.2 Seismic Passive Earth Pressure Charts

Seismic passive earth pressures shall be estimated using procedures that account for the friction between the retaining wall and the soil, the nonlinear failure surface that develops in the soil during passive pressure loading, and for wall heights greater than 5 feet, the effects of inertial forces in the soil from the earthquake.

C.X.7.2.1.2

The seismic passive earth pressure becomes important for walls that develop resistance to sliding from the embedded portion of the wall. For these designs it is important to estimate passive pressures that are not overly conservative or unconservative for the seismic loading condition. This is particularly the case if displacement-based design methods are used, but it can also affect the efficiency of designs based on limit-equilibrium methods.

Method of Estimating Passive Pressures

If the depth of embedment of the retaining wall is less than 5 feet, the passive pressure can be estimated using static methods given in Section 3 of the AASHTO *LRFD Bridge Design Specifications.* For this depth of embedment the inertial effects from earthquake loading on the development of passive pressures will be small.

For greater depths of embedment, the inertial effects of ground shaking on the development of passive pressures should be considered. Shamsabadi et al. (2007) have developed a methodology for estimating the seismic passive pressures while accounting for wall friction and the nonlinear failure surface within the soil. Appendix B_X of this section provides charts based on this development for a wall friction of 2/3rds of the soil friction angle (ϕ) and a range of seismic coefficients ($k_h = k_{av}$), ϕ values, and soil cohesion (c).

The seismic coefficient used in the passive seismic earth pressure calculation is the same value as used for the seismic active earth pressure calculation. Wave scattering reductions are also appropriate to account for incoherency of ground motions in the soil if the depth of the passive zone exceeds 20 feet. For most wall designs the difference between the seismic coefficient behind the wall relative to seismic coefficient of the soil in front of the wall is too small to warrant use of different values.

The M-O equation for seismic passive earth pressure is not recommended for use in determining the seismic passive pressure, despites its apparent simplicity. For passive earth pressure determination, the M-O equation is based on the Coulomb method of determining passive earth pressure, and this method can overestimate the earth pressure in some cases. The M-O equation also does not account for the cohesion of the soil, and as shown in the charts in Appendix B_X, this contribution can be very significant.

Other Considerations

A key consideration during the determination of static and seismic passive pressures is the wall friction that occurs. Common practice is to assume that some wall friction will occur for static loading. The amount of interface friction for static loading is often assumed to range from 50 to 80% of the soil friction angle. Similar guidance is not available for seismic loading. In the absence of any specific guidance or research results for seismic loading, it is suggested that a wall interface friction equal to or greater than 2/3rds of the soil friction angle be used.

Another important consideration when using the seismic passive earth pressure is the amount of deformation required to mobilize this force. The deformation to mobilize the passive earth pressure during static loading is usually assumed to be large – say 2% to 6% of the embedded wall height. Similar guidance is not available for seismic loading and therefore the normal approach during design for seismic passive earth pressures is to assume that the displacement to mobilize the seismic passive earth pressure is the same as for static loading.

X.7.2.1.3.Generalized Limit Equilibrium Method

The generalized limit equilibrium (GLE) method shall be an acceptable method for estimating seismic active earth pressures for locations where one of the following conditions apply: (1) the M-O approach is not suitable because of combinations of steep backslope, seismic coefficient, and soil conditions; and (2) a fill is constrained by a steep cut slope.

The generalized limit equilibrium method shall use the seismic coefficient defined in Article X.4. A 50% reduction in the seismic coefficient shall be permitted as long as permanent displacement or rotation of the wall of 1 to 2 inches is acceptable to the Owner.

The external active force computed from the limit equilibrium method shall be used as the seismic earth pressure. The equivalent pressure representing both static and seismic loading combined shall be distributed uniformly over the wall when used for internal and external stability evaluations.

C.X.7.2.1.3

In some situations the M-O equation is not suitable due to the geometry of the backfill, the cohesive content of the backfill, the angle of the failure surface relative to the cut slope behind the wall, the magnitude of ground shaking, or some combination of these factors. In this situation, a generalized limit equilibrium method involving the use of a computer program for slope stability may be more suitable for determining the earth pressures required for retaining wall design.

Steps in the generalized limit equilibrium analysis are as follows:

- Setup the model geometry, groundwater profile, and design soil properties. The internal vertical face at the wall heel, or the plane where the earth pressure needs to be calculated, should be modeled as a free boundary.

- Choose an appropriate slope stability analysis method. Spencer's method generally yields good results because it satisfies the equilibrium of forces and moments.

- Choose an appropriate sliding surface search scheme. Circular, linear, multi-linear, or random surfaces can be examined in many commercial slope stability analysis programs.

- Apply the earth pressure as a boundary force on the face of the retained soil. The location of the force is assumed at one-third from base (1/3 H, where H is retained soil height) for static cases. For seismic cases the location of the force can be initially assumed at mid height (0.5 H) of the retained soil. However, different application points between 1/3 H and 0.6 H from the base can be examined to determine the maximum seismic earth pressure force. The angle of applied force depends on assumed friction angle between the wall and the fill soil (rigid gravity walls) or the fill friction angle (semi-gravity walls).

- Search for the load location and failure surface giving the maximum load for limiting equilibrium (capacity-to-demand ratio of 1.0).

- Verify design assumptions and material properties by examining the loads on individual slices in the output.

X.7.2.2 Wall Displacement Analysis

Where (1) the C/D ratio for global stability is less than 1.0 or (2) the amount of sliding allowed by the Owner can exceed 1 to 2 inches, thereby supporting a k_{max} reduction factor of greater than 50%, displacements using one of the procedures given in Article X.4 shall be allowed.

For critical structures identified by the Owner, the displacements estimated from the equations in Article X.4 shall be multiplied by 2 to obtain an 84% confidence level.

C.X.7.2.2

For most cases the permanent displacement of the rigid or semi-rigid retaining wall will be small (e.g., < 1 to 2 inches) if the methods described above are followed. However, in situations where the C/D ratio for global stability is less than 1.0 or where a larger reduction in the k_{max}, with accompanying larger permanent displacement, is desired. Methods given in Article X.4 provide a basis of making these displacement estimates.

It is important to recognize that the Newmark approach for estimating horizontal displacement in these Specifications is a simplified representation of potential displacement mechanisms associated with seismic loading. For example, the Newmark approach does not consider rotation and evaluate only translation of the retaining wall. Because of the high center of gravity and imposed force for the seismic loading case, a combination of permanent rotation and translation may occur.

This limitation must be recognized by the designer when using the Newmark approach. However, by limiting the horizontal displacement to a few inches, the rotation or tilt will typically be small – say less than a few degrees. This is likely to be the case as the potential for permanent rotation is minimized by using a reduced resistance factor for toe bearing capacity in the design requirements. If concerns exist about the amount of translation and associated rotation, it will be necessary to use numerical modeling methods to evaluate these effects.

Despite these limitations, the Newmark displacement estimate provides the designer with an understanding of the potential magnitude of displacements. If this magnitude is considered excessive by the Owner because of potential impacts to nearby roadways or utilities, the wall should be redesigned such that resulting displacements are smaller.

X.7.3 Design Requirements

Rigid and semi-rigid gravity retaining walls shall be designed to meet global stability, external stability, and internal stability requirements described in this section of the Specifications.

C.X.7.3

The stability evaluations described in this section involves collaboration between the geotechnical engineer and the structural engineer. Normally, the geotechnical engineer will identify ground motion parameters, earth pressures, base sliding coefficients, and ultimate and allowable bearing pressure values. The structural engineer will typically evaluate external and internal stability. The external stability evaluation involves a check on retaining wall overturning, sliding, and bearing; the internal stability check involves review of detailing of structural sizes and reinforcement to confirm that they meet shear and moment demands.

X.7.3.1 Global Stability

A global stability analysis shall be conducted to determine the capacity of the soil and retaining structure to resist seismic loads. The seismic coefficient (k_{max}) defined in Article X.4 shall be used to evaluate global stability under seismic loading. A 50% reduction in k_{max} shall be permitted for locations where small permanent displacement is tolerable, unless the Owner requires otherwise.

If the capacity-to-demand (C/D) ratio for global stability is 1.0 or higher, the global stability shall be considered acceptable. Otherwise, if the C/D ratio is less than 1.0,

- The potential for movement of the wall during seismic loading shall be considered. Procedures presented in Article X.4 shall be used to determine the potential for permanent ground movement.

- The wall shall be redesigned to meet capacity-to-demand requirements.

C.7.3.1

During seismic loading, one mode of failure for rigid and semi-rigid retaining walls is a slope failure that occurs with the failure surface extending below the base of the retaining wall foundation. This type of failure is normally evaluated using a slope stability computer program.

The geometry for the evaluation is selected to force the critical failure surface below the foundation. Soil properties for this analysis are selected in accordance with discussions in Article X.5 and in Section 10 of the AASHTO *LRFD Bridge Design Specifications.*

If global stability requirements are not satisfied, two options are available. The first is to revise the dimensions of the wall. However, it is often difficult to modify the geometry of the wall without significantly increasing the cost of construction. The alternative is to improve the soil at the wall location, through the use of stone columns, soil mixing, or other methods.

Rigid gravity and semi-gravity retaining walls should not be located above liquefiable soils, unless special studies are conducted to show that performance will be acceptable. If liquefiable conditions exist, procedures discussed in Section Y should be followed when evaluating stability. For most liquefiable sites, the ground should be improved, a pile-support system should be used, or an alternate wall system selected.

X.7.3.2 External Stability

Analyses shall be conducted to show that the retaining wall meets sliding, overturning, and bearing capacity stability based on demands calculated with reduced k_{max}, where appropriate. The ratio of capacity-to-demand (C/D ratio) shall be greater than 1.0 when the following resistance factors are applied to capacity:

Sliding: 1.0

C.X.7.3.2

Resistance factors are lower for bearing than for sliding and overturning to reduce the tendency of the rigid or semi-rigid retaining wall to rotate during seismic loading. This philosophy of requiring bearing to have a lower resistance factor is consistent with principles used when sizing rigid and semi-rigid retaining walls for gravity loads, where in terms of working stress design, factors of safety for bearing have traditionally been

Overturning: 1.0

Toe Bearing: 0.67

Retaining walls that do not meet the overturning and toe bearing C/D ratios identified above shall be redesigned to meet these requirements, as described in the flow chart in Appendix A_X. If the C/D ratio for sliding is not met but the overturning and toe bearing are satisfied, either the dimensions of the footing shall be revised to meet the required C/D ratio or the amount of permanent displacement shall be estimated in accordance with Article X.7.2.2 to determine if permanent displacements are within acceptable performance limits.

Earth pressures defined previously shall be used in the external stability assessment.

higher than those for sliding. As noted in the following discussions, the amount of liftoff of the foundation during overturning is limited to 50%, and this provision limits the rotation potential of the structure, thereby allowing using of a resistance factor of 1.0 for the overturning check.

Sliding Stability

Sliding stability is evaluated using the interface friction at the base of the foundation. The magnitude of the interface friction is determined from the method of construction (e.g., precast versus cast-in-place) and the frictional characteristics of the granular base material. The interface resistance is not modified for dynamic loading effects.

The following relationships can be used to estimate sliding resistance (R_{ult}):

Cohesionless Soil

$$R_{ult} = P \tan \delta \qquad \text{C.X.7.3-1}$$

where P is the normal load and δ is the interface friction angle.

Cohesive Soil

$$R_{ult} = A_e S_u \qquad \text{C.X.7.3-2}$$

where A_e is the area in contact and S_u is the undrained strength of the soil.

The resisting contribution of the passive resistance at the toe of the foundation should be included in the sliding stability assessment. The inertia of the structure and the soil mass above the heel of a semi-gravity retaining wall are included in the stability assessment by multiplying the weight of the soil and the wall by the seismic coefficient. A load factor of 1.0 is used for this evaluation.

The seismic coefficient used for this evaluation is k_{max} after applying the 50% reduction factor. The contribution of weight of the soil above the heel of the retaining wall is included in the determination of sliding resistance.

Overturning Stability

Overturning stability is evaluated by summing moments about the toe of the retaining wall. A maximum liftoff of 50% of the footing width is permitted during the design event.

The inertia and weight of the soil above the heel of the retaining wall are included in this evaluation. The seismic coefficient used for this evaluation is k_{max} after applying the 50% reduction factor.

Bearing Stability

Bearing stability is checked by comparing the peak bearing pressure at the toe of the foundation to the ultimate bearing capacity of the soil, assuming an equivalent rectangular bearing pressure distribution. No reduction is made for eccentric loading. A maximum lift-off of 50% of the footing is permitted during the design event. The ultimate bearing capacity is based on the effective width of the footing (i.e., after liftoff) and calculated using total stress soil properties.

X.7.3.3 Internal Stability

The retaining wall shall be evaluated to show that wall moment and shear forces are acceptable as defined in Sections 5 and 6 of these Specifications and using load and resistance factors provided in Article X.6.

Lateral earth pressure loads acting on the wall shall be the same earth pressures used for external stability evaluations. A uniform pressure distribution shall be assumed. The inertial forces from the soil mass above the heel shall be assumed to be transferred to the heel of the wall through shear of the soil and accounted for in the external stability evaluation. The inertial response of the wall structure shall be included in the assessment of moment and

C.X.7.3.3

The Owner may decide that it is acceptable for a retaining wall to be damaged during a design seismic event. The performance expectation will depend on the function of the wall and the cost of repair or replacement. Once the performance expectation is established, normal procedures can be used for detailing the wall.

Moment and shear forces are determined by adding the total seismic earth pressure force to the inertial response of the wall section, assuming both are in phase. This approach is conservative in the sense that the peak inertial response of the wall mass may not occur at the same time as the peak seismic active pressure. However, to avoid shear forces. The inertial force from the wall mass and the seismic earth pressure shall be assumed to develop concurrently.

Structural performance under these loading conditions shall meet the performance expectations of the Owner.

complex interaction analyses, as well as the potential for unconservative assumptions by assuming that the inertial force and the seismic active pressure are not coincident, the two are assumed to occur at the same time.

The inertial force associated with the soil mass on the wall heel behind the retaining wall is not added to the active seismic earth pressure when detailing the retaining wall. The basis for excluding this inertial force is that movement of this soil mass is assumed to be in phase with the structural wall system with the inertial load transferred through the heel of the wall. Based on typical wave lengths associated with seismic loading, this is considered a reasonable assumption.

When estimating the inertial force on the wall, reduced values of k_{max} are used in the determination of earth pressures. This includes applying a 50% reduction to k_{max} if the wall will displace by several inches. Permanent slip serves as fuse in terms of limiting the acceleration and therefore inertial force developed in the soil mass.

X.8 NONGRAVITY CANTILEVER WALLS

X.8.1 General

Nongravity cantilever walls shall be designed for seismic loading except where conditions given in Article X.4.1 are satisfied or where allowed otherwise by the Owner.

Before conducting the seismic evaluation, nongravity cantilever walls shall be designed to meet all gravity and live load requirements in accordance with the provisions in Sections 3, 5, 6, and 11 of the AASHTO *LRFD Bridge Design Specifications.*

C.X.8.1

The starting point for the seismic design of a nongravity cantilever wall is an acceptable static design, meeting the requirements of the current AASHTO *LRFD Bridge Design Specifications*. Once the static design has been completed, the geometry and structural properties are checked for seismic loading.

As part of the seismic check, the performance expectations for the wall during the design seismic event should be determined through discussions with the Owner. Some of the factors that should be considered in determining performance expectations are summarized in Appendix

A_X.

During seismic loading, the nongravity cantilever wall develops resistance to load through the passive resistance of the soil below the excavation depth. The stiffness of the structural wall section above the excavation depth must be sufficient to transfer seismic forces from the soil behind the wall, through the structural section, to the soil below. The seismic evaluation of the nongravity gravity cantilever wall requires, therefore, determination of the demand on the wall from the seismic active earth pressure and the capacity of the soil from the seismic passive soil resistance.

X.8.2 Method of Analysis

The seismic analysis of the nongravity cantilever retaining wall shall demonstrate that the cantilever wall will maintain overall stability and withstand the seismic earth pressures induced by the design earthquake without excessive structural moments and shear on the cantilever wall section. Limit equilibrium methods or numerical displacement analyses shall be used to confirm acceptable wall performance.

C.X.8.2

The seismic analysis of the nongravity cantilever retaining wall involves estimating the seismic active earth pressure resulting from the inertial response of the soil. Mononobe-Okabe or generalized limit equilibrium methods are used to estimate this pressure. The soil resistance below the excavation level is estimated from the passive resistance of the soil in front of the wall.

Standard Nongravity Cantilever Walls

Typical nongravity cantilever retaining walls are continuous above the excavation level; however, below the excavation level the walls can be continuous, as in the case of sheet pile or secant pile walls, or they can be discrete structural elements as in the case of a soldier pile wall with lagging. In this latter class of walls, the reaction must consider the 3-dimensional effects of the structural member when estimating soil reaction, rather than the 2-dimensional conditions for continuous sheet pile or secant pile walls.

Pile-Supported Nongravity Cantilever Walls

At sites characterized by soft soil conditions, it may not be possible to meet seismic loading demands using semi-gravity retaining walls, sheet pile walls, or nongravity cantilever walls. For these locations piles are often used to support the retaining structure. The pile-supported wall typically consists of a series of vertical piles that support a semi-gravity wall.

For design purposes the pile-supported wall is considered with the nongravity cantilever wall because of the similarities in the load and displacement mechanisms associated with seismic loading. In most situations the pile-supported wall, moves enough during seismic loading to develop seismic active earth pressures; however, the amount of movement may not be 1 to 2 inches necessary to allow reduction in the seismic coefficient by 50%, unless analyses demonstrate that permanent wall movements will occur without damaging the wall components.

Beam-column analyses involving p-y modeling of the soil-pile system will usually be required to make this assessment. The design of the pile-supported retaining wall must consider the group response of the pile system in terms of moments, shears, and displacements – similar to a bridge foundation undergoing seismic loading.

X.8.2.1 Seismic Active Earth Pressure

The total seismic earth pressure behind the wall shall be estimated using either the wedge equilibrium method or the generalized limit equilibrium method, as described in Article X.7.2.1. For locations where the soil behind the wall is relatively homogeneous and cohesionless, the M-O equation shall be an acceptable alternative to the wedge or generalized limit equilibrium methods.

The peak ground surface acceleration identified in Article X.4 shall be used to estimate the seismic active earth pressure. Where 1 to 2 inches of permanent movement of the wall at the excavation level or where the soil behind the wall has a fines content of greater than 15%, a

C.X.8.2.1

The displacement of most cantilever walls during seismic loading is such that active earth pressures will be developed. However, the stiffness of the nongravity wall limits the amount of lateral displacement that will occur during seismic loading for a well-designed wall.

Typically, nongravity cantilever walls are less than 20 feet in height. In this height range wave incoherency effects are usually minimal, and therefore, the peak ground acceleration is appropriate for design. However the peak ground acceleration can be reduced by 50% in many situations:

- Nongravity cantilever walls are often used in native soil conditions, where reduction of the seismic coefficient shall be permissible unless not allowed by the Owner. The resulting active seismic earth pressure shall be distributed as a uniform pressure against the cantilever wall above the excavation level in a similar manner to that used for gravity or semi-gravity walls. However, due to uncertainties in the seismic pressure distributions below the excavation level, in the case of limit equilibrium analyses, both active and passive pressure distributions for seismic loading shall be assumed to be distributed in a similar manner to static pressures.

the soil in front of the wall is excavated downward as the wall is constructed. As discussed in Article X.5, most native soils will contain some fines content. The consequence of the fines content is to reduce the seismic active pressure. As long as the fines content is greater than 15%, a further 50% reduction in the seismic active earth pressure could occur. Either the equation and charts in Appendix B_X or the generalized limit equilibrium method should be used to justify this reduction, as discussed later in this Article.

- The movements associated with a 50% reduction in the seismic active earth pressure for permanent soil movement is estimated to be 1 to 2 inches at the excavation level. This amount of movement will often be acceptable, particularly for freestanding, nongravity cantilever walls after a design seismic event. However, before adopting a 50% reduction, the performance objectives of the wall need to be established with the Owner, and structural analysis must be conducted to confirm that the 1 to 2 inches of permanent movement do not result in excessive structural damage or collapse of the wall.

Note that a 50% reduction for cohesive soil effects on seismic active earth pressure should not be combined with a 50% reduction accounting for permanent soil movement. The maximum reduction in seismic active earth pressure should be 50% unless special studies are conducted to support use of a larger reduction. Special studies could include used of numerical modeling method, if agreed by the Owner.

As noted above, an alternate approach for determining the seismic active earth pressure involves use of the generalized limit equilibrium method. Use of this method for the design of semi-gravity walls is discussed in Article X.7.2.1.3. If used for the design of a nongravity cantilever wall, the geometry of the slope stability model extends from the ground surface to the bottom or toe of the sheet pile or secant pile wall. For soldier pile walls the analysis extends to the excavation level. The seismic active pressure is determined by applying an external load to the midpoint of the wall in the slope stability model. The size of the external load is modified until the resulting stability analysis gives a capacity-to-demand ratio of 1.0 (i.e., FS = 1.0). The resulting load is distributed as a seismic active pressure.

Articles X.7.2.1.1 and X.7.2.1.3 provide additional considerations relative to the determination of the seismic active earth pressure. If the cantilever height of the wall is greater than 20 feet, a reduction in the peak ground acceleration at the ground surface (i.e., $k_{av} = \alpha\ k_{max}$) to account for wave scattering effects can be considered, following the discussions in Article X.4.

X.8.2.2 Seismic Passive Earth Pressure

For the limit equilibrium approach the peak seismic passive resistance below the excavation level shall be determined by computing the seismic passive pressure.

The method used to compute the seismic passive pressure shall consider wall interface friction, the nonlinear failure surface that develops during passive pressure loading, and the inertial response of the soil within the passive pressure wedge for depths greater than 5 feet. Cohesion and frictional properties of the soil shall be included in the determination.

The seismic passive pressure shall be applied as a triangular pressure distribution similar to that for static loading. The amount of displacement to mobilize the passive pressure shall also be considered in the analyses.

C.X.8.2.2

The peak seismic passive pressure is based on the time-averaged mean groundwater elevation, the full depth of the below-ground structural element, and the strength of the soil for undrained loading. The upper 2 feet of soil are not neglected as typically done for static analyses. The wall friction in the passive pressure estimate is taken as 2/3rds times the of the soil strength parameters from a total stress analysis. The effects of live loads are usually neglected from this computation.

Reductions in the seismic passive earth pressure may be warranted to limit the amount of deformations required to mobilize the seismic passive earth pressure, if wall design will use a limit equilibrium method of analysis. In the absence of specific guidance for seismic loading, a resistance factor of

0.67 should be applied to the seismic passive pressure during the seismic check to limit displacement required to mobilize the passive earth pressure.

If the nongravity cantilever wall uses soldier piles to develop reaction to active pressures, adjustments must be made in the passive earth pressure determination to account for the three-dimensional effects below the excavation level as soil reactions are developed. In the absence of specific seismic studies dealing with this issue, it is suggested that methods used for static loading be adopted. One such method, documented in the Caltrans *Shoring Manual*, suggests that soldier piles located closer than 3 pile diameters be treated as a continuous wall. For soldier piles spaced at greater distances, the approach in the Caltrans *Shoring Manual* depends of the type of soil:

- For cohesive soils the effective pile width that accounts for arching ranges from 1 pile diameter for very soft soil to 2 diameters for stiff soils.

- For cohesionless soils, the effective width is defined as 0.08*ϕ*B up to 3 pile diameters. In this relationship ϕ is the soil friction angle and B is the soldier pile width.

During seismic loading, the inertial response of the soil within the passive pressure failure wedge will decrease the resisting capacity of the soil during a portion of each loading cycle. Figures provided in Appendix B_X can be used to estimate the passive soil resistance for different friction values and normalized values of cohesion. A preferred methodology for computing seismic earth pressures with consideration of wall friction, nonlinear soil failure surface, and inertial effects involves use of the procedures documented by Shamsabadi et al. (2007).

X.8.2.3 Wall Displacement Analysis

Numerical displacement analyses shall be permitted as an alternative to the limit equilibrium method discussed in the previous Article. The displacement-based analyses shall show that moments, shear forces, and structural displacements resulting from the peak ground surface accelerations are within acceptable levels. These analyses shall be conducted using a model of the wall system that includes the structural stiffness of the wall section, as well as the load-displacement response of the soil above and below the excavation level.

C.X.8.2.3

Numerical displacement methods offer a more accurate and preferred method of determining the response of nongravity cantilever walls during seismic loading. Either of two numerical approaches can be used. One involves a simple beam-column approach; the second involves the use of a 2-dimensional computer model. Both approaches need to appropriately represent the load-displacement method of the soil and the structural members during loading. For soils this includes nonlinear stress-strain effects; for structural members consideration must be given to ductility of the structure, including the use of cracked versus un-cracked section properties if concrete structures are being used.

Beam-Column Approach

The pseudo-static seismic response of a nongravity cantilever wall can be determined by representing the wall in a beam-column model with the soil characterized by p-y springs. This approach is available within commercially available computer software such as the Ensoft program PY-Wall (Ensoft, 2004) or can be adapted in programs such as LPILE, COM 624, or BMCOL. The total seismic active pressure above the excavation level is used for wall loading. Procedures given in Article X.8.2.1 should be used to make this estimate.

For this approach the p-y curves below the excavation level need to be specified. For discrete structural elements (i.e., soldier piles), conventional p-y curves for piles can be used. For continuous walls or walls with pile elements at closer than 3 diameter spacing, p- and y-modifiers have been developed as part of the NCHRP 12-70 Project to represent a continuous (sheet pile or secant pile) retaining wall. The procedure involves

- Developing conventional isolated pile

p-y curves using a 4-foot diameter pile following API (1993) procedures for sands or clays.

- Normalizing the isolated p-y curves by dividing the p values by 4 feet.
- Applying the following p- and y-multipliers, depending on the type of soil, in a conventional beam-column analysis.

Soil Type	**p-multiplier**	**y-multiplier**
sand	0.5	4.0
clay	1.0	4.0

Supporting information for the development and use of the p-y approach identified above is presented in Volume 1 of NCHRP 12-70 Report (NCHRP, 2008). The earth pressure used as the load in the beam column analysis is determined from one of the limit equilibrium methods, including M-O with or without cohesion or the generalized limit equilibrium procedure, as discussed in Article X.8.2.1. The benefits of the p-y approach are that it enforces compatibility of deflections, earth pressure, and flexibility of the wall system. The method is in contrast to the limit equilibrium method in which the effects of the wall flexibilities are ignored. This is very important for the seismic design and performance of the wall during seismic event. The deformation and rotation of the wall can easily be captured using the p-y approach.

Finite Difference or Finite Element Modeling

Pseudo-static or dynamic finite element or finite difference procedures in computer programs such as FLAC (Itasca, 2007) and PLAXIS (Plaxis BV, 2007) can also be used to evaluate the seismic response of nongravity cantilever walls during seismic loading. For 2-dimensional models it may be necessary to “smear” the stiffness of the structural section below the excavation level to adjust the model to an equivalent 2–dimensional representation if the below-grade portion of the wall is formed from discrete piles (e.g., soldier piles).

The finite difference or finite element approach to evaluating wall response will involve a number of important assumptions; therefore, this approach should be discussed with and agreed to by the Owner before being adopted. As part of the discussions, the possible limitations and the assumptions being made for the model should be reviewed.

X.8.3 Design Requirements

The nongravity cantilever wall shall be designed to meet global and internal stability requirement set forth in this Article. Earth pressures discussed in the preceding Article shall be used as input for these analyses.

C.X.8.3

Two types of stability checks are conducted for the nongravity cantilever wall, global stability and internal stability.

The global stability check for seismic loading involves a general slope failure analysis that extends below the base of the wall. Typically the depth of the wall is 1.5 to 2 times the wall height above the excavation level. For these depths global stability is not normally a concern except where soft layers are present below the toe of the wall.

Internal stability for a nongravity cantilever wall refers to the moments and shear forces developed in the wall from the seismic loads. In contrast to rigid and semi-rigid gravity walls, external stability (i.e., sliding, overturning, and bearing) is not a design consideration for this wall type. By sizing the wall to meet earth pressures, the equilibrium requirements for external stability are also satisfied.

X.8.3.1 Global Stability

The global stability for the nongravity cantilever wall shall be established by conducting limit equilibrium slope stability analyses. Results of these analyses shall demonstrate that the capacity-to-demand ratio is greater than 1.0 under the peak seismic load.

C.X.8.3.1

The global stability analysis is performed with a slope stability program. The failure surfaces used in the analysis should normally extend below the depth of the structure member.

Checks can also be performed for failures below the excavation level but through the

If the C/D ratio is less than 1.0 for global stability, either a displacement analyses shall be conducted to demonstrate that movements are tolerable, or the retaining wall shall be redesigned to meet minimum global stability requirements..

structure. These analyses should include the contributions of the structural section to slope stability. If the structural contribution to capacity is being accounted for in the stability assessment, the moments and shears developed by the structural section needs to be checked to confirm that allowable structural limits are not exceeded.

As long as the capacity-to-demand ratio is greater than 1.0, the nongravity cantilever wall section is stable. If the capacity-to-demand ratio is less than 1.0, the Newmark method can be used to determine permanent displacements. An acceptable alternative is to conduct 2-dimensional numerical analyses using finite element or finite difference modeling methods to estimate permanent displacements as noted in Article X.4.

If permanent displacements are predicted to occur in the analysis, the computed displacement needs to be reviewed with the Owner to confirm that it is acceptable. If the displacement involves the structural section, the consequences of displacements should be evaluated. Displacement-based methods such as the beam column method identified previously should be used in making this assessment.

X.8.3.2 Internal Stability

Moments and shears within the structural section during seismic loading shall be within acceptable limits prescribed in Sections 5 and 6 of the AASHTO *LRFD Bridge Design Specifications* and using load and resistance factors provided in Article X.6..

C.X.8.3.2

Either the simplified earth pressure distribution method given in Section 3 of the AASHTO *LRFD Bridge Design Specifications* or the numerical method described in Article X.8.2.3 of this Article should be used to determine shear and moments for internal stability design. The seismic load for this evaluation is the total earth pressure load distributed uniformly behind the wall.

With the availability of computer programs for conducting beam-column analyses, use of the beam column approach offers a relatively inexpensive and simplified method of providing a realistic assessment of structural response to the seismic loads, and is therefore encouraged for general use.

If shear or bending forces exceed allowable levels for seismic loading (Extreme Event I) based on the seismic earth pressure computed in Article X.8.2.1, a larger section will need to be selected for design and re-analysis.

X.9 ANCHORED WALLS

X.9.1 General

Anchored walls shall be designed for seismic loading except where conditions given in Article X.4.1 are satisfied or where allowed otherwise by the Owner.

Before conducting the seismic evaluation, the anchored wall shall be designed to meet all gravity and live load requirements in accordance with the provisions in Sections 3, 5, 6, and 11 of the AASHTO LRFD Bridge Design Specifications.

C.X.9.1

The seismic design of an anchored wall involves many of the same considerations as the nongravity cantilever wall. However, the addition of one or more anchors to the wall introduces some important differences in the seismic design check as identified in this Article of the Specifications.

The performance expectations for the anchored wall during the design seismic event should be determined through discussions with the Owner. Some of the factors that should be considered in determining performance expectations are summarized in Appendix A_X.

As with the nongravity cantilever wall, the starting point for the seismic design of the anchored wall is an acceptable static design, meeting the requirements of the current AASHTO *LRFD Bridge Design Specifications*. Once the static design has been completed, the geometry is checked for seismic response.

During seismic loading, the anchored wall develops resistance to load primarily through the reaction of anchors that have been installed and tensioned to meet static design requirements. Passive resistance is mobilized by the portion of the retaining wall (e.g., soldier pile, sheet pile, or secant pile) that extends below the excavation level. The stiffness of the structural wall section above the excavation level must be sufficient to transfer the seismic earth pressures developed on the face of the wall to the anchors. The seismic evaluation of the anchored wall requires, therefore, determination of the demand on the wall from the seismic earth pressures and capacity from the anchors and from the passive soil resistance of the wall below the excavation level.

X.9.2 Methods of Analysis

The seismic analysis of the anchored retaining wall shall demonstrate that the anchored wall can maintain overall stability and withstand the seismic earth pressures induced by the design earthquake without exceeding the capacity of the anchors or the structural wall section supporting the soil. Limit equilibrium methods or numerical displacement analyses shall be used to confirm acceptable wall performance.

C.X.9.2

The earth pressures above the excavation level results from the inertial response of the soil mass behind the wall. In contrast to a nongravity cantilever wall, the soil mass includes anchors that have been tensioned to minimize wall deflections under static earth pressures. During seismic loading, the bars or strands making up the unbonded length of the anchor are able to stretch under the imposed incremental seismic loads. In most cases the amount of elastic elongation in the strand or bar under the incremental seismic load is sufficient to develop seismic active earth pressures.

The passive pressure for the embedded portion of the soldier pile or sheet pile wall also plays a part in the stability assessment, as it helps provide stability for the portion of the wall below the lowest anchor. This passive pressure is subject to seismic-induced inertial forces that will reduce the passive resistance relative to the static capacity of the pile or wall section. Most often the embedded portion of the pile involves discrete structural members spaced at 8 to 10 feet; however, the embedded portion could also involve a continuous wall, in the case of a sheet pile or secant pile wall.

X.9.2.1 Seismic Active Earth Pressure and Anchor Loads

If limit equilibrium or beam-column displacement methods are used to estimate the seismic response of the anchored wall, seismic earth pressures above the excavation level shall be determined and used to evaluate seismic anchor loads.

For most locations the seismic active earth pressure shall be estimated using wedge equilibrium or the generalized limit

C.X.9.2.1

The method for determining the seismic active earth pressure involves a number of considerations. These include the appropriate seismic coefficient, the method of analysis, and the location of the anchors.

Seismic Coefficient

The soil within the anchor zone is not completely free to respond to earthquake equilibrium method. For locations where the soil behind the anchored wall face is relatively homogeneous and cohesionless, the M-O method shall be an acceptable alternative to the wedge or generalized limit equilibrium method.

Anchors shall be located behind the limit equilibrium failure surface for seismic loading. The location of the failure surface for seismic loading shall be established using methods that account for the seismic coefficient and the soil properties (i.e., c and ϕ) within the anchored zone.

loading, as would be the case for another wall type. Anchors that extend from the face of the wall to some distance behind the wall limit the amount of deformation that will develop under the incremental seismic load.

As grouted anchors are not ductile elements and as the force contributions from the anchors limit the deformations that the wall will undergo to small amounts, the peak seismic coefficient identified in Article X.4 (k_{max}) should be used in determining the earth pressures. Reductions for wave scattering can be included (i.e., $k_{av} = \alpha\ k_{max}$); however, reductions of the seismic coefficient related to permanent wall displacement should not be used (i.e., not 50% of k_{max} as allowed within the current AASHTO *LRFD Bridge Design Specifications*), unless special studies are performed to confirm that the several inches of permanent wall movement will occur.

As noted previously, the 50% reduction used in AASHTO is based on several inches of permanent displacement occurring. This condition will not occur for most anchors designs. However, if displacement analyses are performed to show that several inches of displacement can occur without overstressing either the anchors or the vertical wall elements, and the permanent movement is acceptable to the Owner, then a reduction can be considered.

Soil Property Characterization

Total stress (undrained) soil strength parameters should be used to model the soil profile. This approach differs from that used for static design of the anchored wall, where effective stress soil parameters are used to represent the soil strength. For soils containing a significant percentage of fine-grained soils, the effects of the fine-grained soil can be significant. Additional discussion of strength property selection is given in Article C.X.7.2.1.

Earth Pressure Computation

Seismic active earth pressures can be

estimated using the M-O equation, wedge methods, or generalized limit equilibrium methods. The decision on the appropriate method should be made on the basis of the soil types and layering at the site, the geometry above the wall, and any external loads within the anchored zone.

The M-O equation is applicable only for homogeneous, cohesionless soil deposits. For locations where such conditions occur, the M-O equation can be used. However, for most locations where an anchored wall will be constructed, native soils that are both layered and contain some fines content will occur. The M-O equation is not appropriate for use in this situation.

For sites where there is cohesive content in the soil, the wedge equilibrium method or figures given in Appendix B_X can be used. If there is sufficient layering to make determination of average total stress soil parameters (c and ϕ) difficult to accomplish with much certainty, the generalized limit equilibrium method can be used for estimating seismic active earth pressures.

When using the generalized limit equilibrium method, the seismic coefficient determined from Article X.4 is used to develop the inertial force. The model for generalized limit equilibrium is developed such that the critical failure surface is tangent to the excavation level. An external force P_{AE} is applied on the face of the wall or in the anchor zone, depending on the capability of the computer software package being used. This force is applied to determine the total reaction that must be resisted by the anchors to achieve stability for the assigned seismic coefficient. The force is assumed to act horizontally as wall friction may not be mobilized behind the wall. By computing an equivalent P_{AE}, the use of the AASHTO static design formula to compute the equivalent pressure distribution and corresponding anchor loads may then be used.

Unlike the static condition where the

earth pressure behind an anchored wall is affected by various construction-related parameters, the seismic earth pressure is controlled largely by limit equilibrium considerations. However, the distribution of earth pressures under seismic loading is not as well known for anchored walls. In lieu of available data, it should be assumed that the seismic pressures have a similar distribution to that for static design.

Anchor Location

Anchors should be located behind the failure surface associated with the calculation of P_{AE}. The location of this failure surface can be determined using either the wedge equilibrium or the generalized limit equilibrium (slope stability) method. [Note that this failure surface will likely be flatter than the requirements for anchor location under static loading given in Section 11 of the current AASHTO *LRFD Bridge Design Specifications*.]. When using the wedge equilibrium or the generalized limit equilibrium method, P_{AE} and its associated critical surface should be determined without the anchor forces.

Once the location of the anchor bond zone is defined, an external stability check should be conducted with the anchor forces included, using the anchor test load to define assume ultimate anchor capacities. This check is performed to confirm that the C/D ratio (i.e., factor of safety) is greater than 1.0. Under this loading condition, the critical surface will flatten and could pass through or behind some anchors. However, as long as the C/D ratio is greater than 1.0, the design is satisfactory.

If C/D ratio is less than 1.0, either the unbonded length of the anchor must be increased or the length of the grouted zone must be lengthened. The design check would then be repeated.

X.9.2.2 Seismic Passive Earth Pressure

The seismic passive pressure in front of the below-grade portion of the wall shall be estimated following the procedures identified in Article X.8.2.2.

C.X.9.2.2

The embedment of the soldier pile or sheet pile wall is usually relatively shallow; therefore, the passive pressure in front of the below-grade portion of the retaining wall is usually relatively low. This reaction is, however, important to the seismic design of the wall.

If the depth of embedment is less than 5 feet, static methods can be used to estimate the passive resistance; otherwise the reductions in the passive pressure from the inertial effects of the seismic load should be accounted for using procedures described by Shamsabadi et al. (2007).

As discussed in Article X.8.2.2, the passive pressure should be determined using methods that account for wall friction and the nonlinear failure surface within the soil. Many soils will behave in an undrained state during seismic loading; therefore, the total stress undrained strength parameters with both c and ϕ should be used to compute the passive pressure.

X.9.2.3 Wall Displacement Analysis

Displacement analyses shall be permitted using numerical models subject to the approval of the Owner.

C.X.9.2.3

Numerical methods can be used in place of the limit equilibrium method described, if appropriate. Procedures given in Article X.8.2.3 can be followed when conducting the numerical analyses, with the difference that anchors are added to the model.

Computer programs such as PY-Wall, BMCOL, FLAC and PLAXIS are capable of handling the anchors in the model. Anchors in the BMCOL and PY-Wall models are represented as elastic or elastic-plastic springs. FLAC and PLAXIS model the anchors as cable elements; i.e., structural components that develop resistance in extension but not compression.

X.9.3 Design Requirements

The anchored wall shall be designed to meet global and internal stability requirements set forth in this Article. Earth pressures discussed in this Article of the Specifications shall be used as input for these analyses.

C.X.9.3

Similar to the nongravity cantilever wall, two types of stability checks are performed for the anchored wall: global stability and internal stability

The global stability check is performed to confirm that a slope stability failure does not occur below the anchored wall; external stability is checked to confirm the anchors will have sufficient reserve capacity to meet seismic load demands; and internal stability is checked to confirm that moments and shear forces within the structural members, including the anchor strand or bar tensile loads and the head connection, are within acceptable levels for the seismic load.

X.9.3.1 Global Stability

The global stability analyses shall establish that the capacity-to-demand ratio is greater than 1.0 (i.e., FS > 1.0) for potential failure surfaces below the retaining wall under the design seismic event.

If the capacity-to-demand ratio is less than 1.0, either (1) the wall shall be redesigned or (2) the amount of deformation associated with the low capacity-to-demand ratio shall be established. For a displacement-based approach the magnitude of movement shall be reviewed relative to the Owner's performance requirements.

C.X.9.3.1

The global stability analysis is the same as that discussed in Article C.X.8.3.1. The seismic coefficient can be reduced for wave scattering but is not further reduced for permanent displacement, unless special studies are performed to support the reduction. For well-designed walls, the amount of deflection during the seismic load will usually not be sufficient to support further reduction in the seismic coefficient

X.9.3.2 Internal Stability

Moments and shears within the structural section and tensile capacity of the anchor strand or bar shall be within acceptable limits prescribed in Sections 5, 6, and 11 of the AASHTO *LRFD Bridge Design Specifications* and using load and resistance factors provided in Article X.6.

C.X.9.3.2

Either the simplified earth pressure distribution method given in Section 3 of the AASHTO *LRFD Bridge Design Specifications* or the numerical displacement method described in Article X.8.2.3 of this section of the Specifications can be used to determine shear and moments for internal stability design

- For the simplified approach the wall force determined by the M-O equation, the wedge equilibrium method, or the generalized limit equilibrium method is distributed in the same manner as shown in Section 3 of the AASHTO *LRFD Bridge Design Specifications* for static loading.

- If a numerical approach is used to determine moments and shear forces in the structural members, the beam-column approach identified for nongravity cantilever walls can be followed, but with the addition of springs to account for the anchors.

If shear or bending forces exceed allowable levels for seismic loading (Extreme Event I) based on the seismic earth pressure, a larger structural section should be selected to meet stability requirements. Checks should be performed for lagging used between soldier piles following the recommendations for static design in FHWA (1998).

X.10 MECHANICALLY STABILIZED EARTH (MSE) WALLS

X.10.1 General

MSE walls shall be designed for seismic loading except where conditions given in Article X.4.1 are satisfied or where allowed otherwise by the Owner.

Before conducting the seismic evaluation, the MSE wall shall be designed to meet all gravity and live load requirements in accordance with the provisions in Section 11 of the AASHTO *LRFD Bridge Design Specifications*.

C.X.10.1

The starting point for the seismic design of an MSE wall is an acceptable static design, meeting the requirements of the current AASHTO *LRFD Bridge Design Specifications*. Once the static design has been completed, the geometry is checked for seismic response.

As part of the seismic check, the performance expectations for the wall during the design seismic event should be determined through discussions with the Owner. Some of the factors that should be considered in determining performance expectations are summarized in Appendix A_X.

The approach taken for the seismic design of an MSE wall is similar to that described for rigid and semi-rigid gravity walls; i.e., the seismic design must demonstrate that the MSE wall performs adequately during the design earthquake for three conditions:

- Global stability
- External stability (i.e., sliding, overturning, and bearing)
- Internal stability

X.10.2 Methods of Analysis

Analyses shall be conducted to show that the MSE wall can resist forces due to seismic earth pressures and inertial forces without excessive sliding or rotation of the structure or structural failure of the reinforcing strips or wall facing elements. Either limit equilibrium or displacement methods shall be used to establish that performance meets design expectations during the seismic event.

C.X.10.2

The current AASHTO seismic design guidelines for MSE walls are largely based on pseudo-static stability methods utilizing the Mononabe-Okabe (M-O) seismic active earth pressure equation. The approach separates dynamic earth pressure components, which are added to static components to evaluate external sliding stability or to determine reinforced length to prevent pull-out failure in the case of internal stability. Accelerations used for these analyses and the concepts used for tensile stress distribution in reinforcing strips have been largely influenced by numerical analyses conducted by Segrestin and Bastick (1988).

Methods of analysis described in previous versions of AASHTO and in FHWA MSE wall design guidelines (FHWA, 1996) include certain factors for estimating earth pressures and inertial loads that have been revised in this section of the Specifications to define a more generalized method of analysis. The rationale for these modifications is described in the NCHRP 12-70 Report (NCHRP, 2008). The NCHRP 12-70 Report provides a discussion of the original methodology developed by Segrestin and Bastick (1988) and the basis for the changes in the approach.

The approach taken for MSE wall design in this section of the Specifications involves use of one of three limit equilibrium methods: (1) the simplified M-O equation,

(2) the wedge equilibrium method or charts for earth pressure coefficient that include cohesion (Appendix B_X), or (3) a generalized limit equilibrium method to estimate wall pressures. A key change from previous M-O methods is that a potential contribution from cohesion in the soil – due to either cohesive content of the soil or capillarity stresses – may be included in the determination of the active earth pressure through the use of either the charts in Appendix B_X or the generalized limit equilibrium method.

An alternate approach involving the use of 2-dimensional finite element or finite difference computer programs may also be acceptable under certain specialized situations. This alternative approach can be very powerful in terms of understanding soil-structure interaction mechanisms during seismic loading. However, considerable skill and experience are required when using this alternate approach, particular when seismic loading is involved. In particular the modeling of the reinforcing layers in the MSE walls is not straight forward and requires special skills in setting up appropriate interface elements for the reinforcing. Most often these numerical methods are more suitable for special research studies. Before using this alternate approach for seismic design of MSE walls, detailed discussions should take place with the Owner.

X.10.2.1 Seismic Active Earth Pressure

Active seismic earth pressures behind the MSE wall shall be determined using either the Mononobe-Okabe (M-O) equation for seismic active earth pressure, the wedge equilibrium method, or the generalized limit equilibrium method. Use of the M-O equation shall be subject to the applicability of the M-O equation at the site as discussed in this Article.

The seismic coefficient used in the earth pressure computation shall be determined

C.X.10.2.1

The demand on the MSE wall during seismic loading includes the inertial response of the wall and the earth pressure developed on the back of the wall. The force on the back of the wall can be estimated by the M-O equation, the wedge equilibrium method, or the generalized limit equilibrium method. These methods are similar to those described previously for rigid and semi-rigid retaining walls, including modifications for wave scattering for walls greater than 20 feet in on the basis of the procedures described in Article X.4. Where small (e.g., 1 to 2 inch) permanent displacements are permissible during the design seismic event, a 50% reduction in k_{max} shall be permitted. A reduction beyond 50% shall be allowed only with the Owner's approval and with displacement analyses that show permanent displacements are within the Owner's performance requirements.

height (i.e., $k_{av} = \alpha\ k_{max}$) . The generalized method is capable of handling a much broader range of design conditions, including problems related to steep backslopes and sloping ground, and therefore is often the preferred of the two.

During a design seismic event, some amount of permanent movement of the wall is usually acceptable to the Owner. By allowing movement, the seismic coefficient used to estimate the active earth pressure can be reduced. A 50% reduction in k_{max} has been used in past editions of the AASHTO *LRFD Bridge Design Specifications* and is adopted in this Article. Generally, this level of reduction results in limited displacements – for example less than 1 to 2 inches.

A large experience base has been developed for designs based on a reduction of 50% in the seismic coefficient used for design. Special studies can be performed to support larger reductions in the seismic coefficient. These special studies should be discussed with and agreed to by the Owner before implementation.

X.10.2.1.1 M-O Equation for Seismic Earth Pressure

Use of the M-O equation for determination of the seismic earth pressure shall be limited to locations where (1) the material behind the MSE wall is a uniform, cohesionless soil within a zone defined by a 3H:1V wedge from the heel of the wall and (2) the combination of peak ground acceleration and backslope angle do not exceed the limits shown in Figure X.7-2.

C.X.10.2.1.1

The M-O equation is inappropriate for use where cohesive soils, rock, or variable non-cohesionless soil types occur within the zone of active pressure development. For these cases the differences in soil properties within the active pressure wedge have a significant effect on the earth pressure calculation – usually resulting in much lower pressures than estimated by the standard M-O equation. The generalized limit equilibrium method described in the next article provides a methodology for handling these variables.

In cases where the soil in the 3H:1V wedge behind the MSE wall is primarily homogeneous, it is possible to use the M-O equation if the soil is a clean, coarse granular material with fines content less than 10%. If the soil has a higher percentage of fines, the charts given in Appendix B_X provide a basis for making adjustments to the seismic active pressure determination.

If the cohesion in the soil behind the wall results primarily from capillarity stresses, then the maximum apparent cohesion should be limited to 50 to 200 psf in consideration of the uncertainties associated with the capillarity stresses under seismic loading conditions (see discussion in NCHRP 12-70 Report (NCHRP, 2008).

Combinations of seismic coefficient and backslope result in the M-O equations not being applicable. In this case the slope angle for the seismic wedge becomes so flat that extremely large seismic loads are estimated. Procedures identified in Article X.7 for retaining walls can be used to determine the limiting conditions.

X.10.2.1.2 Wedge Equilibrium and Generalized Limit Equilibrium Methods for Seismic Earth Pressure

Either the wedge equilibrium method or the generalized limit equilibrium method described in Article X.7.2.1.3 shall be used where the M-O equation described in Article X.10.2.1.1 are not applicable. The force resulting from the limit equilibrium analysis shall be distributed uniformly along the back of the MSE wall.

C.X.10.2.1.2

The wedge equilibrium method or generalized limit equilibrium method are typically used where there is a cohesive content to the soil or where the soil is layered. Appendix B_X provides charts for the wedge equilibrium method.

The generalized limit equilibrium method involves use of a slope stability computer program to estimate wall pressures. The model of the wall is developed by specifying a near vertical, weightless wall at the back of the MSE reinforcing. An external force oriented at the friction angle (upward direction) is imposed at the face of the wall. The force to maintain stability is varied until the ratio of capacity-to-demand ratio is 1.0 (i.e., FS = 1.0) under the imposed seismic coefficient.

This stability analysis should consider all soil or rock layers behind the MSE walls, as well as groundwater conditions.

X.10.2.2 Wall Displacement Analysis

Displacements shall be estimated using one of the procedures given in Article X.4 for cases where (1) the C/D ratio for global stability is less than 1.0 or (2) the amount of sliding allowed by the Owner can exceed 1 to 2 inches, thereby supporting a k_{max} reduction factor of greater than 50%.

For critical structures identified by the Owner, the displacements estimated from the equations in Article X.4 shall be multiplied by 2 to obtain an 84% confidence level.

C.X.10.2.2

Use of the M-O equation involves the same provisions as given in Article X.7.2.1.1 for rigid and semi-rigid retaining walls. When using the M-O equation, the seismic coefficient (k_{max}) after adjustments for factors such as wave scattering and displacement (i.e., 50% factor), is not further adjusted by a factor of (1.45-A)A, as required within the current AASHTO *LRFD Bridge Design Specification*.

As discussed in the report for the NCHRP 12-70 Project (NCHRP, 2008), the (1.45-A)A adjustment is not appropriate based on the work carried out for the NCHRP 12-70 Project. Further, the common practice of MSE wall vendors to assume that the (1.45-A)A adjustment accounts for site effects (i.e., the same as the F_{pga} factor noted in Article X.4) appears to be an error in interpretation of the original work by Segrestin and Bastick. The approach recommended in this Article of the Specifications is to adjust for site effects using the methodology described in Article X.4.

X.10.3 Design Requirements

MSE wall shall be designed to meet global stability, external stability, and internal stability requirements set forth in this section of the Specifications. Earth pressures and displacement evaluations discussed in the preceding Articles in this Section shall be used as input for these analyses.

C.X.10.3

The global and external stability assessments described in this Article can be accomplished with relatively simple limit equilibrium computer methods for global stability and with spreadsheets or MathCAD templates for external stability and are similar to the assessment for rigid and semi-rigid gravity retaining walls.

Simplified computer methods for evaluation of internal stability consistent with the approach described in this Article have not, however, been developed. Available computer methods for evaluating internal stability, MSEW (Adama, 2005a) and ReSSA (Adama, 2005b), use the methodology described in the current AASHTO *LRFD Bridge Design*

Specifications, and therefore will require modification to account for the procedures being recommended.

X.10.3.1 Global Stability

A global stability analysis shall be conducted to determine the capacity of the MSE retaining structure to resist seismic loads. The seismic coefficient (k_{max}) defined in Article X.4 shall be used to evaluate global stability under seismic loading. A 50% reduction in k_{max} shall be permitted for locations where small permanent displacement is tolerable, unless the Owner requires otherwise.

If the capacity-to-demand ratio (C/D) for global stability is 1.0 or higher under a k_{max} that has been reduced by 50%, the global stability shall be considered acceptable. If the C/D ratio is less than 1.0, either the potential for movement of the wall during seismic loading shall be estimated and reviewed with the Owner for acceptability or the wall shall be redesigned to meet capacity-to-demand requirements. Procedures presented in Article X.4 shall be used to determine the potential for permanent ground movement.

If ground displacement cannot be tolerated for the particular situation or is not acceptable to the Owner, ground improvement methods shall be used to achieve an acceptable condition.

C.X.10.3.1

The global stability analysis is conducted using a conventional slope stability computer program. The model used to represent the wall should extend from the area in front of the wall to the slope or flat area behind the wall. Trial surfaces should also consider both sliding at the bottom of the MSE wall ending in front of the wall and sliding along deeper surfaces that have lower strengths.

The evaluation of global stability should include checks on sliding surfaces that pass through the MSE wall for locations where a fill slope is constructed above the MSE wall to confirm that critical failure conditions do not occur higher in the wall. For these checks a modified wave scattering coefficient (α) would be applicable to reflect failure surface heights for cases where wave scattering is included in the determination of k_{max}.

If a reduced seismic coefficient is used for estimating the seismic earth pressures (i.e., reduction greater than 50% of the peak seismic coefficient), the displacement associated with this reduced seismic coefficient should be estimated following the methods given in Article X.10.2.2.

If the peak seismic coefficient is used in the limit equilibrium analysis and the capacity-to-demand ratio is less than 1.0, then the permanent displacement should be estimated following the methods given in Article X.10.2.2.

X.10.3.2 External Stability

The external stability of the MSE wall shall be evaluated to show that the MSE wall meets sliding, overturning, and bearing stability requirements. The ratio of capacity-to-demand (C/D ratio) shall be greater than 1.0 for the following resistance factors:

Sliding: 1.0

C.X.10.3.2

The checks on external stability treat the MSE wall section as a rigid block. This representation is not correct for the overturning and bearing stability evaluations due to the flexibility of the reinforced soil mass. However, in the absence of rigorous treatments of these mechanisms, the C/D checks for these modes of failure are

Overturning: 1.0

Toe Bearing: 1.0

Earth pressures defined previously shall be used in the external stability assessment. These earth pressures shall be placed behind the reinforced soil block for the stability analysis. In the event that reinforcement is not constant in length, the average reinforcement length shall be used.

included as part of the external stability evaluation.

Sliding Stability

Sliding stability is evaluated using the interface friction at the base of the foundation. The inertia of the reinforced zone is included in this evaluation. The inertial force is defined by the same seismic coefficient used for the earth pressure determination times the entire mass of the soil contained within the reinforcing strips.

The use of the full inertial force in the sliding stability evaluation differs from recommendation in the current AASHTO *LRFD Bridge Design Specifications* and is thought to be more fundamentally accurate, assuming that the soil within the reinforced zone is moving in phase during seismic loading. However, when reinforcing strips start becoming very long, as in the case of MSE walls with steep backslopes in moderately-to-highly seismic areas, some incoherency is likely to occur, which introduces excessive conservatism if the full length of the reinforcing strips is used in the inertia determination. For practical considerations, it is suggested that that maximum zone for inertial response be limited to 0.7 times the height of the MSE walls. In effect this criterion eliminates the contributions of the backslope angle to the inertial force determination – which intuitively is reasonable.

The seismic coefficient that results in a capacity-to-demand ratio of 1.0 (FS = 1.0) is the limiting seismic coefficient. If the seismic coefficient determined for the site is less than this value, displacements must be checked, and if the displacements are too high, the MSE wall design needs to be changed.

Overturning Stability

Overturning stability is evaluated by summing moments about the toe of the MSE wall. The inertia of the wall assuming the full seismic coefficient is used in this evaluation – with limitations as described above.

The inherent flexibility of the MSE wall limits the value of this check. However, low capacity-to-demand ratios will warn the designer of likely distortion of the wall rather than actual overturning. The amount of distortion cannot be estimated with simplified models. Cases where the C/D ratio are not met should be used as a warning to the designer that the facing on the wall and the internal reinforcing strips could be damaged during the seismic event. This damage could be costly to repair, but an overturning collapse of the MSE wall is not expected.

Bearing Stability

Bearing stability is checked by comparing the peak bearing pressure at the toe of the foundation to the ultimate bearing capacity of the soil, assuming an equivalent rectangular bearing pressure. No reduction is made for eccentric loading. A rigid block assumption is used when performing this check. A resistance factor of 1.0 is used for toe bearing (compared to 0.67 for rigid and semi-rigid retaining walls), as MSE walls can better accommodate small rotational deformations.

A maximum theoretical liftoff of 50% of the footing width is permitted during the seismic event. The ultimate bearing capacity is based on the effective width of the footing (i.e., after liftoff) and calculated using total stress soil properties.

While this approach will suggest that liftoff can occur, the flexibility of the MSE soil mass and the lack of tensile strength in the soil make this unlikely.

X.10.3.3 Internal Stability

The internal stability of the reinforcing system and the facing elements shall be evaluated and shown to meet seismic loading demands.

C.X.10.3.3

The current AASHTO design method for seismic internal stability assumes that the internal inertial forces generating additional tensile loads in the reinforcement act on an active pressure zone which is assumed to be the same as that for the static loading case. A bilinear zone is defined for inextensible reinforcements such as metallic strips, and a linear zone for extensible strips.

Whereas it could reasonably be anticipated that these active zones would extend outwards for seismic cases, as for M-O analyses, results from numerical and centrifuge models indicate that the reinforcement restricts such outward movements, and only relatively small changes in location are seen.

In the current AASHTO method, the total inertial force is distributed to the reinforcements in proportion to the effective resistant lengths L_{ei}. This approach follows the finite element modeling conducted by Segrestin and Bastick (1988), and leads to higher tensile forces in lower reinforcement layers. This is the opposite trend to incremental seismic loading used by AASHTO for external stability evaluations based on the M-O active pressure equation.

In the case of internal stability evaluation, Vrymoed (1989) used a tributary area approach that assumes that the inertial load carried by each reinforcement layer increases linearly with height above the toe of the wall for equally spaced reinforcement layers. A similar approach was used by Ling et al. (1997) in limit equilibrium analyses. This concept would suggest that longer reinforcement lengths could be needed at the top of walls with increasing acceleration levels, and the AASHTO approach could be unconservative.

In view of this uncertainty in distribution which has been widely discussed in the literature, a suggested compromise is to use a uniform pressure distribution from the top to the bottom of the wall. This uniform pressure distribution should be determined from the total inertial force using k_{max} (after reduction for wave scattering and permanent ground displacement) and 0.7H.

A computer program MSEW (ADAMA, 2005) has been developed and is commercially available to design MSE walls using the current AASHTO *LRFD Bridge Design Specifications.* The suggested recommendations to modify the seismic design procedure (acceleration coefficients and tensile load distribution) cannot be directly incorporated in the program, but changes to the source code could be made with little effort, and the design impact of the changes examined by studying several examples.

X.11 PREFABRICATED MODULAR WALLS

X.11.1 General

Prefabricated modular walls shall be designed for seismic loading except where conditions given in Article X.4 are satisfied or where allowed otherwise by the Owner.

Before conducting the seismic evaluation, the prefabricated modular wall shall be designed to meet all gravity and live load requirements in accordance with the provisions in Section 11 of the AASHTO LRFD Bridge Design Specifications.

C.X.11.1

The prefabricated modular wall develops resistance to seismic loads from both the geometry and weight of the wall section. The primary design issues for seismic loading are global stability, external stability (i.e., sliding, overturning, and bearing), and internal stability. External stability includes the ability of each lift within the wall to also meet external stability requirements. Interlocking between individual structural sections and the soil fill within the wall needs to be considered in this evaluation.

The starting point for the seismic design of the prefabricated modular wall is an acceptable static design, meeting the requirements of the current AASHTO *LRFD Bridge Design Specifications*. Once the static design has been completed, the design is checked for seismic response.

As part of the seismic check, the performance expectations for the wall during the design seismic event should be determined through discussions with the Owner. Some of the factors that should be considered in determining performance expectations are summarized in Appendix A_X.

X.11.2 Method of Analysis

The prefabricated modular wall shall be analyzed to show that the wall can

C.X.11.2

The seismic design of the prefabricated modular wall generally involves the same withstand seismic forces from seismic earth pressures and from inertial forces of the wall without excessive sliding or rotation of the wall and without exceeding stress limits within the structural system.

Either limit equilibrium or displacement methods shall be used to establish that performance during seismic loading meets design expectations.

seismic analyses as used for rigid and semi-rigid gravity walls described in Article X.7 of this section of the Specifications. The one significant difference is that the sliding and overturning stability at different levels within the modular wall also need to be confirmed. For these checks the earth pressure will differ not only because of the reduced height, but also because of different wave scattering effects.

The approach taken for prefabricated modular wall design in this Article involves the use of the simplified M-O equation, the wedge equilibrium method, or a generalized limit equilibrium method to estimate wall pressures. The contributions of any cohesion within the soil behind the wall should be accounted for in the earth pressure determination. Either the generalized limit equilibrium or the figures in Appendix B_X can be used to account for these effects.

An alternate approach involving the use of 2-dimensional finite element or finite difference computer programs is also acceptable. As noted for other walls, considerable skill and experience are required when using these numerical methods, particular when seismic loading is involved. Most often these numerical methods are suitable for special studies. Before using this alternate approach for seismic design of prefabricated modular walls, detailed discussions should take place with the Owner.

X.11.2.1 Seismic Earth Pressure

Procedures given in Article X.7.2.1 shall be followed to determine the seismic earth pressure that will be imposed on the wall. Wall pressures shall be estimated at multiple heights behind the wall for use in external and internal stability checks.

The seismic coefficient used in the earth pressure computation shall be determined following the procedures described in Article X.4. Where small (e.g., 1 to 2 inch) permanent displacements are permissible

C.X.11.2.1

Prefabricated modular walls will often be constructed at locations where soil conditions preclude the use of the Mononobe-Okabe (M-O) equation. The cohesion in these soils has a significant effect on the earth pressure, which the M-O equation cannot capture in a simple relationship. In this situation the wedge equilibrium method or the generalized limit equilibrium method offers an alternative for estimating wall pressures. For those special conditions where the fill behind during the design seismic event, a 50% reduction in the peak ground acceleration used in design shall be permitted. A reduction beyond 50% shall be allowed only with the Owner's approval and with displacement analyses that show permanent displacements are within the Owner's performance requirements.

the wall is a clean homogeneous granular material, the M-O equation for seismic active pressure can be used.

If soils are homogeneous but have a cohesive content, the figures in Appendix B_X can be used to account for the cohesive contribution. See Article X.7.2.1.1 for additional considerations relative to the determination of seismic active earth pressures, including the contributions from cohesion.

Procedures for estimating the seismic coefficient to use in the generalized limit equilibrium, wedge equilibrium, or M-O equation are as described in Article X.7.2.2. A reduction in the peak seismic coefficient for wave scattering (k_{av}) is permitted for wall heights between 20 and 70 feet as discussed in Article X.4. Other provisions are as described in the commentary to Article X.7.2.1.

The geometry of the prefabricated modular wall is such that determination of the passive earth pressure at the face of the wall can usually be ignored. For those cases where the wall is embedded more than a few feet, the passive pressure for seismic loading should also be determined and used in the stability analyses. Procedures described in Article X.7.2.1.2 can be used to determine the passive earth pressure.

X.11.2.2 Wall Displacement Analysis

Displacements shall be estimated using one of the procedures given in Article X.4 for cases where (1) the C/D ratio for global stability is less than 1.0 or (2) the amount of sliding allowed by the Owner can exceed 1 to 2 inches, thereby supporting a k_{max} reduction factor of greater than 50%.

For critical structures identified by the Owner, the displacements estimated from the equations in Article X.4 shall be multiplied by 2 to obtain an 84% confidence level.

C.X.11.2.2

The displacement estimate for the prefabricated modular wall can be made by following the steps outlined in Article X.7.2.2 of the Specifications.

X.11.3 Design Requirements

The prefabricated modular wall shall be designed to meet global stability, external stability, and internal stability requirements as set forth in this Article. Earth pressures and displacement evaluations discussed in the preceding Articles shall be used as input for these analyses.

C.X.11.3

The seismic performance of the prefabricated modular block wall should be conducted following the same general procedures as used for the rigid and semi-rigid gravity wall, as described in Article X.7.3.

The primary difference for this wall type relative to a rigid or semi-rigid gravity wall is that sliding and overturning can occur at various heights between the base and top of the wall, as this class of walls typically uses gravity to join sections of the wall together.

The interior of the wall is normally filled with soil, and this provides both additional weight and shear between structural elements. The contributions of the earth, as well as the batter on the wall, need to be considered in the analysis.

X.11.3.1 Global Stability

Procedures given in Article X.7.3.1 shall be used to check global stability. The results of these analyses shall demonstrate that the capacity-to-demand ratio is greater than 1.0. If the capacity-to-demand ratio is less than 1.0, displacements shall be estimated or the wall shall be redesigned to meet the capacity-to-demand requirements.

C.X.11.3.1

The global stability check needs to consider failure surfaces that pass through the wall section, as well as below the base of the wall. The check on stability at mid level must consider the contributions of both the soil within the wall and any structural interlocking that occurs for the particular modular wall type.

When checking stability at the mid level of a wall, the additional shear resistance from interlocking of individual structural members will depend on the specific wall type. Usually interlocking resistance is provided by the wall supplier. The interlocking forces are often such that the critical load case becomes external and internal stability rather than global stability within the wall height.

X.11.3.2 External Stability

The external stability of the prefabricated modular wall shall be evaluated to show that the prefabricated wall section meets sliding, overturning, and bearing stability requirements. The sliding

C.X.11.3.2

The evaluation of external stability will be similar to procedures described in Article X.7.3.1 with the additional provision that checks need to be performed at different heights within the wall, as discussed above and overturning requirements shall be satisfied for both the entire wall, as well as individual levels within the wall.

The ratio of capacity-to-demand shall be greater than 1.0 using the following resistance factors:

Sliding:	1.0
Overturning:	1.0
Toe Bearing:	0.67

Earth pressures defined previously shall be used in the external stability assessment. If the ratios identified are not satisfied, the prefabricated modular wall shall be re-sized to meet the required capacity-to-demand ratios.

for global stability. These additional checks need to consider the capacity from interlocking structural members of the wall relative to each other. Careful review of the particular wall type will be required to evaluate these forces.

When evaluating sliding and overturning stability at varying heights within the wall, the seismic wall pressure needs to be adjusted for scattering effects if scattering adjustments are included in the analyses. Generally, the scattering factor decreases as the wall height above the elevation of interest decreases. For evaluations where the height of the wall above the elevation is 20 feet or less, the scattering coefficient should be 1.0.

X.11.3.3 Internal Stability

Internal stability of the prefabricated modular wall shall be evaluated for maximum moments and shears developed during sliding, overturning, and bearing for load and resistance factors provided in Article X.6.. Design requirements shall be consistent with the Owner's performance expectations for Extreme Event I.

C.X.11.3.3

This class of walls is typically involves proprietary wall systems comprised of wall segments that interlock. The interlock is often through gravity connections, but mechanical systems can also be used. In most cases it will be necessary to require the wall vendor to provide submittals showing that the wall will meet the Owner's performance objectives under the imposed seismic loads.

For these design checks, the earth pressure determined from Article X.11.2.1 should be used by the vendor with the required resistance factors, the soil bearing capacity, and the soil sliding resistance to show internal stability.

X.12 SOIL NAIL WALLS

X.12.1 General

Soil nail walls shall be designed for seismic loading except where conditions given in Article X.4 are satisfied or where allowed otherwise by the Owner.

Before conducting the seismic evaluation, the soil nail wall shall be designed to meet all gravity and live load

C.12.1

Soil nail walls respond to seismic loading much the same as an MSE wall. Inertial forces developed in the soil mass behind the wall must be resisted by the mass of the soil nail wall area. Critical modes of failure that must be considered include the global failure of the soil nail wall area, as well as the requirements in accordance with the FHWA soil nail design guideline (FHWA, 2003).

external stability issues of sliding along or under the base of the wall, overturning of the wall, and bearing capacity of the soil supporting the wall.

The soil nail wall also must consider the internal stability of the wall. In this case internal stability refers to the load transfer between the soil nails and the surrounding soil. These nails are typically located on 5- to 6-foot spacing and consist of a high strength (75 ksi) steel rebar varying in size from No. 8 to No. 13 or larger. During seismic loading, the nails need to transfer seismic forces without exceeding the soil-grout interface strength, the tensile capacity of the soil nail, and the load transfer at the face of the wall.

As with other wall types, the starting point for the seismic design of the soil nail wall is an acceptable static design. No AASHTO *LRFD Bridge Design Specifications* currently exist for soil nail walls. An NCHRP project titled *LRFD Soil-Nailing – Design and Construction Specifications* was completed in 2005 as part of NCHRP Project 24-21 (NCHRP, 2005). The NCHRP report includes recommended specifications and commentary on the design of soil nail walls. The information in the NCHRP 24-21 report may eventually form the basis for a new section in the AASHTO *LRFD Bridge Design Specifications* covering soil nails. However, at the time that this section of the Specifications was prepared, the status of the NCHRP 24-21 work was still being considered by AASHTO committees, and therefore, a decision was made not to use it as a basis for preparing this section of the Specifications.

Until the AASHTO *LRFD Bridge Design Specifications* are updated to include soil-nail walls, the design of soil nail walls should be completed following guidelines in the FHWA Geotechnical Engineering Circular No. 7 *Soil Nail Walls* (FHWA, 2003). This circular describes two computer codes that are used in the design of soil nail walls, GOLDNAIL and SNAIL. The approach recommended in the FHWA Circular uses the same seismic amplification factor as recommended for MSE walls in the current AASHTO *LRFD Bridge Design Specifications*. The approach also allows for reduction in the seismic coefficient if the wall can tolerate displacement.

The seismic design of soil nail walls described in this section of the Specifications differs from methods given in FHWA Geotechnical Engineering Circular No. 7 *Soil Nail Walls* (FHWA, 2003) in two important areas: (1) a different method is used to develop the seismic coefficient and (2) the equations for estimating the amount of permanent displacement are revised. The methodology still relies on the computer programs GOLDNAIL or SNAIL to perform the stability assessments, as these methods are considered by the profession as acceptable methods for soil nail wall design, and they do not depend on the changes from the FHWA method being suggested in this section of the Specifications.

As part of the seismic check, the performance expectations for the wall during the design seismic event should be determined through discussions with the Owner. Some of the factors that should be considered in determining performance expectations are summarized in Appendix A_X.

X.12.2 Method of Analysis

Analyses shall be conducted to show that the soil nail wall can resist forces due to seismic earth pressures and inertial forces without excessive sliding or rotation of the soil nail wall and without failure of the structural components of the wall. Either limit equilibrium or displacement methods shall be used to establish that performance meets design expectations during the seismic event.

C.12.2

The approach taken for soil nail wall design in this section of the Specifications involves the use of the standard limit equilibrium methods incorporated within the computer programs GOLDNAIL or SNAIL. These programs are used rather than a conventional slope stability program because a number of factors are involved in the determination of the soil nail length, and these are best determined by a purpose-built computer program. The seismic coefficient described in Article X.4 is used as input to the analyses. The seismic coefficient can be reduced by up to 50% if small permanent displacements of the soil nail wall area can be accepted, which is usually the case.

When setting up the seismic model for the soil nail wall in the computer program GOLDNAIL or SNAIL, it is very important to model the soil with soil properties that account for the cohesion in the soil, as well as the frictional characteristics. For most cases of seismic loading the total stress strength parameters of the soil will be appropriate for design.

Alternate methods of evaluating the seismic response of the soil nail wall can also be used. These alternate methods include using a 2-dimensional finite element or finite difference computer program. Similar to the comments made for other wall designs, before using this alternate approach for seismic design of soil nail walls, detailed discussions should take place with the Owner.

X.12.2.1 Seismic Coefficient

The seismic coefficient used in the soil nail analysis shall be determined on the basis of the procedures described in Article X.4. Where small (e.g., 1 to 2 inch) permanent displacements are permissible during the design seismic event, 50% reduction in k_{max} shall be permitted. A reduction beyond 50% shall be allowed only with the Owner's approval and with displacement analyses that show permanent displacements are within the Owner's performance requirements.

C.X.12.2.1

The seismic coefficient recommendation is different than what is given in the FHWA Circular No. 7 (FHWA, 2003). As discussed in Article X.10.2.1, the (1.45-A)A amplification adjustment factor is based on soil-structure interaction studies conducted by Segrestin and Bastick for MSE walls. Results of further review of this adjustment factor made during the NCHRP 12-70 Project (NCHRP, 2008) concluded that this adjustment is not appropriate for MSE walls, and therefore, use of this adjustment for soil nail walls is not recommended.

A reduction in the peak seismic coefficient by 50% is allowed as long as small permanent displacements are permissible to the Owner, similar to the reduction used for other types of gravity walls. Modifications for wave scattering are also permitted for walls greater than 20 feet in height.

X.12.2.2 Displacement Analysis

Estimates of soil nail wall displacements shall be made where (1) the capacity-to-demand ratio from the soil nail analysis is less than 1.0 (i.e., FS < 1.0) or (2) where required by the Owner.

Procedures discussed in Article X.4 shall be used to make the displacement estimate. For critical structures identified by the Owner, the maximum displacements shall be multiplied by 2 to obtain an 84% confidence level.

C.X.12.2.2

The displacement estimate for the soil nail wall can be made by following the steps outlined in Article X.7.2.2 of this section of the Specifications.

X.12.3 Design Requirements

The soil nail wall shall be designed to meet global stability, external stability, and internal stability requirements set forth in this Article. The seismic coefficient discussed in the preceding Articles shall be used as input for these analyses.

C.12.3

The seismic design requirement for a soil nail wall will be met by confirming that when the seismic coefficient is used in the SNAIL or GOLDNAIL analyses that the appropriate pullout, tensile strength, and punching shear requirements are satisfied.

The computer programs for evaluating soil nail wall design include checks on global stability. Alternate checks can be conducted on global and external stability as discussed below.

X.12.3.1 Global Stability

Where desired, the global stability shall be checked with a limit equilibrium slope stability computer program to confirm that the capacity-to-demand for the soil nail wall during seismic loading is greater than 1.0. If the capacity-to-demand ratio is less than 1.0, the displacement of the wall during seismic loading shall be determined or the wall shall be redesigned to meet capacity-to-demand requirements.

C.X.12.3.1

A limit-equilibrium computer program can be used to perform the stability check. The seismic coefficient used in this check is the coefficient discussed in Article X.4 adjusted for permanent wall displacement. For walls in excess of 20 feet in height, adjustments for wave scattering can be considered, as discussed in Article X.4.

If a generalized limit equilibrium analysis is conducted, the total stress strength parameter of the soil, with both c and ϕ, should be included in the slope stability model. If the resulting capacity-to-demand ratio is less than 1.0 (FS < 1.0), the displacement of the wall needs to be determined following the method given in Article X.7.2.2 or the wall should be redesigned.

X.12.3.2 External Stability

External stability for sliding, overturning, and bearing shall be checked to confirm that the capacity-to-demand ratio is satisfied for seismic loading.

Where independent evaluations of capacity-to-demand ratio are used to evaluate external stability (i.e., not using SNAIL or GOLDNAIL), the ratio of capacity-to-demand shall be greater than 1.0 for the following resistance factors:

Sliding:	1.0
Overturning:	1.0
Toe Bearing:	1.0

Earth pressures defined by the generalized limit equilibrium methods shall be used in the external stability assessment.

C.X.12.3.2

In most cases the external stability of the soil nail wall will be sufficient if computer programs such as GOLDNAIL or SNAIL are used. For those cases where an alternate simplified method is used, the capacity-to-demand ratio should be checked for the design. Simplified computer or hand methods can be used to make these checks.

If simplified computer or hand methods are used, the wedge equilibrium method or the generalized limit equilibrium method, as discussed for rigid and semi-rigid gravity walls, should be used to estimate the seismic earth pressure. As noted previously, the M-O procedure is generally limited in use to locations where soils are homogenous and granular. These conditions often do not occur at the planned location of a soil nail wall. Soil nail walls are normally used for cut slopes where the natural geology includes layers of varying soil types. The soil nail walls are also best suited for sites where some amount of soil cohesion exists to meet face stability requirements during construction. For sites with cohesion either the generalized equilibrium approach or figures in Appendix B_X can also be used for estimating the seismic coefficient.

X.12.3.3 Internal Stability

Results of the seismic analysis shall confirm that the soil nail wall meets internal stability requirements for punching shear at the face of the wall and tensile strength of the soil nail under the assigned seismic coefficient using load and resistance factors provided in Article X.6..

C.X.12.3.3

Results of analyses using GOLDNAIL and SNAIL will allow evaluation of the punching shear at the wall face and whether nail reinforcing requirements are met for the additional loading from the seismic event. Results also provide information on the type of the failure: nail pullout failure, slippage at the bar-grout interface, tensile failure of the nail, and bending and shear of the nail.

For seismic design checks the limiting values of nail pullout failure force, bar-grout interface shear, and permissible bending and shear forces in the nail should be the same as used in static design.

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APPENDIX A_X

STRATEGY FOR OWNER DECISION-MAKING ON ACCEPTABLE DISPLACEMENTS FOR RETAINING WALLS

This appendix provides a strategy for Owners to use when determining the amount of permanent displacements that is acceptable for certain retaining wall types during seismic loading. This strategy is to be used with Section X of the Specifications and Commentaries prepared as part of the NCHRP 12-70 Project *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments.*

A_X.1 BACKGROUND

The Specifications and Commentaries prepared for the NCHRP 12-70 Project include provisions that allow for permanent displacements of some types of retaining walls during seismic loading. The specific types of retaining walls that can accommodate permanent displacements include gravity and semi-gravity walls, mechanically stabilized earth (MSE) walls, soil nail walls, and prefabricated modular walls.

For this set of retaining walls sliding and rotation of the walls can occur during a seismic event. The permanent movement can be associated with global stability, where the entire soil mass with the retaining wall moves; or it can be associated with sliding of the retaining wall, which is part of the external stability assessment required within the Specifications and Commentaries. In many situations some small amount of permanent sliding or rotation is permissible after the design seismic event. In these cases the structure can withstand movement without structural damage or collapse.

If permanent movement of the retaining wall is permissible, there are significant benefits to the Owner:

- The Specifications and Commentaries allow reduction in the seismic coefficient used in design by 50% if 1 to 2 inches of permanent movement are acceptable. This reduction in the seismic coefficient is similar to the procedure that appears in the current AASHTO *LRFD Bridge Design Specifications*. However, greater movement is associated with the method given in current AASHTO Specifications. The NCHRP 12-70 Project refined the level of displacement that is associated with the 50% reduction, such that the permanent movements are approximately half of what currently is cited in the current AASHTO *LRFD Bridge Design Specifications*. If the same displacement magnitude is maintained as in the current AASHTO *LRFD Bridge Design Specifications*, the reduction in the seismic coefficient can be greater than 50%. As the seismic coefficient is reduced, the seismic demands on the retaining wall decrease.
- The NCHRP 12-70 Specifications and Commentaries provide a methodology for estimating the amount of deformation for Owners who cannot or do not want to use the 50% reduction. This methodology is useful in situations where larger deformations than associated with the 50% reduction in seismic coefficient are acceptable (e.g., more than 1 to 2 inches) and where the capacity-to-demand (C/D) ratio does not meet the target of 1.0 with the 50% reduction. Rather than redesigning the wall to achieve the C/D of 1.0 or

more, the amount of permanent displacement can be calculated. If the amount of displacement is within reasonable limits, the seismic design of the retaining wall may be acceptable even though the C/D ratio is less than desired.

An over-riding question in any approach that involves permanent deformations is the amount that is acceptable. The Specifications and Commentaries to the NCHRP 12-70 Project leave this decision to the Owner, who must weigh a number of factors in reaching this decision. Typically, a few inches of movement are acceptable; however, there are situations where even this level of deformation may be unacceptable. On the other extreme, some retaining walls may be able to tolerate several feet or more of movement, particularly if the movement is primarily sliding without rotation. A number of factors should be considered by the Owner when deciding the acceptable amount of permanent displacement and rotation as summarized below.

A_X.2 CONSIDERATIONS FOR ESTABLISHING ACCEPTABLE DISPLACEMENTS

The factors that should be considered when deciding on acceptable levels of permanent displacement range from implications of the movement to likely mode of wall movement. When considering these factors, the Owner should evaluate both the relative consequences of movement and, as appropriate, the cost of designing to avoid the movement.

A_X.2.1 Wall Location and Function

One of the main factors for deciding on the acceptable level of movement involves the location and function of the wall.

- Walls in urban locations usually can tolerate less movement than walls located in the countryside. Part of this relates to effects of wall movement on utilities and other nearby facilities, and part relates to aesthetics. After a design seismic event a wall that has moved 12 inches or more in the countryside may be completely functional and acceptable, but this same wall may not be accepted in an urban environment.

- Walls that support a heavily traveled roadway should usually be designed for smaller displacements than walls that are part of a less traveled roadway. This relates to loss of function if there is damage associated with wall movement. Generally, less traveled roadways can remain unusable for a longer period of time, and therefore, large amounts of damage from permanent movement are acceptable. On the other hand, roadways with heavy use will result in significant traffic and economic disruption if they are out of service for even a few hours. For this situation it may be very important to limit displacement to levels that will have minimal disruption to service.

- Walls that pose a large risk to public safety should be designed for less movement than walls that represent low risk. The relationship between risk and displacement is qualitative and relates mainly to the uncertainties associated with the deformation prediction. As the amount of displacement is allowed to increase, there is a greater possibility that the wall performs different than predicted – simply because uncertainties in the estimate of movement increase as the magnitude of movement increases. If there is a large risk associated with this performance, then the Owner is obligated to take a more conservative approach to design, which often will mean minimizing acceptable movements.

A$_X$.2.2 Wall Type

Some walls have performed much better than others during seismic events. This better performance resulted for various reasons – better construction, more flexibility, and inherent stability. This better performance also provides some confidence that this set of retaining walls can undergo larger permanent movement in future seismic events.

- Semi-gravity and gravity walls are susceptible to structural damage, and therefore care should be used when designing these walls for large permanent displacements. This caution is related to limitations in design methods, particularly the ability to estimate the rotation of the wall associated with any translation. The consequences of failure for this type of wall can be a relatively rapid collapse, and therefore requires a more cautious approach to design. However, a well-designed semi-gravity or gravity wall should be able to slide 1 to 2 feet without danger.

- Some walls such as MSE and soil nail walls are relatively flexible, and therefore can undergo relatively large displacements without failure. The internal reinforcing systems for these walls also provide significant redundancy in the event that some internal failures occur. Past experience suggests that when these walls failure, the damage is relatively limited. Movements of more than 1 to 2 feet should be tolerable for this type of wall.

Note that some wall types, such as nongravity cantilever walls and anchored walls, were not included in the set of walls identified as allowing permanent displacement. For these walls the amount of permanent movement that can be tolerated without structural damage for some walls can be small, and therefore, the 50% reduction in the seismic coefficient for displacement is not allowed without special study.

AX.2.3 Wall Geometry

The following factors related to geometry may influence the allowable displacements of a retaining wall:

- Generally, shorter walls can tolerate more permanent displacement than taller walls. The height consideration results for several reasons: (1) the seismic forces for shorter walls are lower, (2) the overturning moments are lower, and (3) the life-safety issues are less. The issue of overturning becomes particularly important for tall walls, where there is a tendency for rotation even where efforts are made to force the wall to slide before it overturns.

- Walls that are designed with a batter are better able to handle permanent movement than walls with a vertical face. This consideration relates primarily to the tendency of rotation to occur with sliding. Often the rotation is more of an aesthetic than design concern – as the wall appears to be tilting towards an overturning failure. By incorporating 5 or 10 degrees of batter in the wall, any rotation during a design seismic event will be less noticeable.

- The slope of the ground above and below the wall will affect the capacity of the wall during seismic loading. Walls that have sloping ground downslope can become less stable with displacement, and walls with slopes above the top of the wall can lead to failure of the slope above the wall if large permanent movements of the wall occur. As the steepness of the slope increases, these secondary effects become more significant, even when they are considered during design.

AX.2.4 Types of Soil

The type of soil at a site also should be considered when establishing displacement limits. This consideration is related to both the loads that develop on the wall and the response of the wall to these loads.

- Displacement of walls located on stiff clays or dense cohesionless soils will likely have a higher reliability than for softer soils. As soils become softer, the load-displacement mechanisms become more complex, and the possibility of unexpected performance becomes greater.
- Walls constructed with cohesive soil behind the retaining wall will likely have an inherent level of conservatism incorporated in the design, even when following the wedge equilibrium or the generalized limit equilibrium methods described in the Specifications and Commentaries. This conservatism will generally lead to smaller deformations during the seismic event than are being predicted.
- The confidence in displacement predictions for liquefiable soils is relatively low. If liquefaction is predicted at a wall location, it is generally better to mitigate the liquefaction condition or to select a wall type that will perform adequately if the soil does liquefy. While it is possible to make estimates of wall displacement using residual strengths, the possibility of performance being different than expected increases for this situation.

AX.2.5 Implications of Wall Movement

Perhaps the easiest consideration to understand is the effects that wall movement will have on other facilities in proximity to the wall. Examples of these effects are summarized below.

- Utilities, sidewalks, and pavements located in front of or behind the wall could be affected by permanent movement of the soil. Generally, the zone affected in front of the wall will be triangular extending from the base of the wall coming to the surface at approximately 2.5 times the embedded depth of the wall. Anything within this zone will potentially be displaced outward and upward. The amount of displacement can be approximated by the amount of permanent displacement being estimated.
- Most permanent displacements above the wall will occur within approximately 1 wall height behind the wall (i.e., defined by a 45 degree slope extending from the bottom of the wall). Any utilities, sidewalks, pavements, or other permanent structures located in this area could undergo both outward movement and settlement. The amount of outward movement would be approximated by the permanent movement, and the settlement would be as much as 10 percent of this outward movement.
- Wall aesthetics are also affected by permanent displacements. As noted previously, wall rotation often occurs with taller walls, and this rotation can be visually unacceptable. Movement of MSE walls can also result in unevenness of panel locations, which can be

observed by the public. Generally, as the amount of movement increases, the amount of distortion becomes more noticeable.

AX.3 Approach for Defining Acceptable Displacements

As summarized above, many factors must be considered when deciding on the acceptable level of displacement for a retaining wall. These factors make the development of a simple strategy for establishing the permanent displacement difficult. As soon as displacements of more than 1 to 2 inches are being considered, the Owner should perform a rigorous review of the possible consequences of movement to the wall and to facilities located in proximity to the wall.

Figure A_X-1 shows the steps that the Owner might use in conjunction with these Specifications and Commentaries to define an acceptable limit for permanent displacement of retaining walls. Figure A_X-2 shows the overall design process for retaining walls.

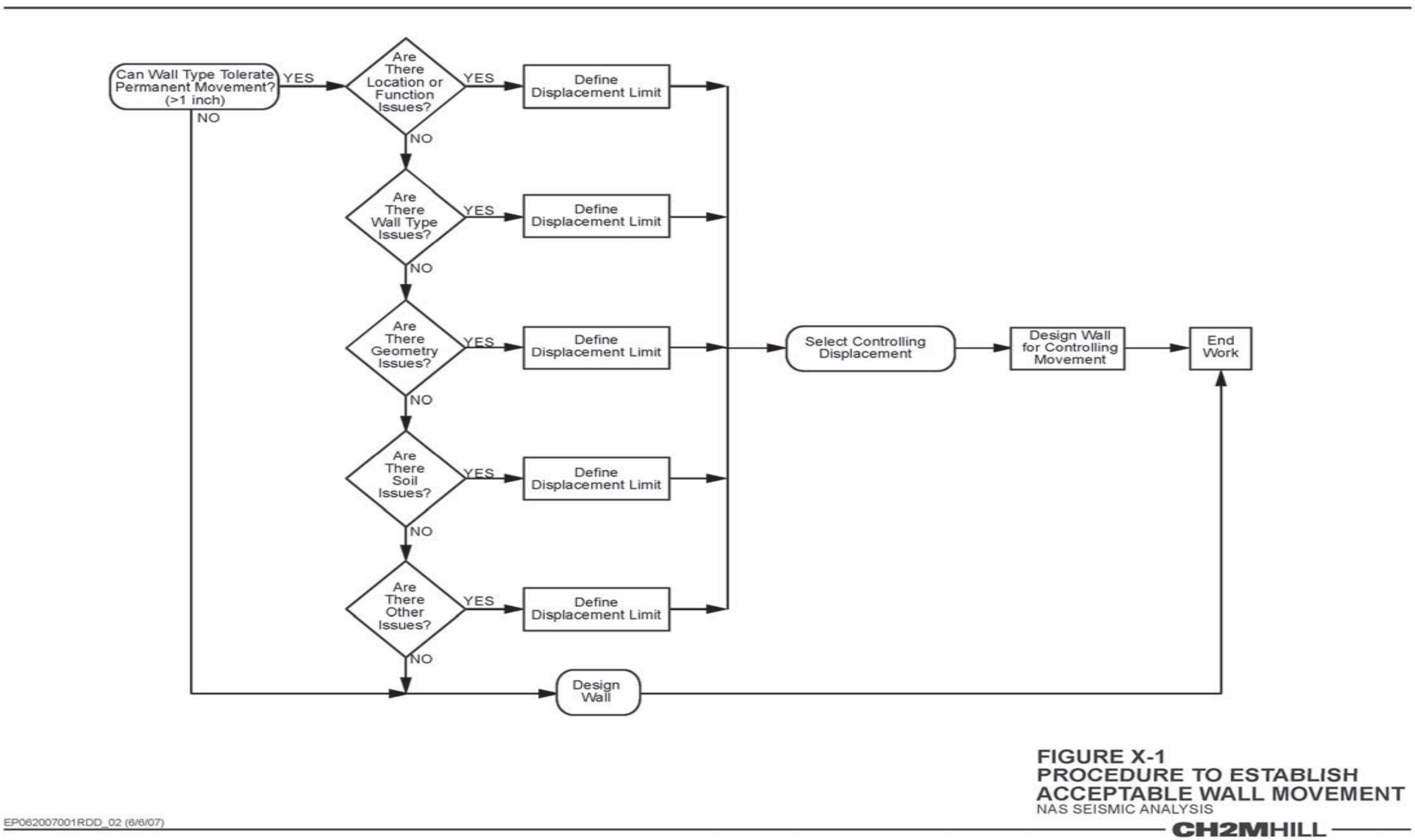


FIGURE X-1
PROCEDURE TO ESTABLISH ACCEPTABLE WALL MOVEMENT

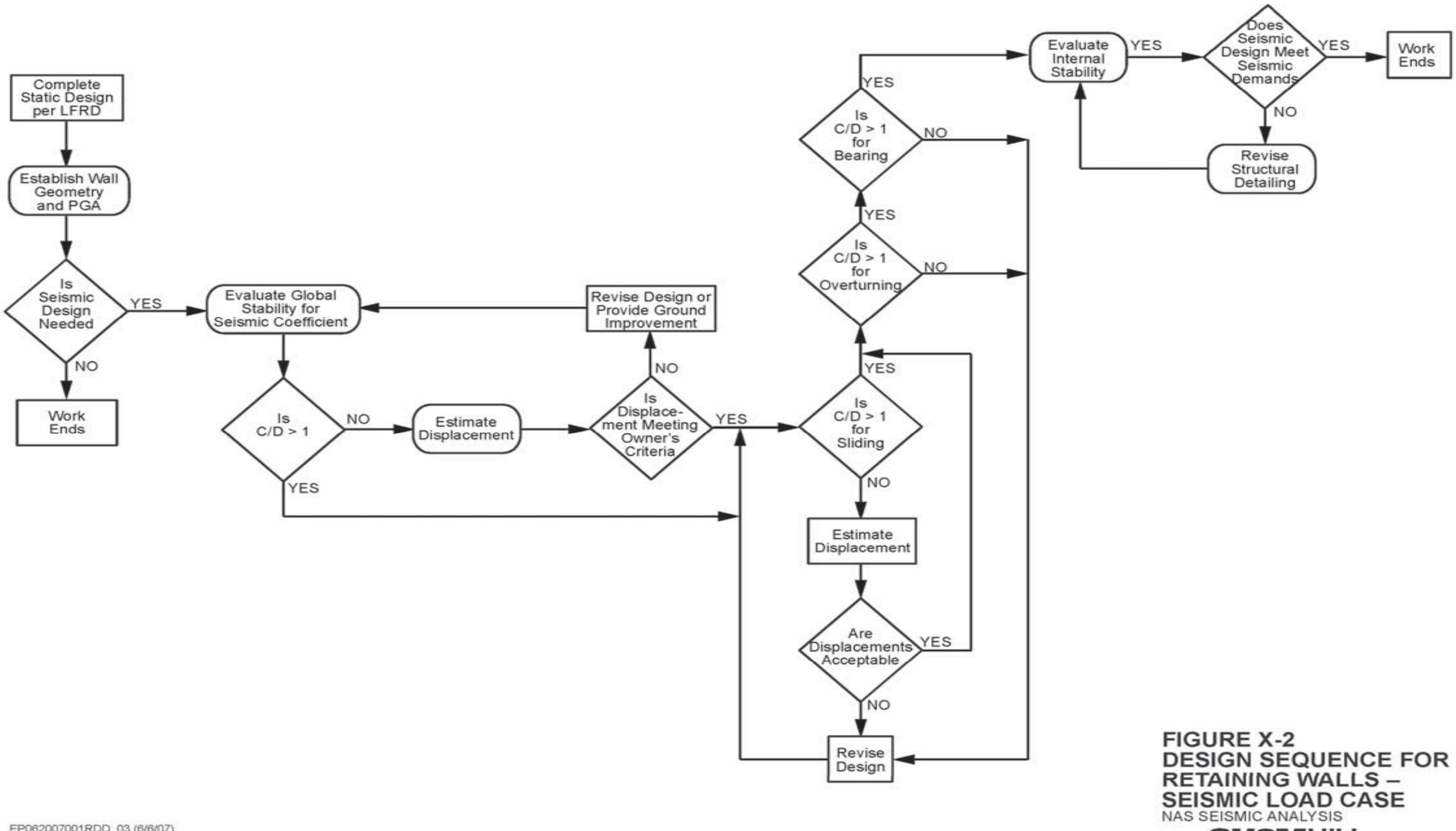


FIGURE X-2
DESIGN SEQUENCE FOR RETAINING WALLS – SEISMIC LOAD CASE
NAS SEISMIC ANALYSIS
CH2MHILL

APPENDIX B_X

CHARTS FOR DETERMINING SEISMIC ACTIVE AND PASSIVE EARTH PRESSURE COEFFICIENTS WITH COHESION

B_X.1 GENERAL

The following charts can be used to estimate the seismic earth pressure coefficient for active and passive loading, if the site is characterized by homogeneous deposits of soil containing a cohesive content. Generally, soils with more than 15% fines content can be assumed to be undrained during seismic loading. For this loading condition, total stress soil parameters, ϕ and c, should be used with these charts.

B_X.1.1 Seismic Active Pressure Coefficient

The M-O equation for seismic active earth pressure determination has many limitations, as discussed in NCHRP (2008). These limitations include the inability to account for cohesion that occurs in the soil or a backfill soil that differs from the native soil conditions. Shamsabadi (2006) has addressed these two primary limitations by re-deriving the seismic active earth pressure using a Coulomb-type wedge analysis.

Equation B_X.1-1 presents the equation developed by Shamsabadi (2006), and Figure B_X.1-1 shows the terms in the equation. This equation is very simple and practical for the design of the retaining walls, and the equation has been calibrated with slope stability computer programs.

$$P_{AE} = \frac{W\left[(1-k_v)\tan(\alpha-\phi)+k_h\right] - \mathrm{C\,L}\left[\sin\alpha\tan(\alpha-\phi)+\cos\alpha\right] - \mathrm{C_A}\,H\left[\tan(\alpha-\phi)\cos\omega+\sin\omega\right]}{\left[1 + \tan(\delta+\omega)\tan(\alpha-\phi)\right]*\cos(\delta+\omega)} \qquad B_X.1\text{-}1$$

The only variable in Equation B_X.1-1 is the failure plane angle α. Values of friction angle (ϕ), seismic horizontal coefficient (k_h), seismic vertical coefficient (k_v), soil cohesion (C), soil wall adhesion (C_a), soil wall friction (δ), and soil wall angle (ω) are defined by the designer on the basis of the site conditions and the AASHTO seismic hazard maps.

The recommended approach in this Section of the Specifications is to assume that $k_v = 0$, and k_h = the PGA adjusted for site effects (i.e., $k_h = k_{max}$ or k_{av} if the wall is greater than 20 ft in height). A 50% reduction in the resulting seismic coefficient is used when defining k_h if 1 to 2 inches of permanent ground deformation is permitted during the design seismic event. Otherwise, the peak ground acceleration coefficient should be used. Equation B_X.1-1 can be easily implemented in spreadsheet. Using a simple spreadsheet, the user can search for the angle α and calculate maximum value of P_{AE}. In particular this equation is very useful for a given angle α as shown in Figure X.7-2. The method is implemented in Caltrans Computer Program CT-Flex (Shamsabadi, 2006).

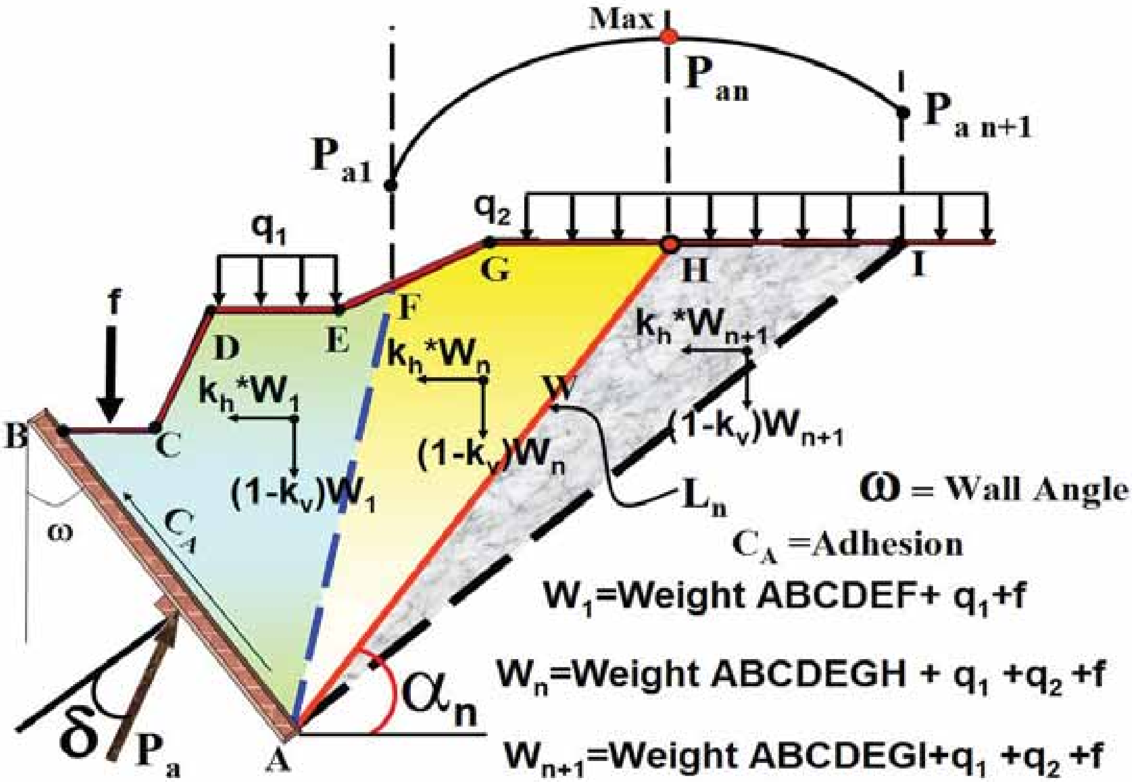


Figure B_X.1-1. Active Seismic Wedge

The following charts were developed using Equation B_X.1-1. These charts are based on level ground behind the all and a wall friction (δ) of 0.67 ϕ. Generally, for active pressure determination the wall interface friction has a minor effect to the seismic pressure coefficient. However, either the generalized limit equilibrium method or the charts can be re-derived for the specific interface wall friction if this effect is of concern or interest.

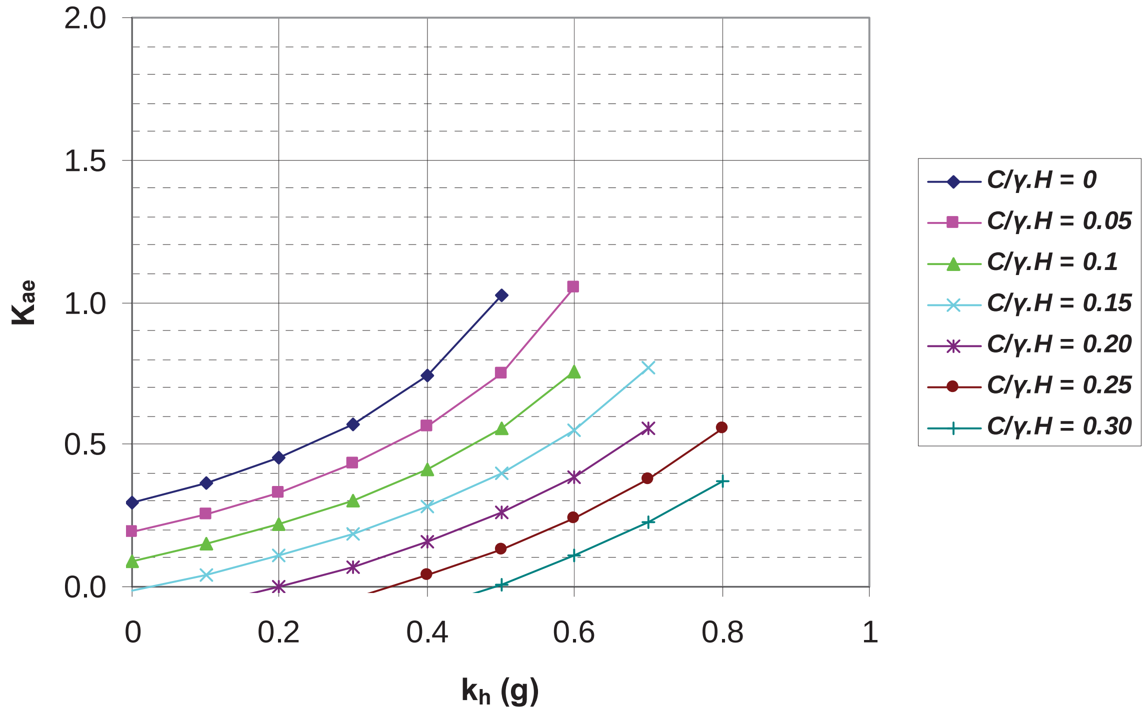


Figure B$_X$.1-2. Seismic Active Earth Pressure Coefficient for ϕ = 30°[2]

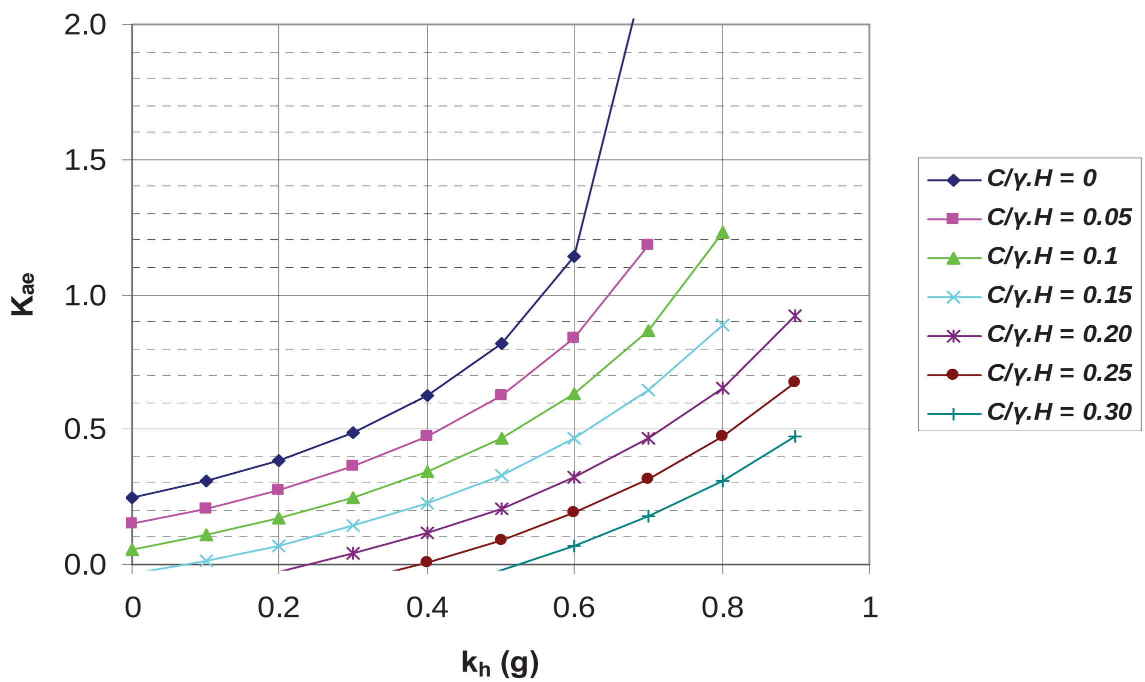


Figure B$_X$.1-3. Seismic Active Earth Pressure Coefficient for ϕ = 35°

[2] **$k_h = k_{max} = k_{av}$ for wall heights greater than 20 ft.**

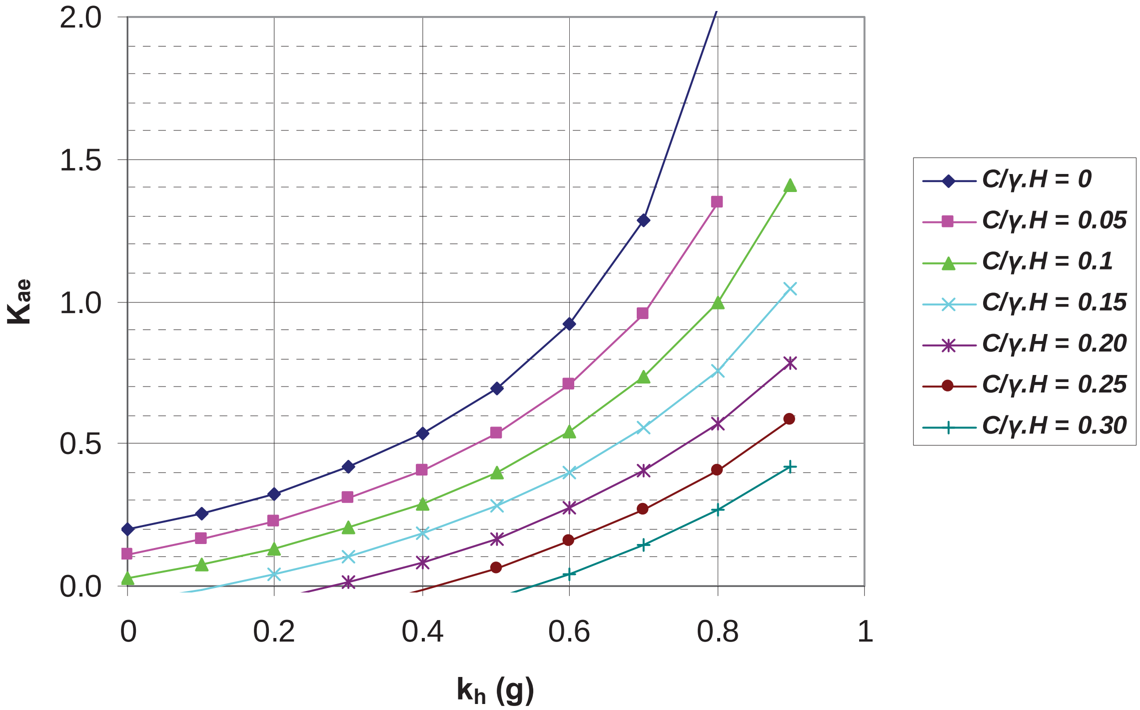


Figure B_X.1-4. Seismic Active Earth Pressure Coefficient for ϕ = 40°

B_X.2 Seismic Passive Pressure Coefficient

This section provides charts for determination of seismic passive earth pressures coefficients for a soil with both cohesion and friction based on the log spiral method. These charts were developed using a pseudo-static equilibrium method developed Shamsabadi (2006). The method includes inertial forces within the soil mass, as well as variable soil surface geometries and loads.

Equations used in Shamsabadi's approach are given below. Figure B_X.2-1 defines the terms used in the equation.

$$dE_i = \frac{W_i(1-K_v)\left[\tan(\alpha_i+\phi)-K_h\right]+CL_i\left[\sin\alpha_i\tan(\alpha_i+\phi)+\cos\alpha_i\right]}{\left[1 - \tan\delta_i\tan(\alpha_i-\phi)\right]*\cos\delta_i} \qquad (B_X.2\text{-}1)$$

$$P_P = \frac{\sum_1^i dE}{\left[1 - \tan\delta_w\tan(\alpha_w-\phi)\right]*\cos\delta_w} \qquad (B_X.2\text{-}2)$$

$$K_P = \frac{2P_P}{\gamma h^2} \qquad (B_X.2\text{-}3)$$

where ϕ is the soil friction angle, c is the cohesion, and δ is wall interface friction.

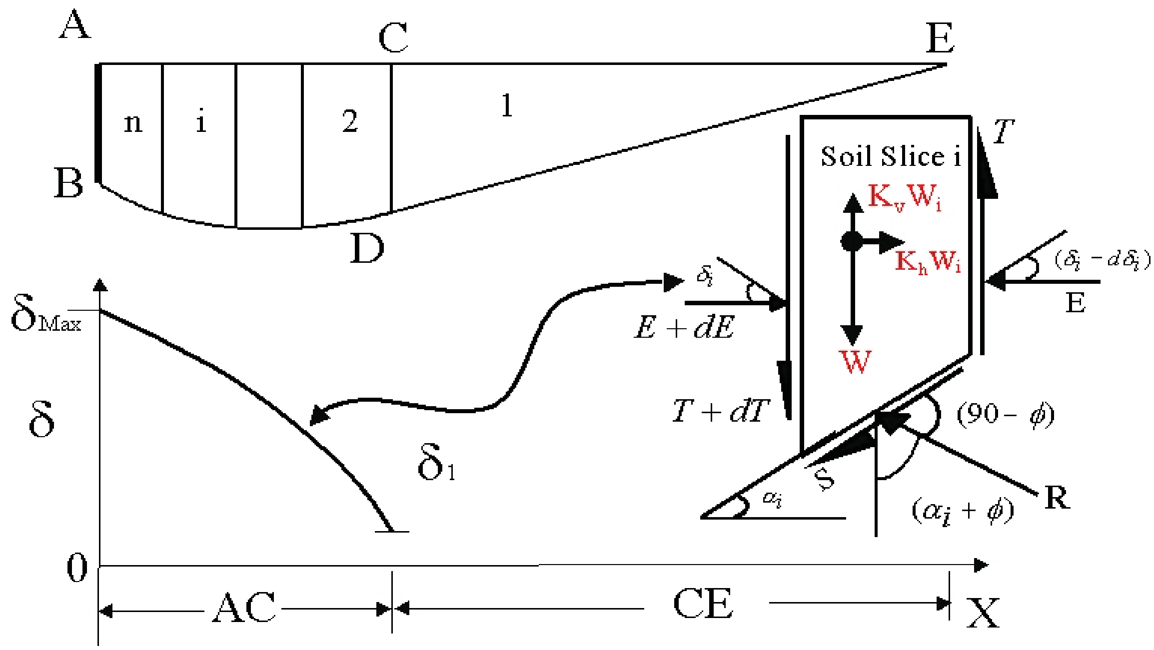
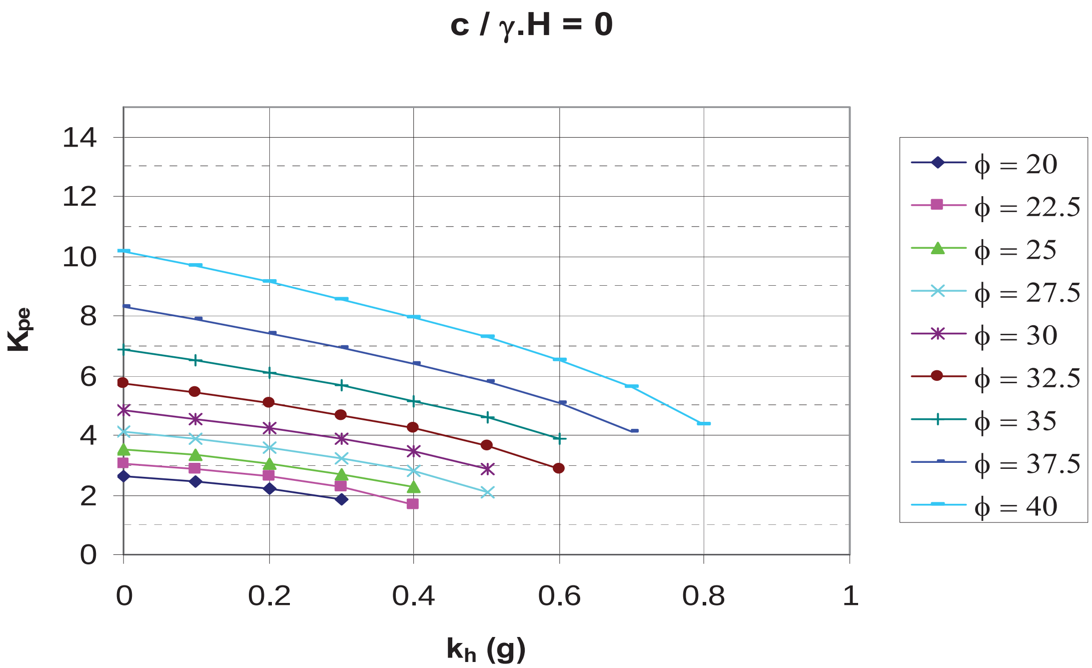


Figure BX.2-1. Limits and shape Seismic Interslice Force Function (Shamsabadi, 2006).

As shown the method of analysis divides the sliding mass of the backfill into many slices. It is assumed that the shear forces dissipate from a maximum at the wall face (AB) to the induced seismic shear forces at the face (CD) of the first slice as seen in Figure B_X.2-1. This methodology is incorporated into the Caltrans computer program CT-Flex (Shamsabadi, 2006).

The methodology described above was used to develop a series of charts (Figures B_X.2-2 through B_X.2-4) for a level backfill condition. These charts can be used to estimate the seismic passive pressure coefficient. The interface friction for these charts is 0.67ϕ. Procedures described by Shamsabadi (2006) can be used to estimate the seismic passive coefficient for other interface conditions and soil geometries.



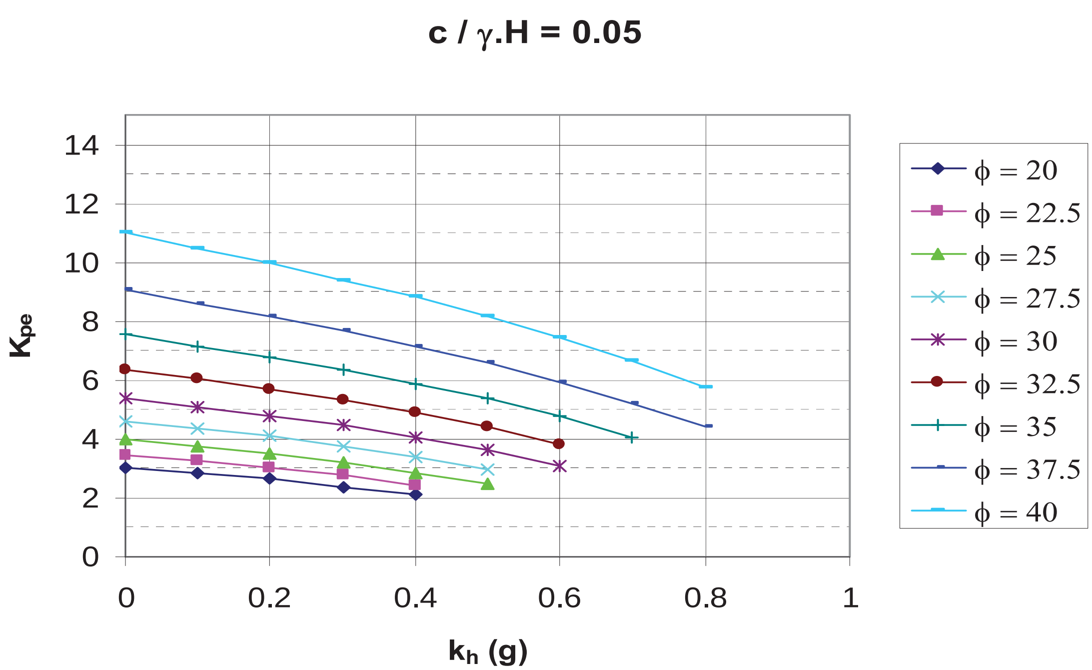
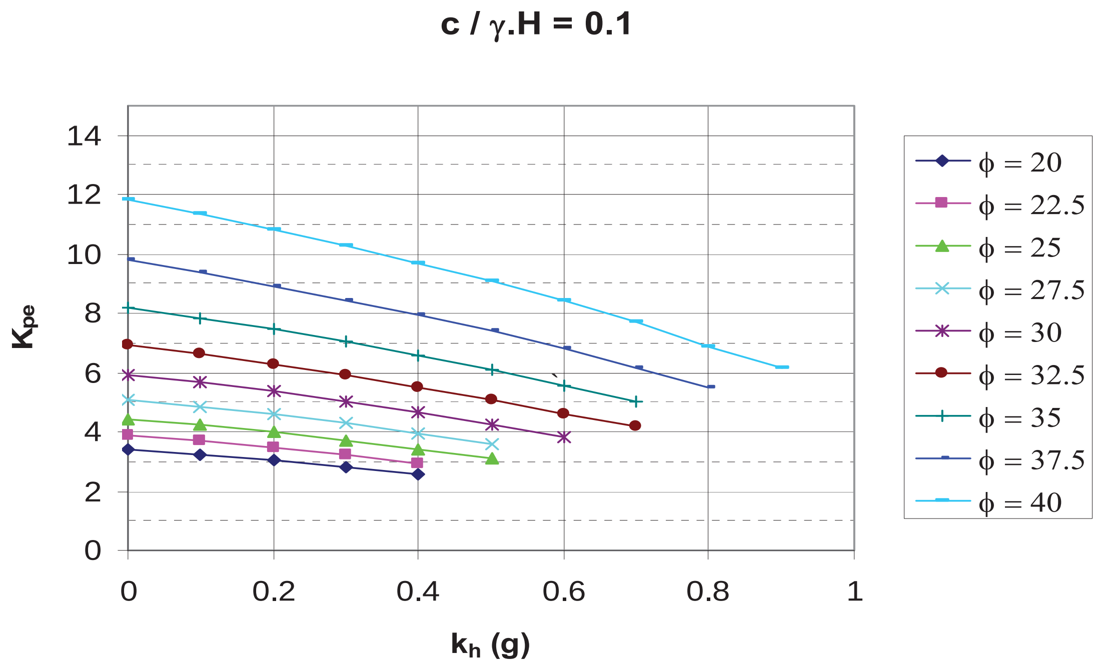


Figure B_X.2-2. Seismic Passive Earth Pressure Coefficient based on Log Spiral Procedure for c/γH = 0 and 0.05 (c = soil cohesion, γ = soil unit weight, and H = retaining wall height).[3]

[3] $k_h = k_{max} = k_{av}$ **for wall heights greater than 20 ft.**



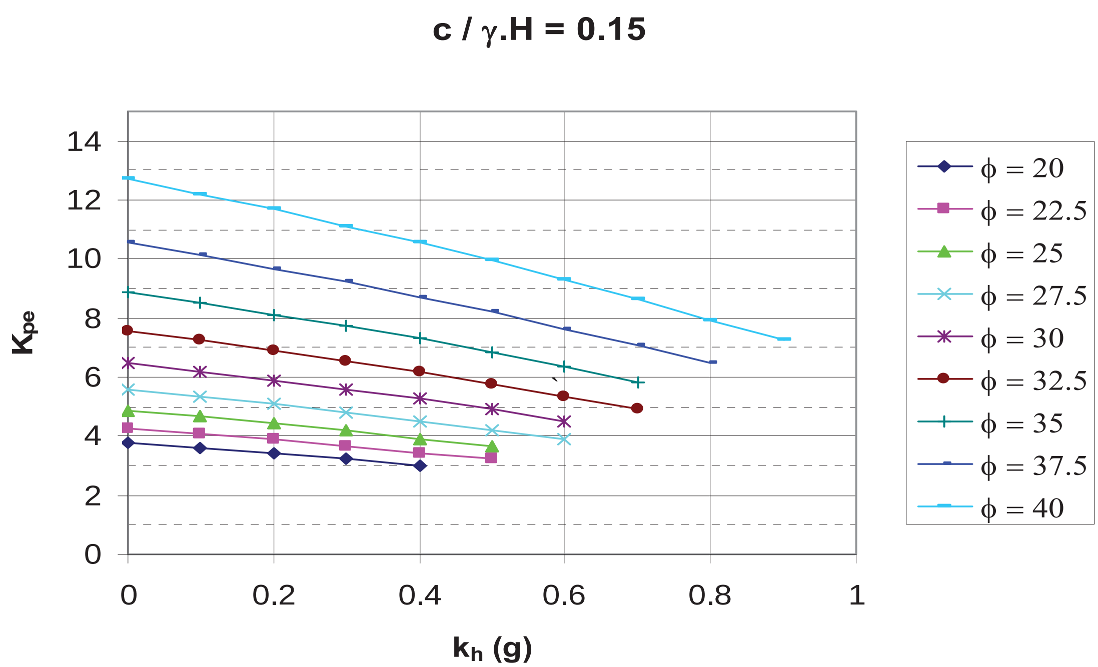
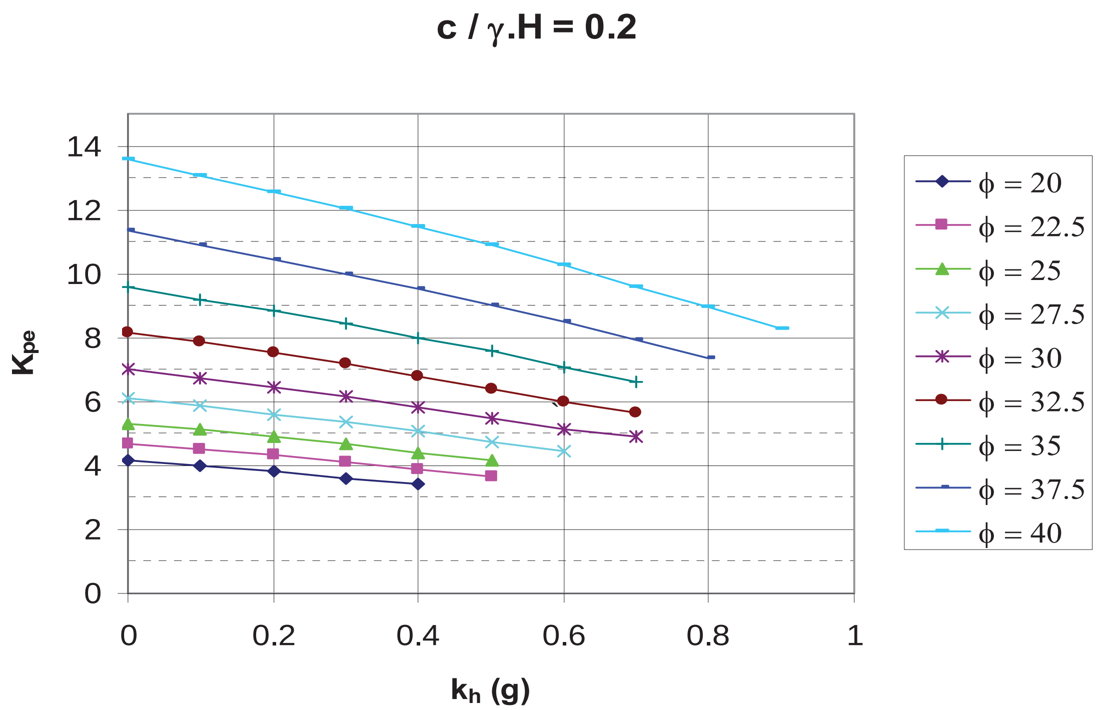


Figure B_X.2-3. Seismic Passive Earth Pressure Coefficient based on Log Spiral Procedure for c/γH = 0.1 and 0.15 (c = soil cohesion, γ = soil unit weight, and H = retaining wall height).



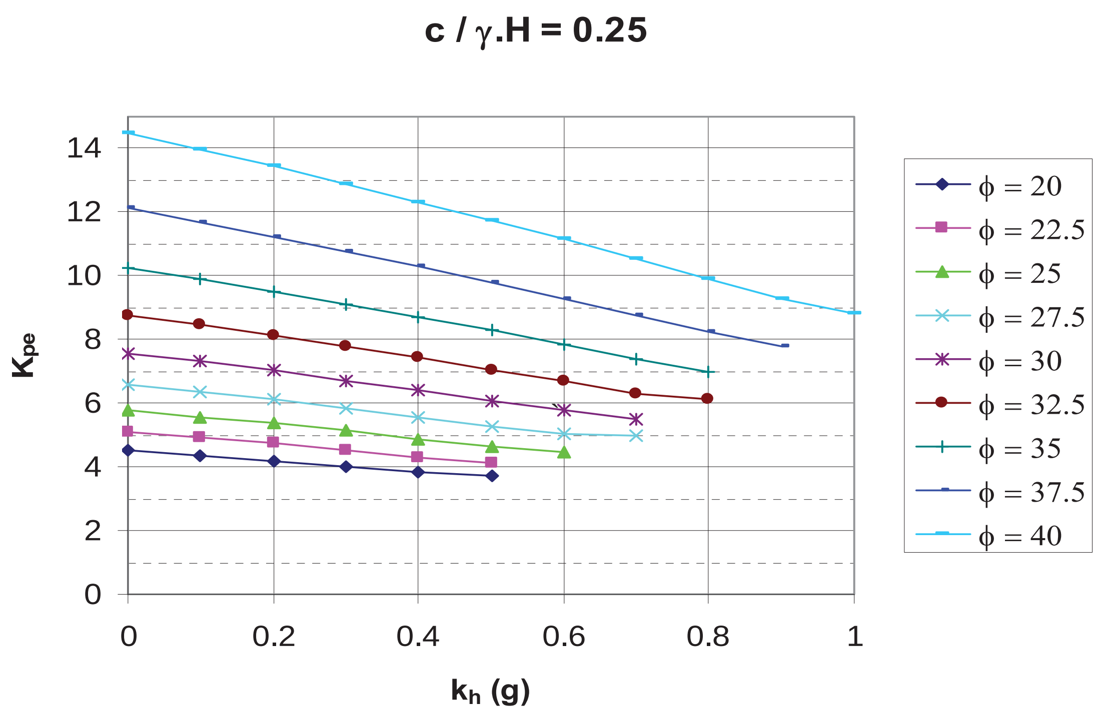


Figure B_X.2-4. Seismic Passive Earth Pressure Coefficient based on Log Spiral Procedure for c/γH = 0.2 and 0.25 (c = soil cohesion, γ = soil unit weight, and H = retaining wall height).

SECTION Y: SLOPES AND EMBANKMENTS

TABLE OF CONTENTS

Y.1 SCOPE

The provisions set forth in this Section shall be used to assess the seismic stability of slopes unless the Owner waives the requirements for this type of assessment.

Prior to evaluating the seismic stability of any natural or embankment slope, the stability of the slope under gravity loads shall be established and shall be shown to have an acceptable ratio of capacity-to-demand (i.e., factor of safety).

Slopes and embankments subjected to seismic loads can represent a hazard to the transportation network if the seismic loads lead to failure of slopes supporting the roadway or slopes located above the roadway. These failures can either be in fills or in hillside cuts. The potential for failure will depend on the geometry of the slope, the properties of the soil making up the slope, groundwater conditions, and the level of earthquake-induced ground shaking for the design seismic event. If the risk of slope failure is high and the consequence of failure is large in terms of public safety or loss of function of the roadway, mitigation of the risk may be required by means of modification to the slope geometry, improvement of the ground or groundwater conditions, or use of structural stabilizing systems.

Some combinations of slope geometry, soil conditions, and groundwater location are very resistant to slope failure even under very high ground shaking levels. In these situations the Owner may choose not to evaluate the seismic stability of the slope. Similarly, the Owner may decide not to mitigate the potential risk from slope failure during the design seismic event if the consequences of failure are small. As a result, an important first step in the seismic analysis of slopes and embankments is to understand the Owner's performance requirements or expectations for the slope or embankment being investigated. Appendix A_Y to this Section provides a methodology that can be used to assess the requirements for seismic slope stability design and mitigation.

Y.2 DEFINITIONS

Cut Slopes—A slope cut into an existing hillside or embankment. By definition the cut slope is always steeper than the existing grade.

Capacity-to-Demand (C/D) Ratio—This term is used to relate the capacity (C) of the soil to resist gravity and earthquake loading to the forces loading the slope (Demand – D). The C/D ratio is equivalent to the factor of safety used during conventional geotechnical design. A C/D ratio less than 1.0 indicates that insufficient soil capacity exists for the loading during the earthquake, while a C/D ratio greater than 1.0 indicates an acceptable condition. Stability is reported in this Section as a C/D ratio rather than a FS to be consistent with terminology used for bridge design.

Engineered Fill—Fill material that has been selected and then placed as part of constructing an embankment. An engineered fill usually is relatively uniform in consistency and is compacted during construction

Fill Slopes—A slope that is formed by filling soil. These slopes are usually constructed of engineered fill; however, they can be loose and represent a natural accumulation of material or material that has been end dumped without control. The fill slope can form the sides of the embankment approach for a bridge.

Y.3 NOTATION

Y.3.1 General

c	= soil cohesion (psf.)
D	= demand (kips or kips/ft.)
F_{pga}	= site factor for PGA (dim.)
F_v	= site factor for spectral acceleration at 1 second (dim.)
g	= gravitational acceleration
k_{av}	= average seismic coefficient after adjustments for wave scattering effects = $\alpha\ k_{max}$ (dim.)
k_{max}	= peak seismic coefficient = F_{pga} PGA = A_s (dim.)
k_y	= yield acceleration coefficient for displacement analysis (dim.)
N_{60}	= Standard Penetration Test (SPT) blowcount adjusted to 60% energy (blows/ft.)
PGA	= peak ground acceleration coefficient on rock (Site Class B) (dim.)
PGV	= peak ground velocity = 55 F_1 S_1/ k_{max} (dim.)
S_1	= spectral acceleration coefficient at 1 second (dim.)
α	= slope height reduction factor (dim.)
γ_p	= load factor (dim.)
ϕ_r	= resistance factor (dim.)
ϕ	= soil friction factor (degrees)

Y.4 SOIL PROPERTIES AND GROUNDWATER CONDITIONS

Y.4.1 General

The types and properties of the soil making up the slope, as well as groundwater conditions

C.Y.4.1

The assessment of subsurface conditions is critical for the evaluation of seismic slope existing in the slope, shall be determined following procedures described in Section 10 of the AASHTO *LRFD Bridge Design Specifications*.

For fill slopes the material existing at the base of the fill and the groundwater conditions below the fill shall also be established following procedures described in Section 10 of the AASHTO *LRFD Bridge Design Specifications*.

stability. Explorations will often be required at the top, middle, and bottom of slopes. Focus needs to be given to low strength or soft soil layers or layers that could liquefy during ground shaking or otherwise soften as repeated cycles of shearing stress occur during the seismic event.

Typically, for embankment slopes explorations should be conducted to a depth of at least the embankment height unless either hard soil or rock layers occur within this depth or available information indicates that deeper soft layers exist. If deeper soft layers exist, the depth of exploration needs to be extended such that the soft layer is adequately characterized for conducting slope stability analyses.

Representative groundwater elevations also need to be established, as the location of groundwater influences the strength of the soil and can determine whether loose, cohesionless soils liquefy.

Y.4.1.1 Soil Strength – Nonliquefiable

Static total stress (undrained) soil strength parameters shall be used for clay soils and for cohesionless soils that contain more than 15% passing the No. 200 sieve, and the static effective stress (drained) soil parameters shall normally be used for relatively clean (e.g., < 15% fines) cohesionless soils during seismic stability analyses.

If the magnitude of the characteristic earthquake is greater than 7.5, the strength of clays shall be multiplied by a factor of 0.9 to account for potential cyclic strength degradation. Best-estimate properties shall be used for all other analyses.

C.Y.4.1.1

The rate of loading during a seismic event is such that most soils will be loaded in an undrained state; i.e., no drainage occurs across the shear plane. Silts and sands with significant content of fine-grained soil (e.g., > 15%) will also behave in an undrained condition. The strength of these soils should be based on the total stress strength parameters, c and ϕ.

Most soils are rate dependent. Under the first cycle of loading, the undrained strength of clay soils will be as much as 40% greater than the “static” strength. However, with repeated cycles of load a reduction in strength occurs due to the effects of pore-water pressure buildup and soil remolding. Research has shown that use of the static total stress strength parameters without modification is an adequate representation of the shearing resistance of the soil, as long as the number of cycles of loading is limited. Where the potential exists for large magnitude earthquakes with many cycles of loading (e.g., M > 7.5 – approximately equivalent to 15 cycles of uniform load), a small reduction in strength is considered appropriate.

A strength reduction factor of 0.9 is suggested for most soils if the magnitude from deaggregation of the design ground motion is 7.5 or greater. A 10 to 15% reduction has commonly been used in practice without regard for the number of cycles of loading. However to avoid introducing excessive conservatism into the stability analysis, the reduction is limited to cases where the equivalent number of cycles is 15 or more, which can be approximately associated with a magnitude 7.5 earthquake.

Certain soils are susceptible to significant strength loss during cyclic loading. Sensitive clays are an example. For these soils it may be necessary to conduct cyclic loading tests on the soil to establish the total stress strength parameters as a function of loading cycles.

Additional guidance on the determination of soil strength for use in seismic slope stability analyses can be found in SCEC (2002).

Y.4.1.2 Soil Strength – Liquefiable

If liquefiable soils are located below or within the slope, special liquefaction studies shall be conducted to determine the potential for liquefaction. If the liquefaction studies determine that liquefaction is likely to occur, the strength of the liquefied soil shall be determined and used in the slope stability analyses.

C.Y.4.1.2

Loose sands, non-plastic silts, and in certain situations gravels located below the water table can lose strength from pore-water pressure buildup during earthquake-induced ground shaking. If the ground shaking is high enough, the soil may liquefy and loose much of its strength. The consequence of this strength loss to an embankment or slope can be large and sometimes very rapid slopes failures. In view of this consequence an assessment of liquefaction potential for a site is a critical step in the seismic slope stability evaluation.

Liquefaction potential can be determined using the generalized method originally published by Seed and Idriss (1983) and updated by Youd et al. (2001). Important updates to the Youd et al (2001) consensus approach for assessing liquefaction potential have been made by Cetin et al. (2004), Moss et al. (2006), and Boulanger and Idriss (2006). These more recent documents should be consulted when performing liquefaction assessments.

Note that these methods for assessing liquefaction potential do not include the effects of the consolidation (static) shear stresses associated with sloping ground. Corrections for the slope ground effects have been developed (e.g., Youd et al., 2001); however, these corrections are not well-established and need to be used with caution.

When the soil liquefies, it is typically characterized by its residual strength. The residual strength has been correlated to Standard Penetration Test (SPT) blowcounts and cone penetrometer test (CPT) end resistance values. Each of these methods has inherent benefits and limitations. Additional information about the determination of liquefaction strength using SPT and CPT procedures is provided in Seed and Harder (1990), Olson and Stark (2002), and Boulanger and Idriss (2006).

Various methods (e.g., Seed and Harder, 1990; Olson and Stark, 2002; and Idriss and Boulanger, 2007) can be used to estimate the undrained strength of soil following liquefaction for large displacements, often referred to as the residual strength of the liquefied soil. For many slopes the reduction in strength without large displacements is needed, particularly at bridge sites on liquefied soils where piles support the bridge abutments. Incremental deformations in these situations based on Newmark analyses may be limited and not sufficient to mobilize the large-deformation, residual strength of the liquefied soil. Little guidance currently exists on estimating the "liquefied strength with limited deformation." One approach (NCHRP, 2003) is to use the residual strength of the liquefied soil in Newmark displacement evaluations, regardless of the deformations that occur. By back-analyzing lateral spread case histories, Olson and Johnson (2007) show that this approach may not be unreasonable

Y.4.1.3 Groundwater Conditions

C.Y.4.1.3

Groundwater conditions shall be determined within the area influencing slope stability. For sites that involve a fluctuating groundwater elevation, such as near rivers and reservoirs where annual water level drawdown occurs or near oceans where tidal changes occur on a daily basis, the time-averaged mean of the long-term groundwater elevation shall be used for seismic slope stability analyses.

The groundwater location within the slope or soil below the slope should be established, particularly if there is a potential for loose cohesionless soils to occur. The determination of groundwater location should consider the long-term as well as daily and seasonal fluctuations in groundwater location. The presence of artesian groundwater conditions within the slope also needs to be established.

In cases where fluctuating groundwater conditions exist, the time-averaged mean level of the long-term elevation is usually suitable for use in design based on the logic that combining an unlikely seismic event and a maximum groundwater elevation represents a very low degree of risk for most locations. If however, the groundwater remains at a high level for a number of months, prudent design is either to use the high level in the seismic slope stability analyses or to check performance under the high level.

Y.5 SEISMIC LOADS AND LOAD FACTORS

Y.5.1 General

Seismic loads in the slope shall be determined using a seismic coefficient estimated in accordance with simplified method summarized in Article X.4 or using numerical modeling methods, subject to the approval of the Owner. A load factor $\gamma_p = 1.0$ shall be used in conjunction with the methodology give in this Section of the Specifications to determine the seismic loads.

The load factor for live load in Extreme Event I (per AASHTO *LRFD Bridge Design Specifications* Section 3) shall be determined on a project-specific basis, except where the slope supports a heavily traveled roadway. For this case live loads shall be included in seismic design, and the load factor (γ_p) for live load shall be at least equal to 0.5.

As in the case of retaining walls (Article X.4.4), where limited (e.g., 1 to 2 inches) permanent displacement of the slope is allowed

C.Y.5.1

The simplified method described in Article X.4 allows determination of the seismic coefficient ($k_{max} = F_{pga}$ PGA) within the slope. Adjustment factors are included within the methodology described in Article X.4 for incorporating permanent soil movement. Effects of wave scattering may also be considered if the slope is greater than 20 feet in height (i.e., $k_{av} = \alpha\ k_{max}$).

The seismic coefficient after adjustment for scattering and permanent soil movement is multiplied by the mass of the soil within the potential failure zone to define the inertial load during seismic loading. Most commercially available slope stability computer programs allow this load and the resulting pseudo-static seismic stability to be determined by specifying the seismic coefficient and allowing the program to search for critical failure surfaces.

Vertical accelerations during ground

by the Owner, a 50% reduction in the maximum seismic coefficient shall be permitted.

For sites that are not susceptible to liquefaction or are not comprised of sensitive soil conditions, a seismic analysis of a cut or fill slope is not required if the site-adjusted peak ground acceleration coefficient (i.e., F_{pga} PGA) at the ground surface for the site is less than the values listed in the following table, unless allowed or required otherwise by the Owner.

Slope Angle	F_{pga} PGA
3H:1V	0.3
2H:1V	0.2

If liquefiable or sensitive soils exist within or support the slope, the minimum acceptable acceleration without requiring a stability analysis shall be 0.15g, as long as the SPT blowcount measured in the field at an energy of 60% (N_{60}) is greater than 5 blows per foot (bpf).

shaking are generally neglected from the seismic stability assessment. The rationale for neglecting the vertical acceleration is that for soils with strengths dominated by friction the cyclic increases and decreases in normal stresses on potential failure planes (and associated increases and decreases in strength) due to vertical acceleration time histories, tend to cancel out the net effects on incremental slope displacements in, for example, a Newmark displacement analysis. In the case of cohesive soils, changes in normal stresses will not affect soil strengths, and hence the vertical accelerations have minimal effect on displacements.

The location of the critical failure surface during seismic loading will usually be flatter than the failure surface determined for gravity loading. Therefore, the computer analyses should be allowed to “search” for the critical surface rather than fixing the failure surface for gravity loading and then applying the seismic coefficient.

When using scattering concepts in Article X.4.3, it is necessary to estimate the height of the slope involved in the wave scattering phenomenon. The height of the slope is defined as the maximum distance between the ground surface and the potential failure surface. As with the design of retaining walls, a scattering factor of 1.0 should be used if the height of the slope is less than 20 feet.

The slope angle used in screening refers to the average angle of the slope above the retaining wall. If the slope is characterized by a non-uniform slope condition, the average angle of the slope should be used. Linear interpolation can be used when determining the need for a seismic analysis for slopes between those given in the table.

For critical slopes the simplified method given in Article X.4 may not adequately model the geometry or soil conditions within the slope. In this case numerical methods involving the use of 2-dimensional finite element or finite difference methods offers an alternative approach for determining the seismic loads in the slope. Because of the stoctastic nature of earthquake ground motions, the earthquake demand for dynamic analyses needs to utilize multiple sets of input records. Current practice is to use either three or seven earthquake records during numerical modeling. If three records are used, the results are enveloped to define the expected response. This approach is generally considered conservative, and the trend has been to conduct analyses for more sets of input motions so that the results are statistically more stable (i.e., achieving a reliable mean and standard deviation). Experience has been that it is necessary to analyze a minimum of seven sets of spectrum-compatible input motions to obtain a statistically stable estimate of response. The response spectra for these records, whether three or seven are used, should be consistent with the design response spectra at rock level.

A cutoff on the lower level of earthquake loading requiring a seismic analysis was set on the basis of the slope angle. For most slopes meeting the static C/D ratio of 1.0 using the static resistance factors of 0.75 or 0.65 as given in Section 11 of the AASHTO *LRFD Bridge Design Specifications* (i.e., FS = 1.3 and 1.5, respectively), the inertial force resulting from k_{max} will still result in a C/D ratio of 1.0 or higher (i.e., FS $\geq$ 1.0). For this condition the slope is predicted to be stable under seismic loading.

If liquefaction is possible at a site because of low SPT blowcount or CPT end resistance values, the no-analysis ground acceleration limit must be reduced to a lower value. As long as the soils are not extremely loose (e.g., SPT blowcount < 5 bpf), liquefaction is very unlikely for peak ground surface acceleration levels of 0.15g or less. For convenience the SPT blowcount for this cutoff is the field value adjusted for 60% energy (i.e., N_{60}).

Y.6 LIMIT STATES AND RESISTANCE FACTORS

Y.6.1 General

Seismic performance of slopes and embankments shall be evaluated in accordance with the requirements of Extreme Event I given in Table 3.4.1 of the Specifications. Except as required otherwise by the Owner, the resistance factor (ϕ_r) during the seismic stability assessment shall be 1.0, except where M > 7.5 as discussed in Article Y4.1.1.

A slope not requiring seismic stability analyses shall demonstrate a capacity-to-demand ratio of greater than 1.0 using resistance factors of 0.75 for natural slopes and 0.65 for engineered slopes (i.e., FS > 1.3 for natural slopes and 1.5 for engineered slopes).

C.Y.6.1

A resistance factor of 1.0 is used in the global stability analysis. While use of a resistance factor of less than 1.0 in limit equilibrium seismic stability analysis will be conservative, for the reasons given in Article X.6 and in view of the unlikely occurrence of the design earthquake, use of a resistance factor of 1.0 is recommended. Lower resistance factors can lead to costly mitigation procedures that have a low likelihood of being needed.

As discussed in Article Y.4.1.1, a reduction in strength using a factor of 0.9 is recommended in the stability analyses if M > 7.5. This reduction is included to account for potential cyclic degradation in strength and is not simply introduced to be conservative.

The use of a resistance factor of 1.0 is particularly critical for displacement-based design methods. If a resistance factor is introduced for displacement-based analyses, estimates of displacements will normally be too high, and therefore, potentially lead to unrealistic decisions regarding the need for modifications to the structure or ground to resist these movements.

Y.7 METHODS OF ANALYSIS

Y.7.1 General

The stability of the slope shall be evaluated using either (1) the seismic coefficient approach in a pseudo-static stability analysis or (2) a slope-displacement method. The selection between the two approaches shall be made on the basis of the complexity of the slope geometry and soil conditions within the slope, the level of ground shaking, and the potential consequences of failure of the slope. Generally, the approach planned by the designer should be discussed with the Owner before the method of analysis is selected.

For the seismic coefficient method, the C/D

C.Y.7.1

The seismic coefficient method involves the use of the limit equilibrium method to compare the resistance that is mobilized by the soil to the demand caused by the combination of gravity loads and the earthquake inertial loads within the slope. If the ratio of capacity-to-demand is less than 1.0 (i.e., earthquake-induced loads exceed the capacity of the soil), slope movement is predicted to occur. The seismic coefficient method does not explicitly quantify the amount of movement; however, the value of the seismic coefficient may be chosen such that it implicitly accounts for the ratio shall be 1.0 or higher. For the displacement method, the estimated displacement shall be less than the performance-based displacement specified or agreed to by the Owner.

amount of movement considered acceptable. For example, by using a seismic coefficient in the stability analysis of 50% of k_{max}, movement of 1 to 2 inches is implied.

The alternative approach is to compute the deformation that results from seismic loading. Several alternatives are available for making the displacement evaluation: (1) simplified charts based on the Newmark method, (2) the Newmark earthquake record integration method, or (3) time-history numerical modeling. While the displacement approach involves more engineering time, the results allow the designer and Owner to judge the potential consequences of slope instability and whether mitigation of the expected condition should be considered.

Y.7.2 Seismic Coefficient Approach

Stability analyses conducted using the seismic coefficient approach shall show that the C/D ratio is greater than 1.0 (i.e., FS > 1.0) under the peak seismic coefficient determined in accordance with Article X.4, where a displacement-based reduction of 50% in k_{max} is permitted.

The ratio of demand to capacity shall be obtained by using a slope stability computer program capable of modeling the variations in slope geometry, soil properties, and groundwater conditions established for the slope. The vertical component of ground shaking shall not be incorporated in the analyses.

C.Y.7.2

The seismic coefficient approach involves introducing a seismic coefficient into a conventional slope stability analysis, and determining the resulting factor of safety. If the seismic coefficient from Article X.4 is used in the analyses and the slope is determined to have a capacity greater than the seismic demand (i.e., FS > 1.0), the slope is considered stable during seismic loading. In this approach the seismic coefficient can be adjusted for wave scattering if the slope height is greater than 20 feet. A 50% reduction in the seismic coefficient is also allowed if the slope can deform 1 to 2 inches, which is usually the case.

Typically, total stress (undrained) strength parameters are used for cohesive soils and effective stress parameters (drained) are used for clean granular soils in the pseudo-static seismic analysis, as long as strength loss is not expected during earthquake loading. If the design seismic event is expected to have magnitude greater than 7.5, the strength used in the seismic stability analysis should be taken as 90% of the static strength to account for the effects of repeated cycles of load, as noted in Article Y.$.1.1. For cases where significant strength loss could occur, such as where liquefiable soils exist, alternate methods that account for the strength loss must be used to estimate the strength during the design seismic event.

A large number of commercially available computer programs exist that can perform both static and pseudo-static limit equilibrium analyses. Most of these programs provide general solutions to slope stability problems with provisions for using the simplified Bishop, simplified Janbu, and/or Spencer's method of slices. Potential sliding surfaces, both circular and polygonal, can usually be pre-specified or randomly generated. Commonly used programs include PCSTABL5 (developed at Purdue University), UTEXAS3 (developed at the University of Texas at Austin), SLOPEW (distributed by Geo-Slope International), and SLIDE (RocScience).

One of the drawbacks in the "seismic coefficient – factor of safety" approach lies in the difficulty of directly relating the value of the seismic coefficient to the characteristics of the design earthquake and slope performance. Use of either the peak ground surface acceleration or the peak average horizontal acceleration over the failure mass in conjunction with a pseudo-static factor of safety of 1.0 may give excessively conservative assessments of slope performance in earthquakes. However, often little guidance on selection of the seismic coefficient as a fraction of the peak ground surface acceleration is available to the designer.

Conventional practice over the last decade or more has been to use a seismic coefficient that is some percentage of the peak ground surface acceleration (k_{max}) occurring at a site. The value can range from less than 50% of the peak k_{max}, depending on the designer's views or the Owner's requirements. If a seismic coefficient of less than the peak is used, the slope is expected to deform during seismic loading. The amount of deformation can be estimated following the procedures in the next article of the Specifications. Generally, if the seismic coefficient used in design is 50% of the peak ground surface acceleration, the amount of permanent slope displacement will be less than a few inches.

The published guidelines by SCEC (2002) for the State of California suggests reducing peak ground acceleration map values in California after modifications for height effects by factors ranging from about 0.3 to 0.6 (depending on earthquake magnitude and peak ground acceleration values) to ensure slope displacements are less than about 6 inches. The 6-inch displacement is a screening value suggested as a potential criterion to determine if a Newmark approach is necessary. This Section of the Specifications recommends a 50% reduction consistent with the discussions in Article X.4 and reflecting a potential displacement of a few inches.

Y.7.3 Simplified Newmark Displacement Method

Where required by the Owner or where the resulting C/D ratio in a Seismic Coefficient Approach is less than 1.0, displacements shall be estimated using the Newmark displacement equations in Article X.4.5. The acceptability of the estimated displacements shall be reviewed with the Owner to determine if the levels of displacement are within performance expectations.

C.Y.7.3

In contrast to the “seismic coefficient” approach, the Newmark sliding block (or more appropriately called permanent seismic deformation) approach involves the explicit calculation of cumulative seismic deformation. The potential failure mass is treated as a rigid body on a yielding base. The acceleration time history of the rigid body is assumed to correspond to the average acceleration time history of the failure mass. Deformation accumulates when the rigid body acceleration exceeds the yield acceleration of the failure mass (k_y) where k_y is defined as the horizontal acceleration that results in a factor of safety of 1.0 in a pseudo-static limit equilibrium analysis.

The following simplified Newmark-type methodology is recommended for slopes and embankments, where the static strength parameters can reasonably be assumed for seismic analyses:

- Conduct static slope stability analyses using appropriate load and resistance factors to confirm that performance meets static loading requirements.

- Establish the k_{max}, spectral acceleration at one second (F_v S_1) from the AASHTO maps (including appropriate site soil modification factors), and earthquake magnitude. Determine the corresponding peak ground velocity (PGV) from correlation between F_vS_1 and PGV provided in Article X.4.

- Modify k_{max} to account for slope or embankment height effects. As discussed in the NCHRP 12-70 Report (NCHRP, 2008), the α factor procedure described in Article X.4 appears compatible with procedures developed by other investigators for slope-height effects, and for this reason, it is recommended for use in determining the effective seismic coefficient for the design of slopes (i.e., $k_{av} = \alpha\ k_{max}$). This approach gives conservative α values when compared to scattering analyses performed for a slope as described in Article X.4 and is also consistent with height reduction factors documented in SCEC (2002). For slopes where the critical sliding surface under seismic loading occurs over zones less than the full slope height, the height of the slide zone should be used to determine α.

- Determine the yield acceleration (k_y) using a pseudo-static stability analysis for the slope (i.e., the seismic coefficient corresponding to a factor of safety equal to 1.0). If $M > 7.5$, use 0.9 times the strength to account for potential effects of repeated load cycles.

- Establish the earthquake slope displacement potential corresponding to the value of k_y/k_{max} using the displacement equations in Article X.4. When using the equations, $k_{av} = \alpha\ k_{max}$ for slopes greater than 20 feet in height.

- Evaluate the acceptability of the displacement based on performance criteria established by the Owner for the specific project site.

Y.7.4 Time History Displacement Method

If displacements are estimated by the Newmark time history method or using 2-dimensional finite difference or finite element computer programs, a set of three to seven earthquake records shall be used in the analyses. The earthquake records shall be selected to be generally consistent with the predicted spectral acceleration at 1 second on rock or an overall fit to the design response spectrum developed on rock for the site.

C.Y.7.4

In some situations the simplified Newmark chart method of estimating displacements will be inadequate, either because the geometry is too complex to be represented by a sliding block or because soil conditions preclude use of the simple model. In these situations either the Newmark time history approach or more rigorous 2-dimensional numerical methods should be used.

In either the Newmark time history approach or the numerical modeling approach, appropriate earthquake records have to be selected. Various electronic databases are available with these records (e.g., Pacific Earthquake Engineering Research Center database). To the extent possible, records should be selected that generally match the earthquake source type, earthquake source distance, peak ground acceleration, and earthquake magnitude considered to be appropriate for the site. Recent publications provide additional guidance on the selection of suites of earthquake records from the available databases (NCHRP, 2003; NEHRP, 2006). The Newmark time history approach requires determination of the yield acceleration for the slope. The yield acceleration (k_y) is defined as the seismic coefficient that results in a factor of safety of 1.0. A trial-and-error method is used to determine the yield acceleration. Consideration can be given to modifying the yield acceleration at some point in the analysis (i.e., time) to account for loss in soil strength due to repeated cycles of loading.

Two-dimensional computer codes are now commonly used to evaluate the seismic performance of slopes. Two software packages are used extensively for this type of analyses, FLAC (Itasca, 2007) and PLAXIS (Plaxis BV, 2007). Both programs allow modeling of the soil stiffness and strength, and porewater pressure effects on soil strength. Structural elements within the soil profile, such as foundation piles and tie-back anchors, can also be included in the model. These capabilities are especially valuable if it is necessary to mitigate slope instabilities. Considerable experience is required to use these programs. Before adopting this approach, discussions should be held with the Owner to review uncertainties and limitations. Reality checks should also be conducted with the simplified chart method to confirm that the detailed analysis is reasonable.

Y.8 STABILITY ASSESSMENT INVOLVING LIQUEFACTION

Y.8.1 General

If liquefiable soils are predicted to occur within or below the slope or embankment, the potential for slope instability during liquefaction shall be evaluated using either a limit equilibrium or displacement-based approach:

- **Limit-Equilibrium Analysis:** For a limit equilibrium analysis the C/D ratio shall be determined using the strength of the liquefied soil in the slope stability analysis. If the C/D ratio for the liquefied case is greater than 1.0 (i.e., FS > 1.), the site is considered stable during the design seismic event. If the C/D ratio is less than 1.0 (i.e., FS < 1.0), the seismic-induced displacements shall be estimated using a displacement-based approach.

- **Displacement-Based Analysis:** For the displacement-based approach, permanent displacements shall be estimated using one of the following methods: (1) the simplified Newmark equations (Article Y.7.3), (2) the time history displacement method (Article Y.7.4), or (3) numerical modeling with a 2-dimensional computer code. For the displacement analysis, reductions in soil strength due to pore-water pressure buildup associated with liquefaction shall be accounted for in the displacement

C.Y.8.1

The limit equilibrium approach for assessing the performance of slopes during liquefaction is the same as for non-liquefiable slope, except that the soil strength for static loading is replaced by the residual strength of the soil As noted in Article C.Y.4.1.2, considerable uncertainty exists regarding the appropriate strength to use for liquefied soils that are not undergoing large deformation or flow in displacement-based evaluations. One option is to conduct laboratory cyclic tests to estimate soil response under the anticipated seismic stress path. Another alternative, which is thought to be conservative, is to use the residual strength of liquefied derived from back analysis of flow failures.

The seismic coefficient used in this analysis is the same as for the nonliquefiable case. No reductions or modifications in seismic coefficient are made to account for the modifications of ground motions from liquefaction. This assumption is usually conservative. Nonlinear effective stress analyses and field studies (e.g., Youd and Carter, 2005) usually show that the peak ground acceleration above the liquefied zone will be decreased; however, the amount of reduction depends on the characteristics of the site and the seismic design event. For conservatism the recommendation is to use the peak ground surface acceleration with determination.

Predicted demand to capacity ratios or deformation shall be reviewed with the Owner, and a decision shall then be made on whether ground improvement methods are required to limit flows or lateral spreading movements.

adjustments for wave scattering and permanent movement (i.e., 50% reduction if 1 to 2 inches of displacement are acceptable) but without an reduction for liquefaction.

Three approaches are currently being used or proposed for evaluating slope displacements where liquefaction is involved:

- **Youd Empirical Method:** The simplest are the empirical relationships, such as suggested by Youd et al. (2002), for estimating displacement during lateral spreading. These relationships are based on empirical correlations between observed lateral displacement, earthquake parameters, and soil conditions. This approach is typically applied near rivers or other locations where slopes are gentle and a free face might exist. Generally results from these methods are considered more suitable for early screening of potential displacement issues and involve too much uncertainty for site-specific design.

- **Two-Step Method:** The second approach involves used of the simplified Newmark equations in a two-step analysis. This approach is based on a Newmark sliding block approach. A pseudo-residual strength is assigned to the liquefied layer using empirical relationships (Seed and Harder, 1990; Olson and Stark, 2002; or Idriss and Boulanger, 2007) for flow failures. The first step involves determination of stability after the end of shaking using the residual strength of layers that have liquefied. Under this condition a seismic coefficient is not applied. If the ratio of capacity-to-demand is less than 1.0, a flow failure is predicted. In this case very large deformations are predicted to occur. An approximate estimate of the magnitude can be made using the Youd et al. (2002) empirical method. If the capacity-to-demand ratio is greater than 1.0, the stability analysis is repeated using the residual strength of the soil and also imposing the seismic coefficient. The yield acceleration is determined, and deformations estimated in accordance with procedures recommended in Article Y.7.3.

- **Numerical Modeling Method:** The third method involves the use of numerical modeling methods. Various computer programs, such as FLAC and PLAXIS, are commonly used to investigate the seismic stability problem where liquefiable soils have been identified. These methods seem to be used extensively by designers – often without having a particularly good understanding or appreciation for the uncertainties of the model. One of the significant criticisms of this approach is that thin layers that lead to ground displacement during liquefaction are not correctly modeled.

Various approaches for dealing with liquefaction-related slope instability will continue to be identified as future research studies are conducted. Unfortunately, there is no current consensus within the profession on the best approach for dealing with liquefaction-related slope stability – each has its pros and cons. The current difficulty in developing a consensus results from uncertainties in two areas: (1) the capacity of the soil in its liquefied state, particularly where there are static shearing stresses (i.e., sloping ground effects) and dilation effects during cyclic loading, and (2) the ground motions to use after the seismic wave travels through the liquefied soil. While numerical methods, such as DESRA (1978), are available to address the latter issue, these methods are limited in availability to most designers.

For many sites the two-step Newmark method identified above can be used. This approach represents a relatively simple method that allows “order of magnitude” displacements to be estimated. While this approach is relatively simple to apply, it is often criticized as it does not address the complexity of the triggering relationship for liquefaction on sloping ground, and it does not properly account for the overall complexity of the problem, particularly the appropriate for liquefied soils undergoing limited deformation. Results of centrifuge research programs also indicate that the methodology may not replicate important mechanisms that occur during seismic loading. Many of these issues are being studied in research being conducted by the Pacific Earthquake Engineering (PEER) Center. Until a consensus is reached on a better simplified method of analysis, the two-step method will be sufficient.

Y.9 GROUND IMPROVEMENT

Y.9.1 General

Subject to the Owner's concurrence, methods of mitigating unacceptable slope performance shall be investigated. Methods of mitigation shall consider the likely depth of ground failure associated with slope instability, the plan area requiring stabilization, the time and expense of the mitigation methods, and environmental effects of mitigation, including the temporary effects of implementing the mitigation.

C.Y.9.1

Mitigation methods can range from regrading the slope which lessens the slope angle to use of in-place soil mixing to improve the soil strength. Mitchell et al. (1998) provide a summary of methods that can be used for improving the stability of slopes. Some of the more common methods used for slope stability mitigation are listed below.

Locations without Liquefaction

- Regrading slope, use of drainage systems, and adding reaction berms (compacted soil buttress structures at toe of slope)
- Stone columns and piles
- Retaining walls and ground anchors

Locations with Liquefaction

- Vibro densification and deep dynamic compaction
- Stone columns and in-place deep soil mixing
- Drainage blankets and columns

The selection amongst the various alternatives requires detailed study and requires a clear understanding of the performance objectives of the improved ground.

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APPENDIX A_Y

STRATEGY FOR OWNER DECISION-MAKING ON ACCEPTABLE DISPLACEMENTS FOR SLOPES AND EMBANKMENTS

This appendix provides a strategy for Owners to use when determining the amount of permanent displacements that is acceptable for slopes and embankments. This strategy is to be used with Section Y of the Specifications and Commentaries prepared as part of the NCHRP 12-70 Project *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments.*

Background

The Specifications and Commentaries prepared for the NCHRP 12-70 Project include provisions that allow for permanent displacements of slopes and embankments. The permanent movement is associated with global stability, where the entire soil mass moves. In many situations some small-to-moderate amount of permanent displacement is acceptable after the design seismic event. In these cases the slope or embankment can be repaired by removing or placing earth. However, in locations where slope or embankment movement affects nearby structures or presents a risk to the public, anything more than a few inches of movements may not be acceptable.

If permanent movement of the slope is acceptable, there are significant benefits to the Owner:

- The Specifications and Commentaries allow reduction in the seismic coefficient used in design by 50% if 1 to 2 inches of permanent movement are acceptable. This reduction in the seismic coefficient is similar to procedure that appears in Section X of these Specifications and in the current AASHTO *LRFD Bridge Design Specifications* for the design of retaining walls. In many cases the slopes or embankments can undergo more than several inches of displacement and not affect the function of the adjacent roadway. In these cases a reduction in the seismic coefficient of greater than 50% may be acceptable.
- The NCHRP 12-70 Specifications and Commentaries provide a methodology for estimating the amount of deformation for Owners who cannot or do not want to use the 50% reduction. This methodology is useful in situations where larger deformations than associated with the 50% reduction in seismic coefficient are acceptable (e.g., more than a few inches) and where the capacity-to-demand (C/D) ratio does not meet the target of 1.0 with the 50% reduction. Rather than regrading the slope or introducing expensive mitigation measures to achieve the C/D of 1.0 or more, the amount of permanent displacement can be calculated. If the amount of displacement is within reasonable limits, the seismic design of the retaining wall may be acceptable even though the C/D ratio is less than desired.

An over-riding question in any approach that involves permanent deformations is the amount that is acceptable. The Specifications and Commentaries to the NCHRP 12-70 Project leave this decision to the Owner, who must weigh a number of factors in reaching this decision. Typically, a few inches of movement are acceptable; however, there are situations where even this level of deformation may be unacceptable. On the other extreme, some slopes and embankments may be able to tolerate several feet or more of movement, particularly if the movement is in a rural area next to a little used roadway. A number of factors should be considered by the Owner when deciding the acceptable amount of permanent displacement as summarized below.

Considerations for Establishing Acceptable Displacements

The factors that should be considered when deciding on acceptable levels of permanent displacement range from implications of the movement to likely mode of slope movement. When considering these factors, the Owner should evaluate both the relative consequences of movement and, as appropriate, the cost of designing to avoid the movement.

Slope Location and Function

One of the main factors for deciding on the acceptable level of movement involves the location and function of the slope.

- Slopes located in urban locations usually can tolerate less movement than slopes located in the countryside. Part of this relates to effects of slope movement on utilities and other nearby facilities, and part relates to aesthetics. After a design seismic event a slope that has moved 24 inches or more in the countryside may be completely functional and acceptable, but this same slope may not be accepted in an urban environment.
- Slopes that support a heavily traveled roadway should usually be designed for smaller displacements than slopes that are part of a less traveled roadway. This relates to loss of function if there is damage associated with slope movement. Generally, less traveled roadways can remain unusable for a longer period of time, and therefore, large amounts of damage from permanent movement are acceptable. On the other hand, roadways with heavy use will result in significant traffic and economic disruption if they are out of service for even a few hours. For this situation it may be very important to limit displacement to levels that will have minimal disruption to service.
- Slopes that pose a large risk to public safety should be designed for less movement than slopes that represent low risk. Generally, the risk to the public increases as the amount of soil movement increases. The volume and rate of movement can also become considerations in this assessment. If there is a large risk associated with soil-mass movement, then the Owner is obligated to take a more conservative approach to design, which often will mean minimizing acceptable movements.

Types of Soil

The type of soil at a site also should be considered when establishing displacement limits. This consideration is related to both the type of failure mechanism and the response of the slope to loads.

- Seismic-induced slope failures in some slopes will be primarily surface sloughing, while other slopes undergo deep rotational failures. The former type of failure often involves simply a maintenance cleanup, while the latter can involve a significant rebuilding effort.

The form of the failure usually is controlled by the types of soil making up the slope – with granular soils involving more surficial failures while cohesive soils involve deeper rotational failures.

- Displacement of slopes that are characterized by brittle soils or soils that decrease in strength with deformation will have a lower reliability than soils that do not soften with strain. It is usually very hard to estimate how much cyclic strain will result in strength softening, making the prediction of deformations difficult.
- Slopes constructed with some cohesive content or formed of cohesive soil have an inherent level of conservatism incorporated in the design, even when following the generalized limit equilibrium methods described in the Specifications and Commentaries. This conservatism will generally lead to smaller deformations during the seismic even than are being predicted.
- The confidence in displacement predictions for liquefiable soils is often relatively low. If liquefaction is predicted at a slope location, it is generally better to mitigate the liquefaction condition. While it is possible to make estimates of slope displacement using residual strengths, the possibility of performance being different than expected increases for this situation.

Implications of Slope Movement

Perhaps the easiest consideration to understand is the effects that slope movement will have on other facilities in proximity to the slope. Examples of these effects are summarized below.

- Utilities, sidewalks, and pavements located in front of, within, or behind the slope could be affected by permanent movement of the soil. The amount of displacement of the utility, sidewalk, or pavement can be approximated by the amount of permanent displacement being estimated.
- Slope aesthetics are also affected by permanent displacements. As noted previously. Generally, as the amount of movement increases, the amount of distortion becomes more noticeable to the public. The slope movement can also alter drainage within the slope and vegetation growing on the slope.

Approach for Defining Acceptable Displacements

As summarized above, many factors must be considered when deciding on the acceptable level of displacement for slopes and embankments. These factors make the development of a simple strategy for establishing the permanent displacement difficult. As soon as displacements of more than 1 to 2 inches are being considered, the Owner should perform a rigorous review of the possible consequences of movement to the wall and to facilities located in proximity to the slope and embankments.

Figure A_Y-1 shows the steps that the Owner might use in conjunction with the Specifications and Commentaries to define an acceptable limit to permanent displacement. Figure A_Y-2 shows the overall design process for slopes and embankments.

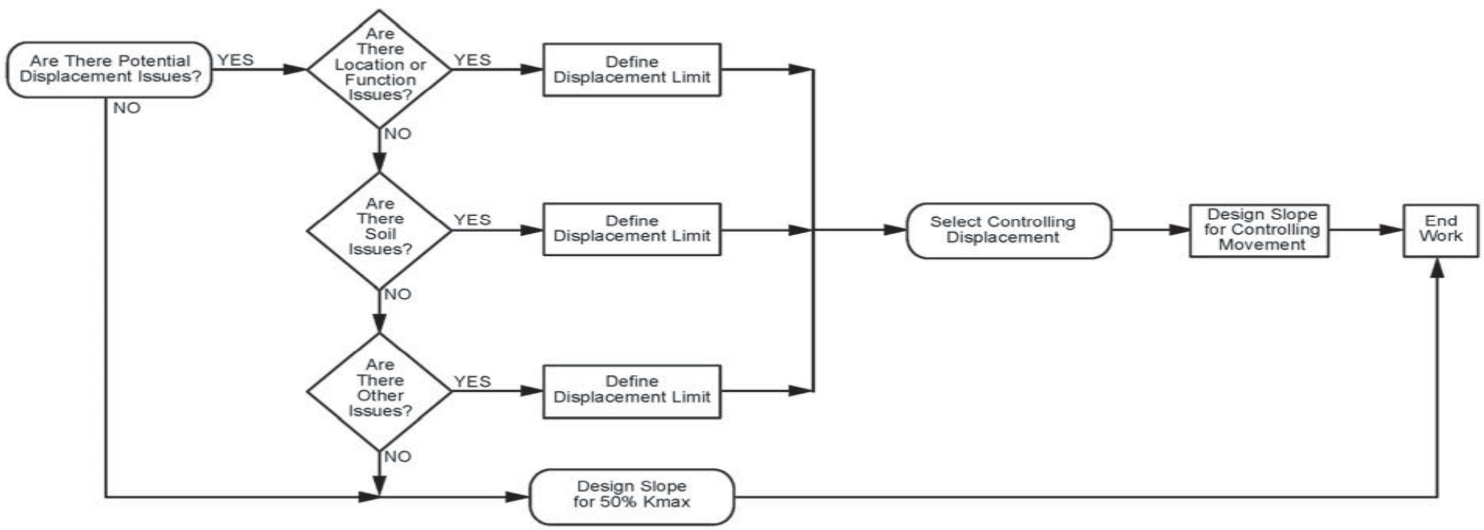

FIGURE Y-1
PROCEDURE TO ESTABLISH ACCEPTABLE SLOPE MOVEMENT
NAS SEISMIC ANALYSIS

EP062007001RDD_06 (6/6/07)

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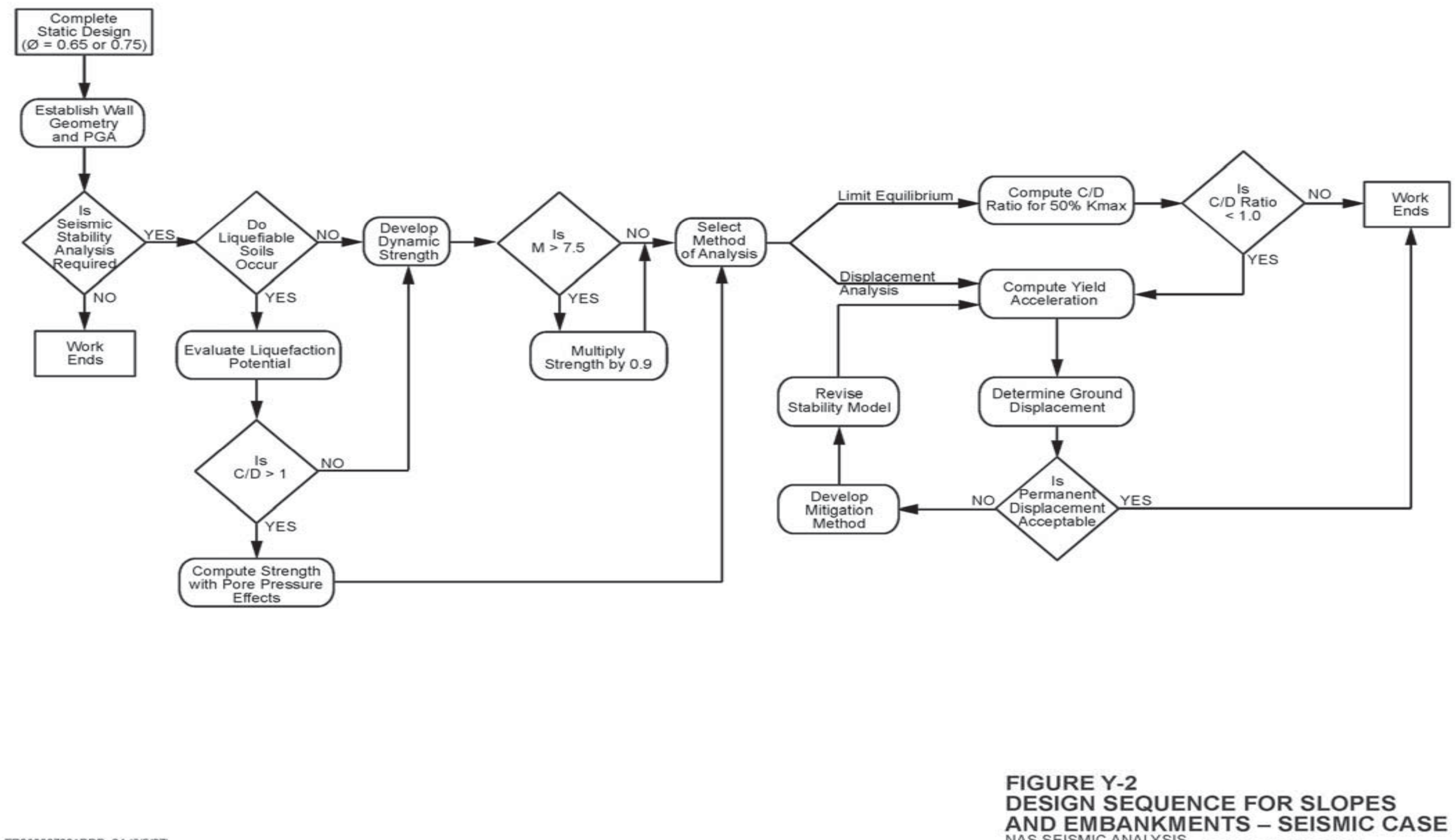

FIGURE Y-2
DESIGN SEQUENCE FOR SLOPES AND EMBANKMENTS – SEISMIC CASE
NAS SEISMIC ANALYSIS
CH2MHILL

SECTION Z: BURIED STRUCTURES (CULVERTS and DRAINAGE PIPES)

TABLE OF CONTENTS

Z.1 SCOPE

The provisions contained in this section shall be used for the seismic design of buried structures commonly used in the highway construction for conveyance of water, utilities, and pedestrians, unless the Owner waives these requirements.

Seismic analysis of buried structures shall not be required under the following conditions:

- The 1-second spectral acceleration from the AASHTO seismic hazard maps adjusted for site effects (F_vS_1) is less than 0.2g, or the design peak ground velocity (PGV) is less than 10 in/sec, except when they are subject to unstable ground conditions including landslides, liquefaction-induced ground movements, fault displacements, or any other type of permanent ground displacements.

- The design F_vS_1 is less than 0.3g and the design PGV is less than 15 in/sec for buried structures with a diameter of less than 3 feet.

Before conducting the seismic analyses and design evaluations described in this Section of the Specifications, buried structures shall be designed for static service and strength limit requirements specified in Section 12 – Buried Structures and Tunnel Liners – of the current AASHTO *LRFD Bridge Design Specifications*.

C.Z.1

This section deals with seismic design of buried structures used for water drainage, utilities, pedestrian undercrossings, and similar types of structures. These structures can be constructed by embankment or trench methods.

The seismic design provisions are those resulting from ground shaking effects (i.e., transient ground displacement, TGD) and not from seismic–induced ground failure. The characteristics of ground failures (i.e., permanent ground displacement, PGD) and their effects on culverts and pipes are extremely complex and must be dealt with on a case-by-case basis.

The provisions contained in this Section apply to buried structures for water conveyance, utilities, and pedestrian structures constructed by embankment or trench methods. The provisions do not apply to highway vehicular traffic tunnel structures built by tunnel boring machines (TBM), mined methods, cut-and-cover methods, and immersed tube tunnel construction. Highway vehicular traffic tunnel structures are usually long in length and used primarily for mass transportation purposes, requiring additional considerations than the provisions provided in this Section. For highway tunnel structures the Owner should specify and/or approve appropriate seismic design requirements.

Based on performance data during past earthquakes, buried culverts and drainage pipes are in general very resistant to earthquake shaking effects (i.e., the transient ground displacement effects, TGD) and have performed better than above-ground structures. Damage to buried structures has rarely occurred under low-to-moderate ground shaking intensity, except when the ground shaking is accompanied by ground failure (i.e., the permanent ground displacement effects, PGD). Therefore, the no-analysis threshold limits are defined at the ground shaking level represented either by F_vS_1<0.2g or by PGV<10 in/sec.

Past observations also suggest that small diameter culverts and drainage pipes (D < 3 feet) have performed even better than their counterparts with larger sizes, resulting in raised shaking intensity threshold limits for the no-analysis requirements.

In some situations, the Owner may decide not to require any seismic analysis or any action to mitigate the potential risk from ground failure conditions (i.e., the PGD effects) if the consequences of the failure are small. In this case the need for seismic design checks may be waived altogether.

Z.2 DEFINITIONS

Buried Structures—Structures generally constructed by embankment or trench methods and including culverts and drainage pipes for water conveyance, utilities, and pedestrian tunnels.

Compressibility Ratio — A term for defining the relative compression stiffness of the ground and a circular structure in the ground.

Flexibility Ratio — A term for defining the relative ovaling stiffness of the ground and a circular structure in the ground. For a rectangular structure this term is used for defining the relative racking stiffness of the ground and the rectangular structure in the ground.

Embedment Depth Ratio — Ratio of soil cover thickness (from ground surface to top of the buried culvert/pipe) to height/diameter of the culvert/pipe.

Z.3 NOTATION

C = compressibility ratio (dim.)
C_{se} = effective shear wave velocity of vertically propagating shear wave (ft/sec.)
d = diameter of circular pipe or culvert (ft.)
E = Youngs modulus (psf.)

E_l = Youngs modulus of liner (pipe) (ft.)
E_m = Youngs modulus of soil (psf.)
F = flexibility ratio (dim.)
F_{rec} = flexibility ratio for rectangular structure (dim.)
F_v = site factor for spectral acceleration at 1 second (dim.)
G = shear modulus (psf.)
G_m = effective, strain compatible shear modulus of ground (psf.)
g = gravitational acceleration
H = soil cover thickness above crown of pipe or culvert (ft.)
h = height of rectangular culvert (ft.)
K_s = racking stiffness (kips/in.)
k_1 = moment coefficient (dim.)
k_2 = thrust coefficient (dim.)
M = moment in pipe (ft-lb/ft.)
M_{max} = maximum bending moment (ft-lb/ft.)
PGA = peak ground acceleration coefficient on rock (Site Class B)..
PGV = peak ground velocity (in./sec.)
R = nominal radius of pipe or culvert (ft.)
R_d = depth dependent stress reduction factor (dim.)
S_1 = spectral acceleration coefficient at 1 second (dim.)
T = thrust in pipe or culvert (lb/ft.)
T_{max} = maximum thrust (lb/ft.)
t = liner cross sectional area per unit length in axial direction (ft.^{2}/ft.)
V = shear in pipe or culvert (lb/ft.)
V_s = shear wave velocity at low shearing strain (ft/sec.)
w = width of rectangular culvert (ft.)
z = depth below ground surface (ft.)

γ = shearing strain in soil (dim.)
γ_{max} = maximum free-field shearing strain in soil (dim.)
γ_p = load factor (dim.)
γ_t = total soil unit weight (pcf.)
ϕ_r = resistance factor (dim.)
υ = Poisson’s ratio (dim.)
υ_m = Poisson’s ratio of soil

υ_l = Poisson's ratio of liner (pipe) (dim.)
σ_v = total vertical soil overburden pressure (psf.)
τ = shearing stress (psf.)
τ_{max} = maximum shearing stress (pcf.)
θ = angle relative to horizontal axis through center of circular pipe/culvert (degrees)
Δ_s = seismic racking deformation (in.)
$\Delta_{free\text{-}field}$ = differential free-field relative displacement (in.)

Z.4 SOIL AND MATERIAL PROPERTIES

Z.4.1 Soil Properties

Soil stiffness parameters [e.g., Young's modulus (E), shear modulus (G), and Poisson's Ratio (υ)] shall be defined for either or both the compacted backfill and natural soils. The effect of strain-level on these stiffness parameters shall be taken into consideration to be consistent with the potential ground shearing strains developed during earthquakes. The shear wave propagation velocity of the ground surrounding the buried structures shall also be determined to allow estimating transient ground shearing strain using either simplified analytical solutions or site response analysis procedures.

The type, compacted density, and strength properties of the foundation soils and backfill soils shall be resistant to liquefaction and subject to minimal liquefaction-induced soil degradation effect.

Subsurface exploration shall be carried out to determine the potential presence of geotechnical/geological seismic hazards that may affect the performance of buried structures under the design seismic event. These hazards may include, but are not limited to, faulting, landslides, liquefaction, liquefaction-induced lateral spread, mudflow, or subsidence.

The types and anticipated behavior of the foundation soils and backfill soils surrounding the buried structures are described in Section 12 of the AASHTO *LRFD Bridge Design Specifications*. Section 10 of the AASHTO *LRFD Bridge Design Specifications* provides guidance on methods to use for field explorations and laboratory soil testing.

The methodology used in seismic design of buried structures is a deformation-based analysis, and therefore it is important that realistic soil stiffness be used under the seismic loading condition in order to properly account for the soil-structure interaction effect between the culvert/pipe structure and the surrounding ground.

Previous studies including those from laboratory test results have shown that the shear modulus values are dependent on the shearing strain levels. At low shearing strain amplitudes ($\gamma < 0.0001\%$) the shear modulus values can be reliably estimated from the field-measured shear wave velocities, such as by using the seismic cone, downhole, cross-hole, P-S logging, and SASW (spectral analysis of surface wave) techniques. As the shearing strain increases, the shear modulus degradation effect becomes important. The shearing strain level is also a function of the ground shaking intensity. As the ground motion intensity increases, the shearing strain increases, resulting in reduced equivalent shear modulus.

Typical relationships between the shear modulus degradation and the shear strain level can be found in the Electric Power Research Institute (EPRI) report *Guidelines for Determining Design Basis Ground Motions, Volume 1: Method and Guidelines for Estimating Earthquake Ground Motion in Eastern North America* (EPRI, 1993).

Shear wave propagation velocity measured in the field corresponds to the values measured at the very small strain level. The effective shear wave velocity and the corresponding strain-compatible soil modulus during earthquake shaking should be reduced for strain compatibility before using for engineering design purposes.

Z.4.2 Material Properties

Material properties for various buried structure systems (including aluminum pipe and structural plate structures, steel pipe and structural plate structure, concrete pipe and structures, steel reinforcement, and thermoplastic pipe) shall be in accordance with those specified in Section 12 of the AASHTO *LRFD Bridge Design Specifications*.

Z.5 SEISMIC LOADS AND LOAD FACTORS

Seismic loads in buried structures shall be determined using ground displacement-based (or ground strain-based) procedures, either following the simplified method summarized in Articles Z.7.3.1 and Z.7.3.2 or using the numerical modeling method outlined in Article Z.7.3.3. A load factor (γ_p) of 1.0 shall be used

The current AASHTO *LRFD Bridge Design Specifications* require that the load factor (γ_p) for live load in combination with the seismic load should be determined on a project-specific basis. On a heavily traveled roadway where the buried structure is likely to be subjected to the live in conjunction with the methodology given in this Section of the Specifications to determine the seismic loads.

The load factor for live load in Extreme Event I (per AASHTO LRFD Bridge Design Specifications Section 3) shall be determined on a project-specific basis.

load on a nearly continuous basis, the load factor for live load should usually be taken at least equal to 0.5. Using a load factor of 0.5 is reasonable for a wide range of values of average daily truck traffic (ADTT).

Z.6 LIMIT STATES AND RESISTANCE FACTORS

Seismic effects on buried structures shall be investigated for Extreme Event I, as specified in Table 3.4.1-1 in Section 3 of the AASHTO *LRFD Bridge Design Specifications.*

Resistance factors for buried structures shall be taken as $\phi_r = 1.0$. Values of resistance factors for the geotechnical design of foundations for buried structures shall also be taken as $\phi_r = 1.0$.

Z.7 SEISMIC DESIGN

Z.7.1 General Requirements

Under the seismic loading condition, buried structures shall be investigated, as a minimum, for the following potential failure modes depending on the type of the structures:

1. For metal structures:
   - Buckling
   - Flexure
   - Seam failure

2. For concrete structures:
   - Flexure
   - Shear
   - Thrust

3. For thermoplastic pipe:
   - Buckling
   - Flexure

4. For liner plate:
   - Buckling
   - Flexure

C.Z.7.1

The seismic design of culverts and pipes can be separated into two categories, rigid and flexible, depending on the flexibility of the pipe or culvert.

Flexible Culverts and Pipes

For static service and strength limit design, Section 12 of the current AASHTO *LRFD Bridge Design Specifications* requires as a minimum the following main design considerations (in addition to the seam failure) for flexible culverts/pipes: (1) buckling, and (2) flexibility limit for construction.

Except for large box structures or other large spans with shapes other than circular (NCHRP, 2002), the flexural strength consideration (i.e., bending moment demand) is generally not required for flexible culverts/pipes. However, seismic loading is in general non-symmetric in nature and therefore may result in sizable

- Seam strength

bending in a culvert or pipe structures (even for flexible and/or circular shape culverts/pipes).

In view of the non-symmetric nature of seismic loading, it is important that both seismically induced bending (flexure) and thrust (buckling) be evaluated for the seismic performance of flexible culverts/pipes such as thin corrugated metal pipes (CMP) and thermoplastic conduits (e.g., corrugated HDPE pipes).

Rigid Culverts and Pipes

Rigid highway culverts/pipes consist primarily of reinforced concrete shapes that are either precast or cast-in-place. Unreinforced concrete culverts/pipes are not recommended for use in seismic regions.

Unlike the flexible culverts/pipes, the rigid culverts must develop significant ring stiffness and strength to support external pressures. Hence, they are not as dependent upon soil support as flexible culverts, and bucking is generally not an issue with rigid converts/pipes.

Z.7.1.2 Seismic Loading Effects

In addition to normal loads, buried structures shall be designed to accommodate the effects resulting from two types of seismic loads:

- Ground shaking (i.e., transient ground displacement, TGD).
- Ground failure (i.e., permanent ground displacement, PGD).

When subject to TGD, buried structures as a minimum shall be evaluated for the following modes of deformation resulting from vertically propagating shear waves:

- Ovaling deformation for buried structures with circular cross section.

C.Z.7.1.2

Two types of loadings can occur to a pipe or culvert during a seismic event. One is the vibratory effects, as characterized by the peak ground acceleration or ground velocity. The vibratory effects are a transitory response, lasting for a few tens of seconds to several minutes. The other involves permanent ground displacements when the ground fails under the inertial loads of the earthquake.

Ground Shaking Ground shaking refers to the vibration of the ground produced by seismic waves propagating through the crust of the earth. When subject to ground shaking (TGD), the response of a buried linear culvert/pipeline structure can be described in terms of three

- Racking deformation for buried structures with rectangular or other non-circular cross sections.

The methods of analysis for evaluating the ovaling/racking deformation effects shall be in accordance with those specified in Articles Z.7.3.1 and Z.7.3.2 using the simplified procedures or in Article Z.7.3.3 using the numerical modeling procedure.

When subject to ovaling/racking deformations, a flexural type failure mode due to the combined effects of bending moment and thrust force shall be checked for both rigid culverts/pipes (such as those constructed with reinforced concrete) and flexible culverts/pipes (typically, thin-walled conduits constructed with steel, aluminum, or thermoplastic such as HDPE or PVC). For flexible culverts/pipes, the potential failure mode by buckling shall also be evaluated.

The general methodology for evaluating the effects of PGD (i.e., ground failure) shall be in accordance with those outlined in Article Z.7.5.

principal types of deformations: (a) axial deformations, (b) curvature deformations (refers to Figure Z.7-1), and (c) ovaling (for circular cross section) or racking (for rectangular cross section) deformations (Figure Z.7 - 2).

The axial and curvature deformations are induced by components of seismic waves that propagate along the culvert/pipeline axis. The potential failure modes for long, continuous pipeline structures subject to axial and curvature deformations consist of (1) rupture due to axial tension (or pull out for jointed segmented pipelines), or (2) local bucking (wrinkling) due to axial compression and flexural failure.

If the pipeline is buried at a shallow depth, a continuous pipeline in compression can also exhibit beam-buckling behavior (i.e., global bucking with upward buckling deflections). It should be noted, however, that typical culvert structures for transportation applications are generally of limited length. For this condition it is in general unlikely to develop significant transient axial/curvature deformations along the culvert structures.

The potential axial/curvature failure modes mentioned above are not likely to take place during the earthquake. The main focus of the seismic evaluation for highway culvert structures, therefore, will not be on the effects of axial/curvature deformations. Instead, the main concern will concentrate on transverse deformations of culverts and pipes.

The ovaling or racking deformations of a buried culvert/pipe structure may develop when waves propagate in a direction perpendicular or nearly perpendicular to the longitudinal axis of the culvert/pipe, resulting in a distortion of the cross-sectional shape of the structure.

Design considerations for this type of deformation are in the transverse direction. Figure Z.7.2 shows the ovaling distortion and racking deformation associated with a circular culvert/pipe and a rectangular culvert, respectively. The general behavior of the structure may be simulated as a buried structure subject to ground deformations under a two-dimensional, plane-strain condition.

Ovaling and racking deformations may be caused by vertically, horizontally or obliquely propagating seismic waves of any type. Many previous studies have suggested, however, that the vertically propagating shear wave is the predominant form of earthquake loading that governs the ovaling/racking behavior.

Ground Failure

Ground failure broadly includes various types of ground instability such as faulting, landslides, liquefaction (including liquefaction-induced lateral spreading, settlement, flotation, etc.) and tectonic uplift and subsidence. These types of ground deformations are called Permanent Ground Deformations (PGD).

Each of these PGDs may be potentially catastrophic to culvert/pipeline structures, although the damages are usually localized. To avoid such damage some sort of ground improvement is generally required, unless the design approach to this situation is to accept the displacement, localize the damage, and provide means to facilitate repairs.

The characteristics of PGD and its effects on culvert and pipes are extremely complex and must be dealt with on a case-by-case basis.

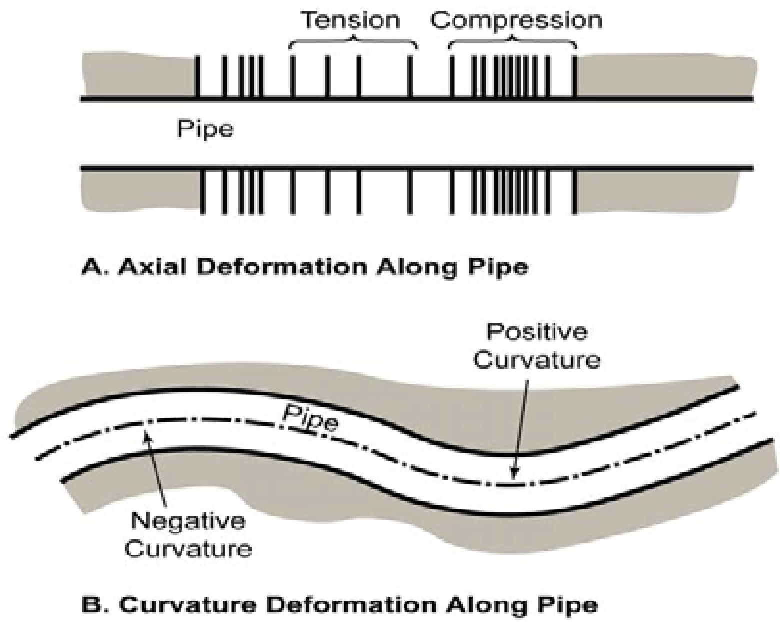

Figure Z.7-1 Axial/Curvature Deformations

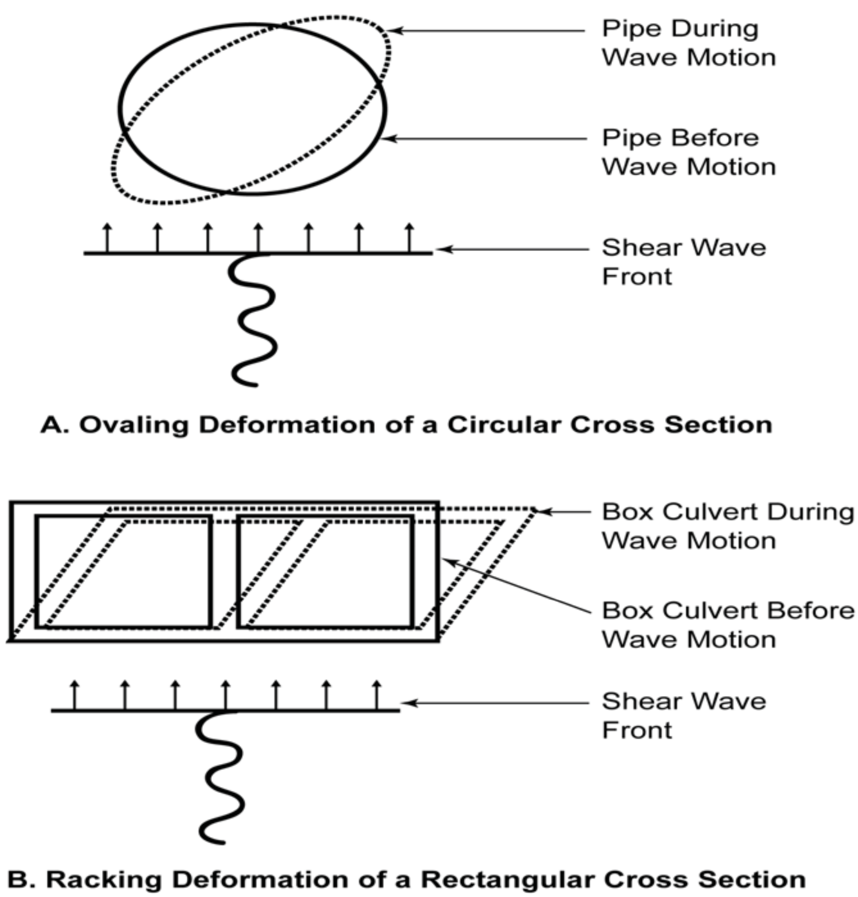

Figure Z.7-2 Ovaling/Racking Deformations

Z.7.3 Method of Analysis - Transient Ground Displacement

Z.7.3.1 Simplified Method for Ovaling of Circular Conduits

Ovaling effects on circular culverts/pipes shall be based on the maximum free-field ground strain (or displacement) caused by the governing vertically propagating shear waves of the design earthquake event. The analysis shall take into account the soil-structure interaction effect to provide safe as well as realistic design.

The interface condition (i.e., full-slip versus no-slip) at the interface between the exterior wall of the structure and the surrounding soils shall be conservatively assumed to attain maximum structural response in terms of bending moments and thrust/hoop forces.

C.Z.7.3.1

An important aspect for evaluating TGD effects on culvert/pipe structures is to first determine the ground strain in the free-field (in this case free-field shearing strain, γ_{max}) and then determine the response of the structures to the ground strain.

The most appropriate design ground motion parameters to characterize the transient ground motion effects for buried structures include the site-adjusted peak ground acceleration (F_{pga} PGA) and peak ground velocity (PGV) (in this case the shear wave peak particle velocity). The site-adjusted PGA is more suitable for estimating the maximum free-field shearing strain for structures buried to relatively shallow depth (say, <75 feet from ground surface). For structures buried to a significant depth (say, >75 feet from ground surface) using PGV parameter is more appropriate.

The ground strain is somewhat location dependent. Given the same site-adjusted PGA value, the anticipated peak ground strain for a Central and Eastern United States (CEUS) site could be much smaller than that for a site in Western United States (WUS). Estimating PGV from the site-adjusted 1-second spectral acceleration F_vS_1, as specified in Article X.4.2, provides more accurate results in computing ground strains.

It should also be noted that the effective shear wave velocity of the vertically propagating shear wave (C_{se}) should be compatible with the level of the shearing strain that may develop in the ground at the elevation of the conduit under the design earthquake shaking. For stiff to very stiff soil, C_{se} /V_s may range from 0.6 to 0.85 where V_s is the small-strain shear wave velocity measured in the field. For soft soil it is recommended that C_{se} be computed from a site response analysis.

Z.7.3.1.1 Maximum Free-Field Ground Strain/Displacement

The maximum free-field ground strain caused by the governing vertically propagating shear waves shall be computed using either simplified methods or a site response analysis.

C.Z.7.3.1.1

Two simplified methods can be used to estimate the maximum free-field ground strain, one for depths less than 75 feet and one for depths greater than 75 feet.

Depths Less Than 75 Feet

For most highway culverts/pipes, the burial depths are generally relatively shallow (i.e., within the upper 50 feet of the ground surface). Under this condition, it is reasonable to estimate the maximum free-field shear strain using the earthquake-induced shear stresses dividing the shear stiffness (i.e., the strain-compatible effective shear modulus) of the surrounding ground.

In this simplified method the maximum free-field ground shear strain (γ_{max}) is calculated using the following equation:

$$\gamma_{max} = \tau_{max}/G_m \qquad \text{(C.Z.7.3-1)}$$

Where:

G_m = the effective strain-compatible shear modulus of the ground surrounding the culvert/pipe structure.

τ_{max} = the maximum earthquake-induced shear stress and can be estimated as follows:

$PGA\ \sigma_v\ R_d$

σ_v = the total vertical soil overburden pressure at the depth corresponding to the invert elevation of the pipe and is determined using the following equation:

γ_t (H+d)

γ_t is the total unit weight; H is the soil cover thickness measured from ground surface to the crown elevation; and d is the diameter of the circular culvert/pipe.

R_d the depth dependent stress reduction factor and can be estimated using the following relationships:

for $z < 30$ ft

$R_d = 1.0 - 0.00233z$

for $30 \text{ ft} < z < 75 \text{ ft}$

$R_d = 1.174 - 0.00814z$

z = Depth of interest = (H+d)

<u>Depths Greater Than 75 Feet</u>

The second simplified method involves use of the following simplified strain formula and is more suitable for structures buried to significant depth (e.g., greater than 75 feet) from the ground surface:

$$\gamma_{max} = PGV / C_{se} \qquad \text{(C.Z.7.3-2)}$$

Where:

γ_{max} = maximum free-field shear strain at the elevation of the conduit

PGV = peak ground velocity, PGV) <u>at the conduit elevation</u> and can be conservatively estimated using the correlation between PGV and the site-adjusted, 1-second spectral acceleration F_vS_1, as specified in Article X.4.2.

C_{se} = effective shear wave velocity of soil surrounding the conduit, as discussed in Article C.Z.7.3.1.

<u>*Site Response Analyses*</u>

Alternatively, the maximum free-field shearing strain can also be estimated by a more refined free-field site response analysis (e.g., using site response analysis program SHAKE – Schnabel et al., 1972).

When a site response analysis is performed, the design maximum free-field shearing strain should be based on the maximum shearing strain value computed for the full vertical profile of the conduit (i.e., from the crown to the invert). Using the <u>average</u> maximum ground strain value (i.e., differential shear displacement between the crown and the invert divided by the height of the conduit) may lead to significant underestimation of structure response, particularly for conduit with shallow burial depth (e.g., for Embedment Depth Ratio less than 2.0, where the embedment ratio is the ratio of soil cover thickness to height of the conduit).

Z.7.3.1.2 Soil-Structure Interaction Effects

To account for the soil-structure interaction effects, two relative stiffness parameters [Compressibility Ratio (C) and Flexibility Ratio (F)] shall be derived using the following formula:

$$C = \{E_m (1-\nu_l^2) R\} / \{E_l\ t (1+\nu_m)(1-2\nu_m)\} \quad \text{(Z.7.3-1)}$$

C.Z.7.3.1.2

The flexibility ratio (F) represents the relative distortion stiffness between the surrounding soil and the lining and tends to govern the bending response (distortion) of the lining. The compressibility ratio (C) represents the relative ring compression stiffness between the surrounding soil and the lining and tends to dominate the thrust/hoop forces in the lining. When F < 1.0, the lining is considered stiffer than the

$$F = \{E_m (1-\nu_l^2) R^3\} / \{6 E_l I_l (1+\nu_m)\} \quad \text{(Z.7.3-2)}$$

Where:

E_m = Youngs modulus of soil

ν_l = Poisson's ratio of liner

R = Nominal radius of the conduit

E_l = Youngs modulus of liner

t = Lining cross sectional area per unit length in the axial direction

ν_m = Poisson's ratio of soils

ground, and it tends to resist the ground and therefore deforms less than that which occurs in the free-field. On the other hand, when F > 1, the lining is expected to deform more than the free-field.

As the flexibility ratio continues to increase, the lining deflects more and more than the free-field and may reach an upper limit as the flexibility ratio becomes infinitely large. This upper limit deflection is equal to the deformations displayed by a perforated ground (i.e., an excavated conduit in the ground with no lining stiffness).

The strain-compatible elastic modulus of the surrounding ground (E_m) should be derived using the strain-compatible shear modulus (G_m) corresponding to the effective shear wave propagating velocity (C_{se}).

Z.7.3.1.3 Design Forces

Using the estimated maximum ground strain (γ_{max}) and the two soil-structure interaction parameters derived above (C and F), the maximum bending moment ($M_{max}(\theta)$) and the maximum thrust/hoop force ($T_{max}(\theta)$), can be derived as follows:

$$M_{max}(\theta) = \{(1/6) k_1 [E_m / (1 + \nu_m)] R^2 \gamma_{max}\} \sin 2\theta \quad \text{(Z.7.3-3)}$$

(full-slip interface condition)

$$T_{max}(\theta) = \{k_2 [E_m / 2(1 + \nu_m)] R \gamma_{max}\} \sin 2\theta \quad \text{(Z.7.3-4)}$$

(no-slip interface condition)

Where:

θ = the angle relative to the horizontal axis through center of circular culvert/pipe

C.Z.7.3.1.3

In most cases, the actual condition at the interface is between full-slip and no-slip. In computing the forces in the lining, it is prudent to investigate both cases and the more critical one should be used in design. The full-slip condition gives more conservative results in terms of maximum bending moment ($M_{max}(\theta)$). This conservatism is desirable to offset the potential underestimation (about 15 percent) of lining forces resulting from the use of a pseudo-static model used in deriving these close-form solutions in lieu of the dynamic loading condition (i.e., some dynamic amplification effect). Therefore, the solutions derived based on the full-slip interface assumption should be used in evaluating the bending moment response of a circular conduit (i.e., culverts/pipes).

The maximum thrust/hoop force ($T_{max}(\theta)$) calculated using the full-slip assumption, however, may be significantly

$k_1 = 12\,(1-\nu_m)\,/\,(2F+5-6\nu_m)$

(Figure Z.7-3)

$k_2 = 1 + \{F[(1-2\nu_m) - (1-2\nu_m)C] - \tfrac{1}{2}(1-2\nu_m)^2 C+2\} / \{F[(3-2\nu_m) + (1-2\nu_m)C] + C[5/2-8\nu_m+6\nu_m^2] + 6 - 8\nu_m\}$

(Figures Z.7.4 through Z.7.6)

underestimated and may lead to unsafe results, particularly for thin-walled conduits (i.e., flexible culverts/pipes) where buckling potential is the key potential failure mode. Therefore, the no-slip interface assumption should be used in assessing the lining thrust response, unless a more refined analysis is conducted to more accurately account for the actual field condition (i.e., semi-non-slip condition).

The seismic forces presented in this section are incremental to other normal loading cases. For evaluating the seismic behaviors of the structures the seismic forces should be combined with appropriate other loading cases (such as the effects of dead load, earth load, or live load, if applicable) using the Extreme Event I, as specified in Table 3.4.1-1 in Section 3 of the AASHTO *LRFD Bridge Design Specifications*. In combining the seismic forces with other loads it is important to note that the seismic forces are not uniformly distributed throughout the culvert/pipe lining. Instead, they distribute as a function of θ, specifically as presented by the two equations for $M_{max}(\theta)$ and $T_{max}(\theta)$, where θ is the angle relative to the horizontal axis through the center of the circular culvert/pipe. The maximum values of the M_{max} and T_{max} occur at $\theta = 45^o$, 135^o, 225^o, and 315^o (i.e., the shoulder and knee locations of the circular culvert/pipe).

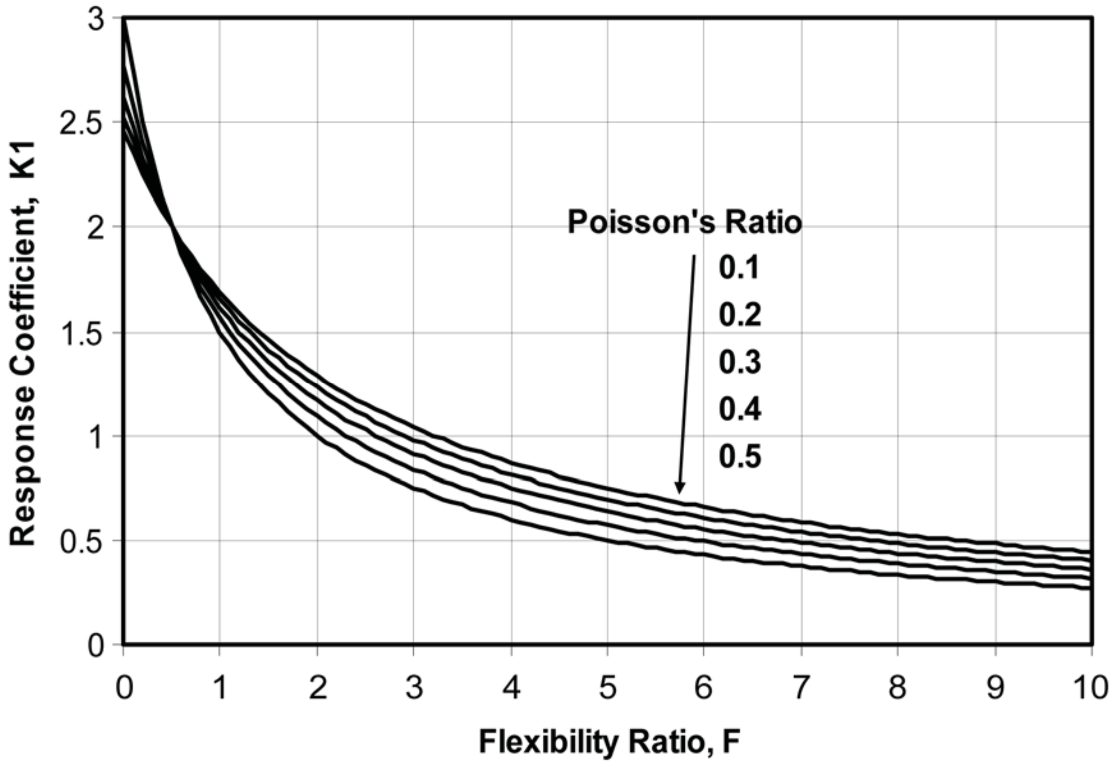

Figure Z.7-3 Bending Response Coefficient, k_1 (Full-Slip Interface)

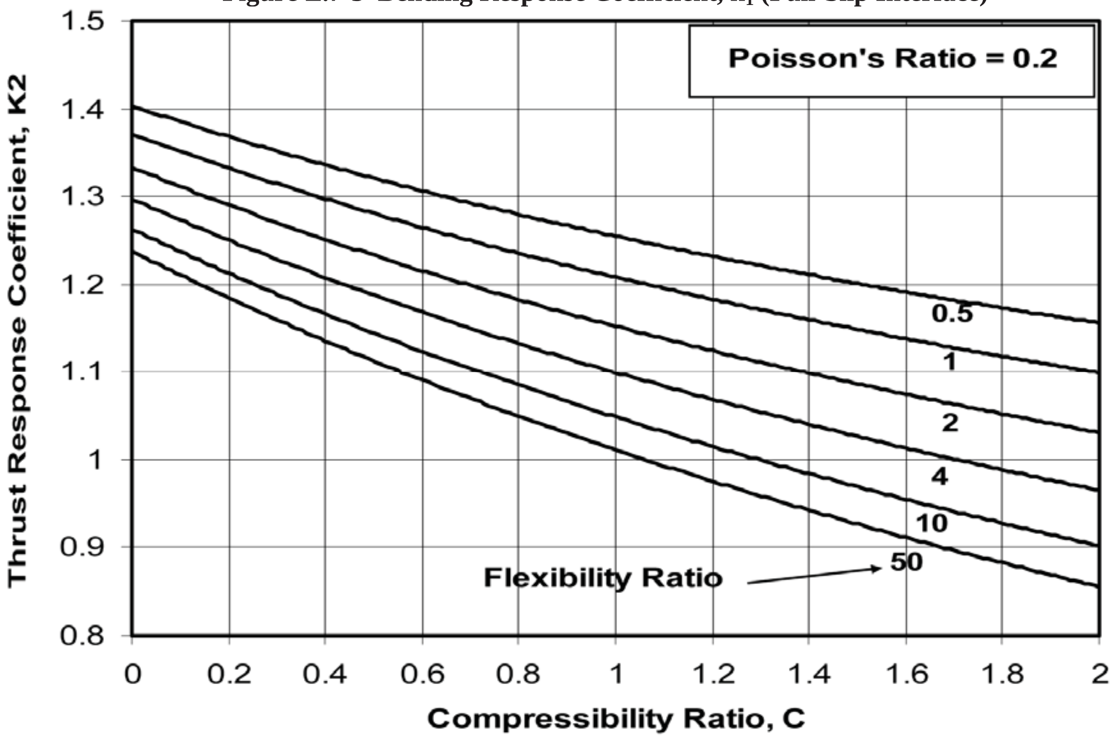

Figure Z.7-4 Thrust/Hoop Force Response Coefficient, k_2
(No-Slip Interface; Soil Poisson's Ratio = 0.2)

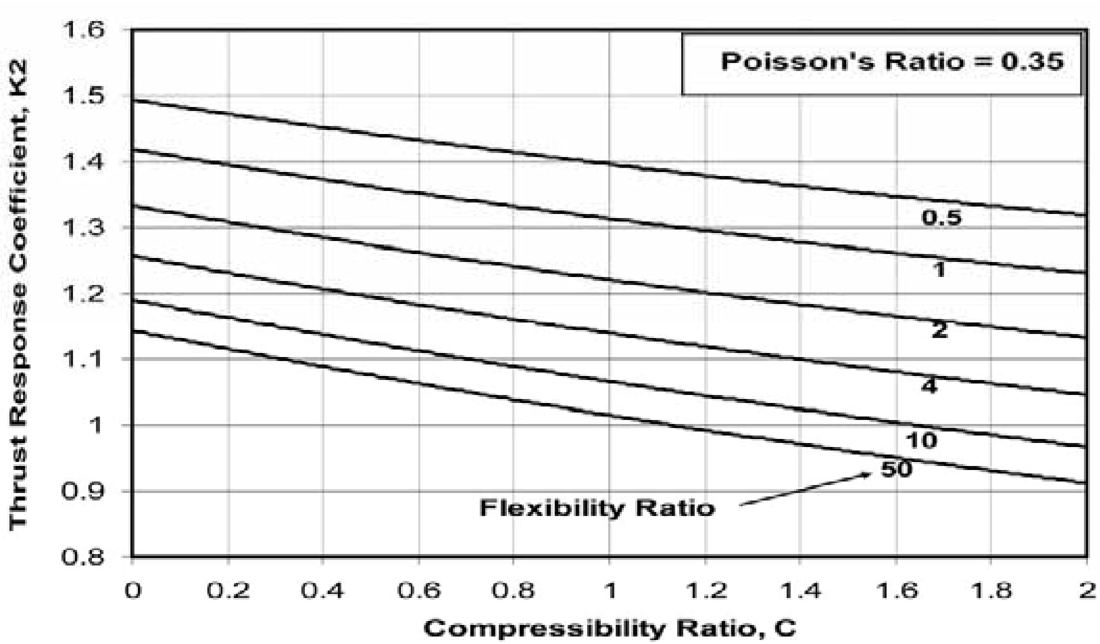

Figure Z.7-5 Thrust/Hoop Force Response Coefficient, k_2
(No-Slip Interface; Soil Poisson's Ratio = 0.35)

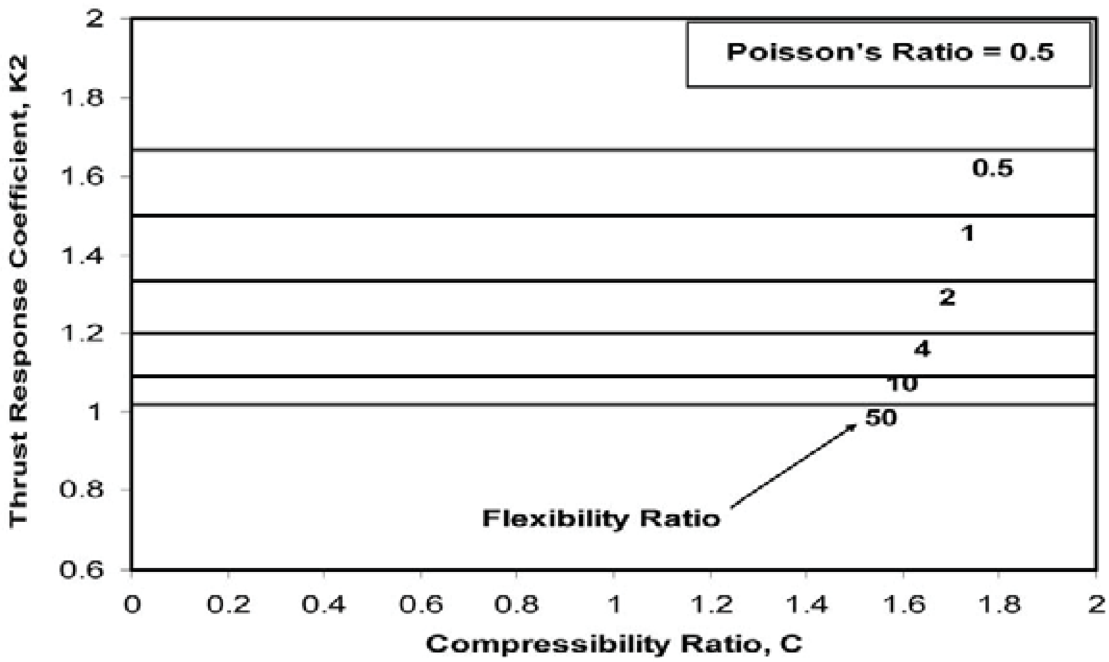

Figure Z.7-6 Thrust/Hoop Force Response Coefficient, k_2
(No-Slip Interface; Soil Poisson's Ratio = 0.5)

Z.7.3.2 Simplified Method for Racking of Rectangular Conduits

Racking deformations shall be defined as the differential sideways movements between the top and bottom elevations of the rectangular structures, shown as "Δ_S" in Figure Z.7-7. The resulting structural internal forces or material strains in the structure lining associated with the seismic racking deformation (Δ_S) shall be derived by imposing the differential deformation on the structure in a simple structural frame analysis.

C.Z.7.3.2

The simplified procedure presented in this section applies to box type buried culverts of rectangular and square shapes and various configurations including flat top three-sided, one-on-one two barrels, one-by-one two barrels, etc. It may also be used to provide approximate estimates of seismic effects on buried culverts of non-box type configurations and shapes (e.g., arch type). This can be achieved, for example, by (1) deriving the racking stiffness (K_S) by applying a unit lateral force at the top of the crown in Step 2, and (2) deriving the flexibility ratio (F_{rec}) by using an equivalent width of the structure (by equal area method) in Step 3.

The procedure for determining Δ_S, and the corresponding structural internal forces [bending moment (M,), thrust (T), and shear (V)] taking into account the soil-structure interaction effects, is presented below:

- ***Step 1.*** Estimate the free-field ground shearing strains γ_{max} (at the structure elevation) caused by the vertically propagating shear waves from the design earthquakes (as discussed in Article Z.7.3.1.1). Determine the differential free-field relative displacements corresponding to the top and the bottom elevations of the rectangular/box structure ($\Delta_{free\text{-}field}$) in Figure Z.7-7 by:

$$\Delta_{free\text{-}field} = h \cdot \gamma_{max}$$

Where:

h = height of the rectangular structure

- ***Step 2.*** Determine the racking stiffness (K_S) of the structure from a simple structural frame analysis. For practical purposes, the racking stiffness can be obtained by applying a unit lateral force at the roof level, while the base of the structure is restrained against translation, but with the joints free to rotate. The structural racking stiffness is defined as the ratio of the applied force to the resulting lateral displacement.

- ***Step 3.*** Derive the flexibility ratio (F_{rec}) of the rectangular structure using the following formula:

$$F_{rec} = (G_m / K_S) \cdot (w/h)$$

Where:

w = width of the structure
G_m = average strain-compatible shear modulus of the surrounding soil

The flexibility ratio is a measure of the relative racking stiffness of the surrounding soil to the racking stiffness of the structure (Wang, 1993).

- ***Step 4.*** Based on the flexibility ratio obtained form Step 3 above, determine the racking ratio (R_{rec}) for the structure using Figure Z.7-8 or the following expression:

$$R_{rec} = 2F_{rec} / (1+F_{rec})$$

- ***Step 5.*** Determine the racking deformation of the structure (Δ_S) using the following relationship:

$$\Delta_S = R_{rec} \cdot \Delta_{free\text{-}field}$$

- ***Step 6.*** Compute the seismic demand in terms of internal forces (M, T, and V) as well as material strains by imposing Δ_S upon the structure in a frame analysis as depicted in Figure Z.7-9.

The analysis procedure presented in this section is intended to address the incremental effects due to earthquake-induced TGD only. The seismic effects of transient racking/ovaling deformations on culverts and pipes must be considered additional to the normal load effects including surcharge, pavement, and wheel loads, and then compared to the various failure criteria considered relevant for the type of culvert structure in question, as specified in Article Z.7.1 - *General Requirements.*

If more accurate results are required for structures that have non-regular shapes (or long-span) and/or in highly variable subsurface conditions, a more refined numerical modeling method as discussed in Article Z.7.3.3 is recommended, particularly for an important culvert structure in high seismic area.

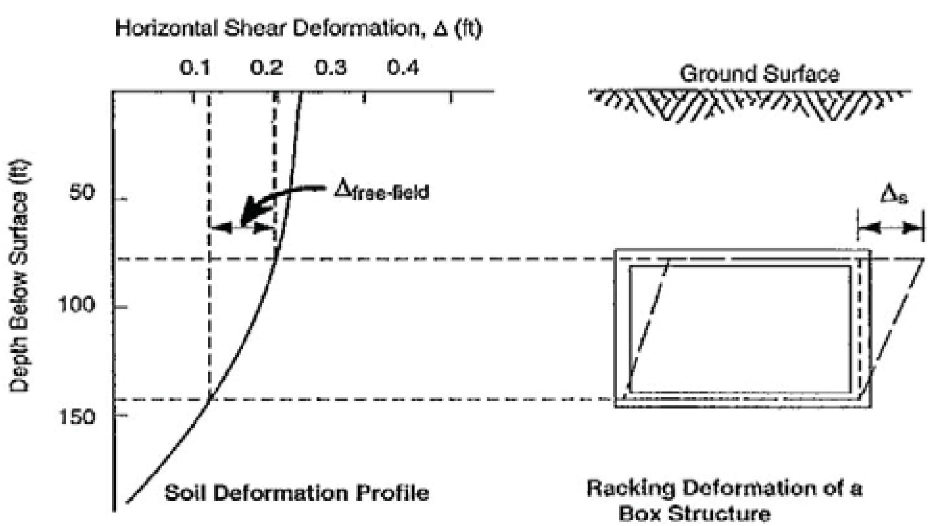

Figure Z.7-7 Racking Deformations of A Rectangular Conduit

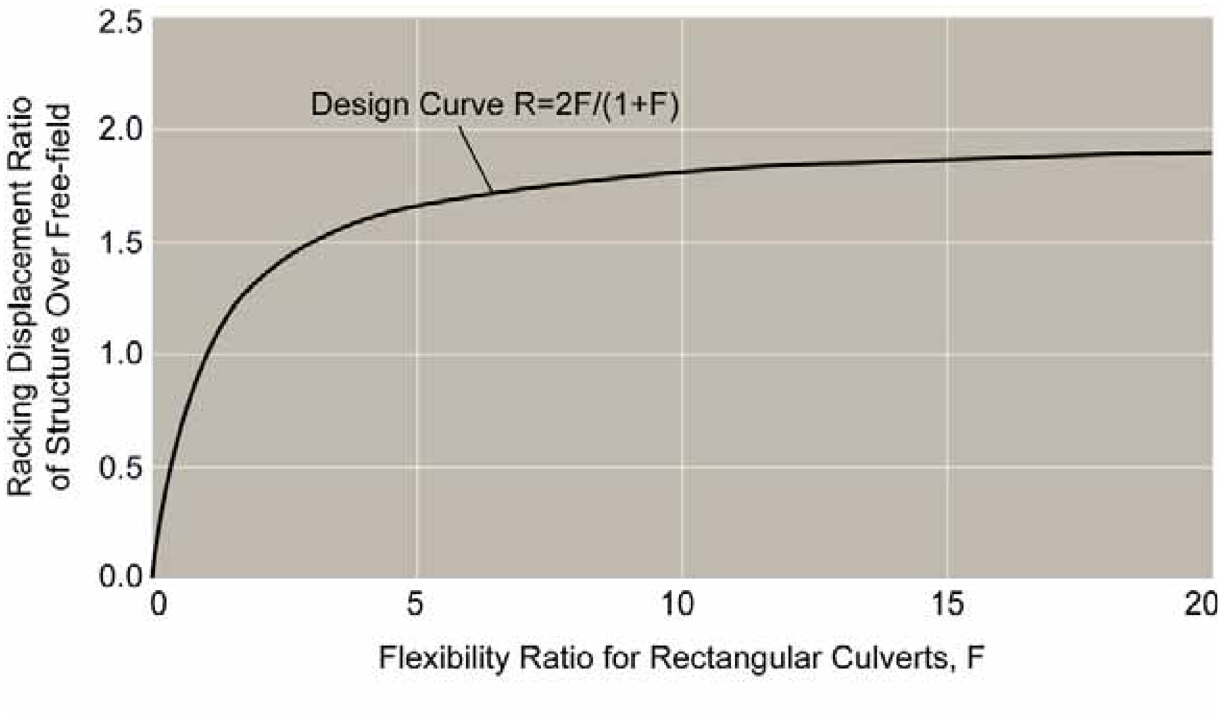

Figure Z.7-8 Racking Ratio between Structure and Free-Field

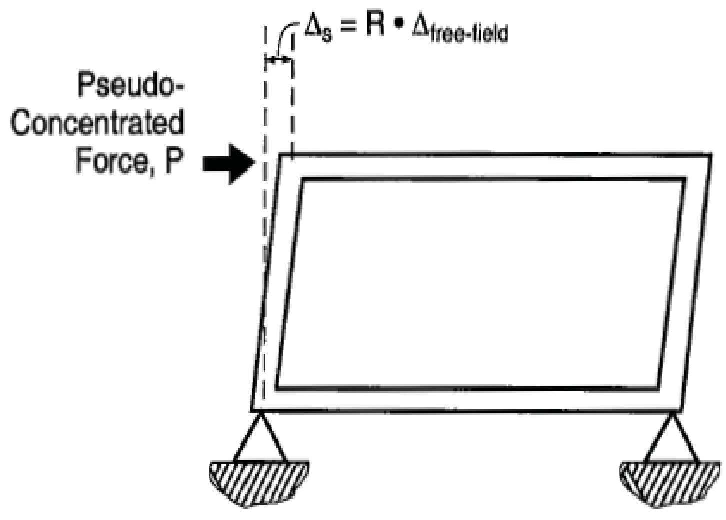

Figure Z.7-9 Simple Frame Analysis of Racking Deformations

Z.7.3.3 Numerical Modeling Methods

In situations where the simplified methods for estimating seismic forces are inadequate, more rigorous 2-dimensional soil-structure interaction continuum numerical modeling methods shall be used. The numerical modeling shall be based on either (1) pseudo-static seismic coefficient induced deformation methods or (2) dynamic time history analyses.

C.Z.7.3.3

There are a number of situations that may warrant the use of numerical modeling, including where (1) the geometry is too complex to be represented by a circular or box-type structure, (2) long-span culverts that may be sensitive to seismic loads are being used, (3) critical (important) structures are identified, (4) highly variable subsurface conditions occur, and (5) in high seismic areas.

Two types of two-dimensional finite element (or finite difference) continuum numerical modeling methods can be used:

- Pseudo-static seismic coefficient induced deformation methods, where the free-field soil deformation profile is generated (induced) by seismic coefficients and distributed in the finite element/finite difference domain that is being analyzed. The seismic coefficients can be derived from a separate one-dimensional, free-field site response analysis.

- Dynamic time history analysis methods, where the entire soil-structure system is subject to seismic excitations using ground motion time history records as input. The ground motion records must be selected to be consistent with the design response spectra and design earthquake source characteristics.

The pseudo-static seismic coefficient induced deformation method (the simpler of the two methods discussed above) is a generally accepted method of analysis for underground structures buried at shallow depths and is particularly suited for conventional highway culverts/pipes where the burial depths are generally shallow (i.e., within 75 feet from ground surface). In this analysis it is assumed that ground stability is not of concern. The general procedure in using this method is outlined below:

- Perform one-dimensional free-field site response analysis (e.g., using SHAKE program). From the results of the analysis derive the maximum ground acceleration profile expressed as a function of depth from the ground surface.

- Develop the two-dimensional finite element (or finite difference) continuum model incorporating the entire excavation and soil-structure system, making sure the lateral extent of the domain (i.e., the horizontal distance to the side boundaries) is sufficiently far to avoid boundary effects. The side boundary conditions should be in such a manner that all horizontal displacements at the side boundaries are free to move and vertical displacements are prevented (i.e., fixed boundary condition in the vertical direction and free boundary condition in the horizontal direction). These side boundary conditions are considered adequate for a site with reasonably leveled ground surface subject to lateral shearing displacements due to horizontal excitations.

- The strain-compatible shear moduli of the soil strata computed from the one-dimensional site response analysis (e.g., using the SHAKE program) should be used in the two-dimensional continuum model.

- The maximum ground acceleration profile (expressed as a function of depth from the ground surface) derived from the one-dimensional site response analysis is applied to the entire soil-structure system in the horizontal direction in a pseudo-static manner.

- The analysis is executed with the culvert structure in place using the prescribed horizontal maximum acceleration profile and the strain-compatible shear moduli in the soil mass. It should be noted that this pseudo-static seismic coefficient approach is not a dynamic analysis and therefore does not involve displacement, velocity, or acceleration histories. Instead, it imposes ground shearing displacements throughout the entire soil-structure system (i.e., the two-dimensional continuum model) by applying pseudo-static horizontal shearing stresses in the ground. The pseudo-static horizontal shearing stresses increase with depth and are computed by analysis as the product of the total soil overburden pressures (representing the soil mass) and the horizontal seismic coefficients. The seismic coefficients represent the peak horizontal acceleration profile derived from the one-dimensional free-field site response analysis. As discussed above the lateral extent of the domain in the two-dimensional analysis system should be sufficiently far to avoid boundary effects. In this manner, the displacement profiles at the two side boundaries are expected to be very similar to that derived from the one-dimensional free-field site response analysis. However, in the focus area near the culvert construction the displacement distribution will be different from that of the free field, reflecting the effects of (1) soil-structure interaction, and (2) the earth mass removed for constructing the culvert.

The above procedure can be used for culvert structures with any geometry.

Z.7.4 Permanent Ground Displacements

Stability of ground surrounding buried structures, including natural and backfill soils located within a zone that may influence the performance of the structures during and after earthquakes, shall be considered in the design. This assessment shall consider the potential for ground failure from liquefaction, slope instability (landslide), and fault displacements.

C.Z.7.4

The effects of liquefaction and liquefaction-induced ground deformations should be evaluated. These effects include the following: (1) uplift, buoyancy, and flotation of the buried structures; (2) large lateral displacement; and (3) post-liquefaction settlements and deformations, total as well as differential.

An initial screening study (NCEER, 1997) should be carried out, followed by more refined analyses and evaluations of the impact on the proposed structures, to assess the risk of liquefaction-related permanent ground displacement. If the liquefaction impact analyses yield unacceptable performance of the structures, mitigation measures should be incorporated into the design.

The evaluation for seismically induced landslides and slope instability, if identified, should also be conducted in accordance with the procedures specified in Section Y, followed by impact study. If the impact analyses yield unacceptable performance of the structures, mitigation measures should be incorporated into the design.

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APPENDIX A_Z

STRATEGY FOR OWNER DECISION-MAKING ON SEISMIC DESIGN OF BURIED STRUCTURES

This appendix provides a strategy for Owners to use when deciding whether seismic design of a buried structure should be performed. This strategy is to be used with Section Z of the Specifications and Commentaries prepared as part of the NCHRP 12-70 Project *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments* (NCHRP, 2008).

Background

The Specifications and Commentaries prepared for the NCHRP 12-70 Project include provisions for transient ground displacements (TGD) and permanent ground displacements (PGD) of buried structures. These provisions apply to buried structures for water conveyance, utilities, and pedestrian structures constructed by embankment or trench methods. The provisions do not apply for tunnels constructed for vehicular traffic.

Generally seismic design for buried structures should be considered in those situations that represent a life safety issue or potentially have significant direct or indirect economic impacts. A life safety issue might involve collapse of a heavily used pedestrian tunnel that results from racking of the structure walls. An example of a significant direct impact might be collapse of a water conveyance pipe below a roadway embankment that results in very high repair costs. An indirect impact might be collapse of a drainage culvert required to drain water from behind a roadway embankment – resulting in a large volume of water collecting behind the embankment and eventually leading to failure of the embankment.

The implications of failure and the requirements for repair differ for TGD and PGD, and these differences need to be considered when deciding whether there is a need for seismic design:

- Transient ground displacements (TGDs) involve displacements that occur during the earthquake. By following the methods described in Section Z of the Specifications, the buried structure can be designed to handle the transient strains associated with earthquake loading. In some cases structures not designed for TGD could collapse during a design earthquake – either through ovaling or racking failures, and would have to be replaced or to repair the damage. This could mean excavating through a roadway embankment or mobilizing a tunnel boring machine. Either methods of repair could be very expensive.
- A buried structure designed for TGDs may still fail or be damaged during an earthquake if permanent ground displacement (PGD) occurs. The PGDs result from permanent movement of the earth associated with liquefaction, seismic-induced slope failures, differential settlement, and fault displacement. It is much more difficult to design for PGDs because of the large earth forces that are associated with permanent ground displacement. Procedures given in Section Y and NCEER (1997) can be used to evaluate the potential for and magnitude of these permanent ground movements. The normal assumption is that the buried structure will move with the ground. To mitigate the potential for PGD, it is usually

necessary to use ground improvement methods to reduce the potential for ground movement – or sometimes a utility or pipe can be placed in a carrier pipe that protects the utility or drainage system from ground displacement.

The Specifications and Commentaries to the NCHRP 12-70 Project leave the decision on need for seismic design of the buried structures to the Owner, who must weigh a number of factors in reaching this decision. Typically, small amounts of transient or permanent ground movement are acceptable; however, there are situations where even this level of deformation may be unacceptable. When deciding whether the buried structure should be designed for TGD and, in some cases, whether the buried structure should be designed for PGD, the factors summarized in the following section should be considered.

Considerations for Requiring Seismic Design

The factors that should be considered when deciding on the need for a seismic design range from implications of failure to the cost of repair. In most situations the decision will have to be made on a site-specific basis, since the need for seismic design will depend on the geometry of the site, the types of soils, the consequences of failure, and the method of repair.

Location and Function

One of the main factors for deciding on the need for a seismic design involves the location and function of the buried structure:

- Buried structures that are used by pedestrians could involve significant life safety issues. For these structures designs for TGD and PGD are often essential. Likewise, a culvert or utility that has a critical lifeline function, such as providing water for fire suppression or a domestic water supply, should usually be designed for PGD and TGD..
- Buried structures whose failure could affect the stability of a heavily traveled roadway should be designed for TGD and PGD. If the buried structure is located beneath a less traveled roadway, the need for seismic design decreases. In this case it might be less costly to repair the damaged structure than design to handle the TDG or PGD.
- If the buried structure is located such that it will be hard to repair in the event of damage, more consideration should be given to designing the structure for PGD and TGD, particularly in situations where the buried structure has a critical function.

Type of Soil

The type of soil at a site also should be considered when establishing the need for a seismic design of the buried structure. This consideration is related to both the type of TGD and PGD. The implications of the failure are a critical consideration for these soil-related factors:

- Buried structures located on steep slopes are more vulnerable to PGD during a design earthquake, and therefore the risk of damage or collapse increases. Likewise buried structures locate in areas where lateral spreading from liquefaction could occur are at risk. If the function of the buried structure is critical, then the need for PGD design increases.

- Buried structures that are constructed through soils having large stiffness contrasts are more vulnerable to damage. An example of this involves a buried structure that goes from a relatively compliant soil to rock. Sharp changes in the direction of the buried structure can also result in similar flexibility issues. If the function of the buried structure is critical, then the need for PGD design increases.

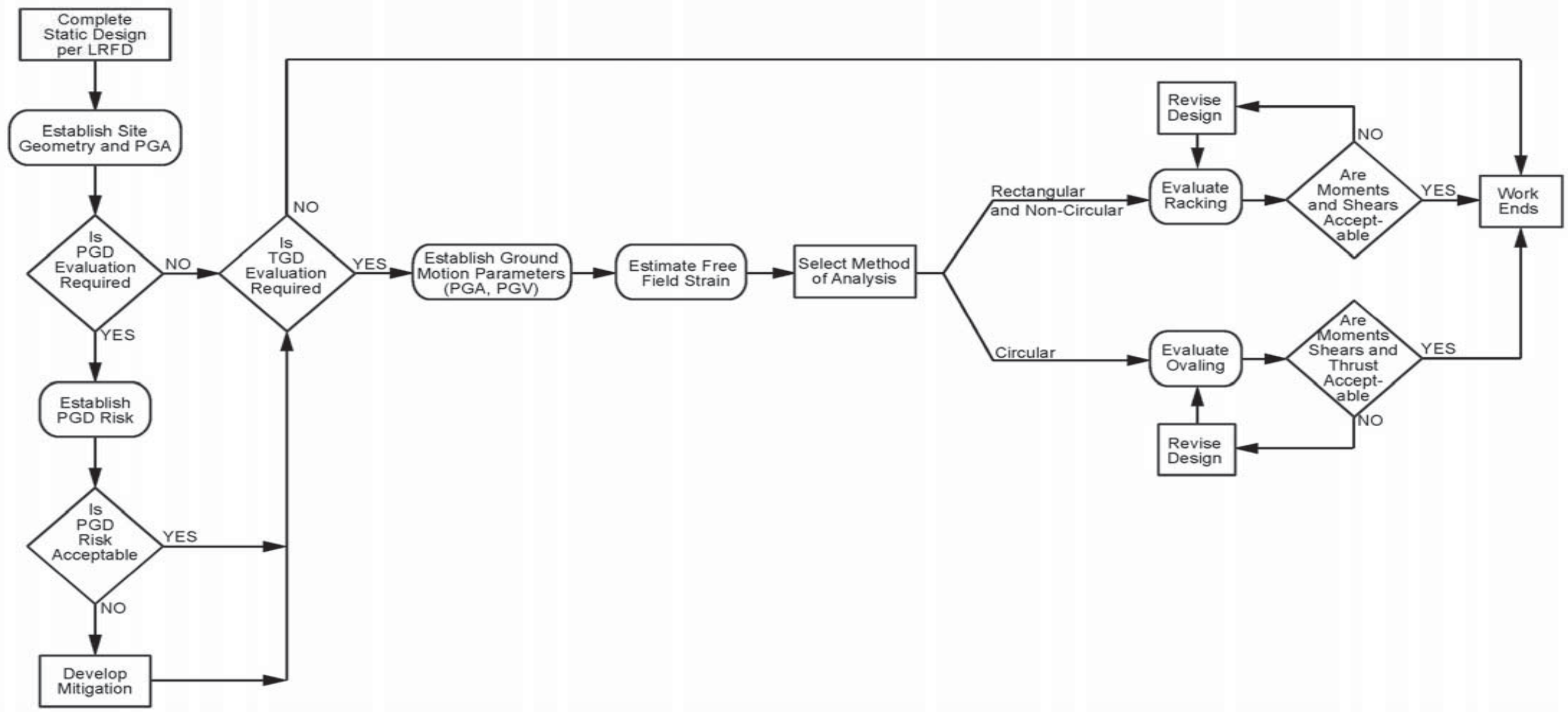
Implications of Wall Movement

Perhaps the easiest consideration to understand is the effects that failure of the buried structure will have on other facilities in proximity to the structure. Examples of these effects are summarized below.

- If the failure of the buried structure could result in undermining of a bridge foundation or retaining wall because the drainage function of the pipe is lost, then the need for TGD and PGD will be higher.
- If failure of the buried structure could result in significant environmental damage, the need for TGD and PGD design increases. For example, if the failure of a large drainage culvert results in excessive sedimentation in a nearby stream, a seismic design may be required.

Approach for Conducting Seismic Design of Buried Structures

Figure A_Z-1 shows the steps that the Owner might use in conjunction with the Specifications and Commentaries to carry-out a seismic design.



DEFINITIONS:

PGD = Peak Ground Displacement

TGD = Transient Ground Displacement

FIGURE Z-1
DESIGN SEQUENCE FOR BURIED STRUCTURES – SEISMIC CASE
NAS SEISMIC ANALYSIS

EP062007001RDD_05 (6/5/07)

Part 2
Example Problems
June 2008

Example Semi-Gravity Retaining Wall Problem

Introduction

The following example demonstrates the application of the proposed seismic design procedure outlined in Section X.7 of the proposed Specifications for the seismic design of conventional semi-gravity retaining walls. The example involves a typical semi-gravity cantilever wall used by DOTs.

For this example the wall was evaluated for three seismic regions, two backfill configurations, and two site soil types. Horizontal and sloped backfill slopes were examined. In one case the wall represented a typical cut-slope and backfill condition, while a second represented a fill condition. Variations in the initial wall dimensions were made during design in order to satisfy design requirements in the static AASHTO *LRFD Bridge Design Specifications* and the proposed Specifications. The active and passive lateral earth pressure coefficients for static and seismic cases were determined using limit equilibrium method, as discussed in the proposed Specifications. Small permanent displacement of the wall was considered permissible, thereby allowing reduction in the seismic coefficient to half its peak value for design.

The following subsections summarize (1) the wall geometry and soil properties used in the examples, (2) the seismicity for the three sites considered, (3) the active and passive earth pressure coefficients, (4) the general methodology followed, (5) the results of the retaining wall analyses, and (6) observations from these analyses. Information from these analyses was used to develop a step-by-step presentation of the example for one of the cases.

Wall Geometry and Soil Properties

The wall geometry consists of a 20-foot high semi-gravity standard cantilever wall with a basic wall geometry shown in Figure 1. The initial dimensions shown in Figure 1 were based on the Standard Plans currently used by Caltrans.

Two different foundation soil-configurations were evaluated. These two configurations are shown in Figures 2 and 3, respectively.

- In Case A the native soil comprises cohesive soil with a friction angle of 10° and a cohesion of 4,000 psf (assumed AASHTO Soil Type C). The native ground will be excavated at a temporary slope of 1:1, as shown on Figure 2, and the wall will be backfilled with granular material with an internal friction angle of 33° and unit weight of 120 pcf. The granular backfill was assigned a cohesion value of either 0 or 200 psf for these analyses. The 200 psf was assumed to result from a small amount of capillarity within the soil. The ground above the top of the wall was assumed to be either flat or sloped at 2:1. This geometry represents a typical case where a roadway is being widened into an existing cut slope. The 2:1 slope above the wall is used to minimize the wall height.

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- In Case B the native soil comprises granular soil with a friction angle of 33° (assumed AASHTO Soil Type D). The native ground is flat, and the wall is backfilled with uniform granular material with an internal friction angle of 33° and unit weight of 120 pcf. The granular backfill was assigned a cohesion value of either 0 or 200 psf for these analyses. As noted above, the 200 psf was assumed to result from a small amount of capillarity in the soil. This example is typical of a widening project. Although an MSE wall might also be used in this case, economic factors led to the use of a semi-gravity wall.

The water table for all analyses was assumed to be well below the base of the wall.

Seismicity

Three sites with different levels of seismic activity were included in this study. Two of the sites are located in the Western United States (WUS), one in Los Angeles area and the other one in Seattle. The third site is located in Central and Eastern United State region (CEUS), in Charleston, South Carolina.

Peak Ground Accelerations (PGA) for each site were determined from USGS/AASHTO Seismic Design Parameters for 2006 AASHTO Seismic Guidelines. Seismic ground accelerations were calculated for an average return period of 1,000 years. PGA values were initially determined for bedrock (Soil Type B) and modified for the foundation Soil Types C and D. A summary of site locations and seismicity data is given in Table 1.

Active and Passive Earth Pressure Coefficients

Both active and passive earth pressure coefficients were calculated using limit equilibrium methods with a seismic coefficient of 50% of the site adjusted PGA (i.e., $k_{max} = F_{pga}$ PGA) as defined in the proposed Specifications. The active earth pressure was calculated based on the Coulomb method. For granular material closed-form Mononobe-Okabe solutions are available. For Case A fill, failure wedge angles were constrained by the high strength native soil. For soils with both cohesion and friction, a trial wedge method was used. For cases involving complicated geometries and non-homogeneous backfills the slope stability method, as discussed in the proposed Specifications, could be adopted.

The active pressure was assumed to have a triangular distribution for static loading. The resultant active force for the static case was applied at one-third point above the base of the wall. For the seismic case, a uniform distribution was assumed, and the resultant active load was applied at the mid-height of the wall. Active pressure was applied as a vertical line extending from the heel of the wall (back of the foundation), as shown in Figures 4 to 6. The soil-on-soil friction angle at this imaginary line was assumed to be the larger of (1) two-thirds of the internal friction angle of the soil, or (2) the slope of the backfill.

The passive earth pressure was calculated using a log-spiral failure surface. Other methods could have been used based on designer's judgment and level of sophistication that may be needed in

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design. Passive earth pressure was assumed to have a triangular distribution for both the static and seismic loadings. The resultant passive force was applied at one-third above the base of the wall.

For walls with foundation keys, two sliding scenarios were evaluated. These scenarios are shown on Figure 7. For the horizontal sliding plane, the passive pressure distribution extended to the bottom of the key. For the inclined sliding plane, the passive pressure distribution was extended to the bottom of the footing. The lesser of these two values was used for design.

Methodology

The wall was initially designed according to AASHTO *LRFD Bridge Design Specifications* for static loading and then checked for seismic loading using design recommendations in the proposed Specifications. Only external stability of the wall was addressed in this study. The external stability evaluations comprise sliding, foundation eccentricity, and bearing capacity checks. It was assumed that global stability checks had been performed, and stability was found to be acceptable under static and seismic loading.

For simplicity no live-load surcharge was considered in this study. It may be necessary to include some percentage of the live load (e.g., live load and a load factor of 0.5) in the analyses, depending on the particular wall geometry, roadway location, and daily use. Load combinations Strength I and Extreme Event I (earthquake) were evaluated. Other load combinations are not controlling the design of the wall.

Two sets of load factors for Strength I load combinations were used for initial static design of the retaining walls. One set induces the maximum eccentricity on the foundation, while the other set induces the maximum bearing pressure. These load combinations are differentiated by Strength I-a and Strength I-b designations. Load factors for these two combinations are summarized on Figures 4 and 5. Load factors of 1.0 were used for all loads in Extreme Event I load combination. These factors are shown on Figure 6.

It should be noted that according to the AASHTO design methodology the passive earth pressure is regarded as a resistance; hence the factors shown on Figures 4 to 6 for passive earth pressure are resistance factors, rather than load factors. In addition, the passive earth pressure was considered in sliding stability calculations only, and was neglected in eccentricity and bearing capacity checks. Load and resistance factors used in this study are summarized in Table 2.

A satisfactory design requires the wall to satisfy the criteria for eccentricity, sliding, and bearing capacity. The following eccentricity criteria, which are identical to AASHTO, were adopted in this study:

- $e / W \leq 1/6$ for Strength I Load Case on Soil
- $e / W \leq 1/4$ for Strength I Load Case on Rock
- $e / W \leq 1/3$ for Extreme Event I Load Case

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Bearing resistance was checked using the equations recommended in Section 10 of the AASHTO *LRFD Bridge Design Specifications*.

Analysis Cases and Results

Twenty cases were examined in the first stage of these analyses. These cases are summarized in Table 3. Results from these analyses are summarized in Table 4. Designs are judged satisfactory if capacity is greater than demand (i.e., C/D > 1.0) with an optimum design where the capacity to demand ratio is 1.0. The basic Caltrans foundation dimensions (Figure 1) were used initially; however, for some cases the foundation dimensions were increased in order to satisfy the design criteria. Some of the cases with sloping backfill did not satisfy the criteria even for the largest footing dimensions considered (W = 15 feet). These cases are identified in Table 4 and discussed below. Detailed calculations for one case (Case 1-a) are shown in Appendix A.

It is well known that walls with granular sloping backfills with zero cohesion have difficulty in satisfying seismic performance criteria, particularly the sliding criteria. The combination of cohesionless sloping backfill and high seismic acceleration may significantly increase the active earth pressure particularly if the fill extent is unconstrained by a cut slope. For example, the estimated active seismic earth pressure coefficient for case 2-a in Table 3 (2:1 sloping backfill, seismic coefficient of 0.3) was approximately 11, more than 25 times larger than static active earth pressure.

Due to large seismic active earth pressures, even with the largest footing dimensions examined (W = 15 ft), the wall design was not adequate for cases 2-a, 2-b, 2-c, 2-d, 2-e, 4-a and 4-c. The static performance for these cases, except 2-a and 2-c, is acceptable, and even for these two cases the static design criteria can be satisfied with minor adjustments to foundation key depth, as illustrated. The seismic design criteria are difficult to satisfy for these conventional wall designs, and could require the use of pile foundations. However, an acceptable design based on the owner's performance criteria may be achieved by adopting a displacement-based design, as noted in the proposed Specifications. As an example, the design of the retaining wall was refined for the above cases using a displacement-based design approach as discussed below.

Displacement-based Design

In order to estimate the permanent displacement of the walls during the earthquake, the yield acceleration for all cases was determined and is reported in Table 5. It should be noted that these values were calculated using the same passive pressure coefficients shown in Table 3. Corresponding displacements are also shown in Table 5.

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Newmark displacement was calculated using correlations in Section X.4.5 of the proposed Specifications:

$$\log(d) = -1.51 - 0.74 \log (k_y/k_{max}) + 3.27 \log (1 - k_y/k_{max}) - 0.8 \log (k_{max}) + 1.50 \log PGV) \quad (1)$$

where PGV can be estimated from the following equation:

$$PGV\ (in/sec) = 0.55\ F_v S_1 \quad (2)$$

A displacement-based design approach was used for cases 2-a, 2-b, 2-c, 2-d and 2-e. For this example, the foundation dimensions were increased (slightly larger footing and 2-foot key depth) until the static design criteria were satisfied, and ensured that for the seismic case that sliding occurred before foundation bearing failure. Then the yield acceleration (k_y) was calculated for the revised wall geometry. The wall was designed for k_y, instead of k_{max}, ensuring all seismic design criteria were satisfied for this level of acceleration. Since the actual ground accelerations were expected to exceed k_y during the seismic event, it was expected that the wall would undergo permanent displacements during the seismic event. The amount of displacement was calculated using Newmark displacement method.

The results of the design calculations are shown in Table 6. The Newmark displacements for all cases are shown in Table 7. The displacements for cases 2-d and 2-e are negligible, while cases 2-c and 2-d are marginal and might be acceptable under some circumstances. Case 2-a displacements appear to be too large and are unlikely to be acceptable.

Effect of Backfill Cohesion

It should be noted that contrary to the results obtained in the previous section, seismic performance of cantilever semi-gravity retaining walls with sloping backfills during past earthquakes has been generally satisfactory. This contradiction can be attributed to two design assumptions:

- assuming an infinite slope of homogeneous soil, and
- assuming cohesionless backfill.

In practice usually the backfill has a finite slope, is made in a cut slope (similar to Case 2 foundation configuration), and almost always has some cohesion.

In order to address this practical issue, the foundation configuration was examined in cases 3 and 4 (refer to Table 3). In order to investigate the impact of backfill cohesion on design, a second series of analyses was conducted. In these analyses, the design was repeated for sloping backfill with 200 psf cohesion. These cases are summarized in Table 7, and the results are shown in Table 8.

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Review of results in Table 8 shows that for practical conditions with a reasonable backfill cohesion of 200 psf, all cases except 2-a, 4-a, and 2-c satisfy the design criteria. Even among these cases, the 4-a and 2-c cases are reasonably close to an acceptable design, and a fully-conformant design can be achieved by minor modifications of the wall dimensions. For cases with very high accelerations and long sloping backfills (e.g., 2-a), a spread footing wall might not fully satisfy the design criteria and a pile foundation might be a better option. However, depending on the performance criteria, a displacement-based design for a spread footing might result in an acceptable design. As an example, case 2-a wall was redesigned using the displacement-based method. The k_y was determined to be 0.24, and for this acceleration the wall satisfied all design criteria except seismic sliding. Then Newmark displacements were computed for $k_{max} = 0.6$ and $k_y = 0.24$. The resulting displacements, as shown in Table 10, are in the range of 5 to 7 inches, and might be acceptable for a large number of practical cases.

Concluding Comments

The initial set of analyses shows the common problem encountered by designers, where it is difficult to design semi-gravity retaining walls if there is a backslope above the wall and if site-adjusted PGA values exceed about 0.3. The cut slope (Case A) was less critical than the fill slope (Case B), all other conditions being equal. This response shows the importance of the soil profile and the nature and geometry of native soils behind the wall.

When the analyses were repeated with a cohesion value of 200 psf included in the analysis, performance improved significantly, such that the only case that did not meet capacity to demand ratios was for a seismic coefficient of 0.6 with a 2:1 backslope with no cut slope. These results demonstrate the importance of taking advantage of the cohesion within any analyses.

A conclusion from this set of analyses is that particular attention has to be given to methods for quantifying the amount of cohesion that can be counted on during the seismic analysis. For existing slopes the effects of cohesion can be identified by collecting high quality samples and conducting laboratory tests. However, gaining acceptance for additional sampling and testing requirements may be difficult. A more acceptable approach would be to develop a relationship between an acceptable value of cohesion and, for example, the fines content (i.e., portion passing the No. 200 sieve) and the plasticity. This issue is even more difficult for apparent cohesion resulting from partial saturation. In this case climate and grain-size distribution would be expected to affect the apparent cohesion.

References

AASHTO (2007), “AASHTO *LRFD Bridge Design Specifications*, Customary U.S. Units,” 4th Edition.

RocScience (2007). “SLIDE: Stability Analysis for Soil and Rock Slopes.” www.rocscience.com.

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Table 1. Site Coordinates and Seismicity Data

Site Coordinates			Soil Type B/C		Soil Type C		Soil Type D	
Longitude	Latitude	Region	PGA	S_1	F_{pga} PGA	F_v S_1	F_{pga} PGA	F_v S_1
-117.9750	34.0500	WUS (Los Angeles)	0.600	0.521	0.600	0.677	0.600	0.782
-122.2500	47.2700	WUS (Seattle)	0.400	0.296	0.420	0.443	0.460	0.535
-079.2370	33.1000	CEUS (Charleston)	0.200	0.099	0.240	0.168	0.298	0.237

Table 2. Load Factors and Resistance Factors

Load	Load Factor		
	Strength I-a	**Strength I-b**	**Extreme Event I**
EAH Active earth pressure, horizontal component	1.50	0.90	1.00
EAV Active earth pressure, vertical component	1.00	1.35	1.00
EV Vertical soil pressure	1.00	1.35	1.00
DC Dead load of structural components	0.90	1.25	1.00
Sliding	**Resistance Factor[1]**		
	Strength I-a	**Strength I-b**	**Extreme Event I**
EPH Passive earth pressure, horizontal component	0.50	0.50	1.00
EPV Passive earth pressure, vertical component	0.50	0.50	1.00
Cohesion, c	0.80	0.80	1.00
Friction angle, ϕ	0.80	0.80	1.00
Bearing Capacity	**Resistance Factor[1]**		
	Strength I-a	**Strength I-b**	**Extreme Event I**
Cohesion, c	0.60	0.60	1.00
Friction angle, ϕ	0.55	0.55	1.00

1. Resistance factor for earth material in AASHTO depends on soil investigation method. Here it was assumed shear strength parameters were estimated from a field exploration program.

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Table 3. Summary of Analyses Cases for Granular Backfill ($\phi = 33°$ and $c = 0$)

Analysis	Backfill Slope	Foundation	PGA	Site Soil Profile	Surface k_{max}	k_h	k_a	k_{ae}	k_p	k_{pe} [g]
1-a	Hor.	Type B	0.6	C or D	0.600	0.300	0.264	0.518	5.700	4.750
2-a	2:1	Type B	0.6	C or D	0.600	0.300	0.433	10.925	5.700	4.750
3-a	Hor.	Type A	0.6	C or D	0.600	0.300	0.264	0.509	5.70/19.81	4.75/19.8[1]
4-a	2:1	Type A	0.6	C or D	0.600	0.300	0.433	1.036	5.70/19.81	4.75/19.8[1]
1-b	Hor.	Type B	0.4	C	0.420	0.210	0.264	0.420	5.700	5.070
2-b	2:1	Type B	0.4	C	0.420	0.210	0.433	6.125	5.700	5.070
3-b	Hor.	Type A	0.4	C	0.420	0.210	0.264	0.420	5.70/19.8[1]	5.07/19.8[1]
4-b	2:1	Type A	0.4	C	0.420	0.210	0.433	0.854	5.70/19.8[1]	5.07/19.8[1]
1-c	Hor.	Type B	0.4	D	0.460	0.230	0.264	0.440	5.700	5.000
2-c	2:1	Type B	0.4	D	0.460	0.230	0.433	7.192	5.700	5.000
3-c	Hor.	Type A	0.4	D	0.460	0.230	0.264	0.440	5.70/19.8[1]	5.00/19.8[1]
4-c	2:1	Type A	0.4	D	0.460	0.230	0.433	0.895	5.70/19.8[1]	5.00/19.8[1]
1-d	Hor.	Type B	0.2	C	0.240	0.120	0.264	0.344	5.700	5.360
2-d	2:1	Type B	0.2	C	0.240	0.120	0.433	1.326	5.700	5.360
3-d	Hor.	Type A	0.2	C	0.240	0.120	0.264	0.344	5.70/19.8[1]	5.36/19.8[1]
4-d	2:1	Type A	0.2	C	0.240	0.120	0.433	0.672	5.70/19.8[1]	5.36/19.8[1]
1-e	Hor.	Type B	0.2	D	0.298	0.149	0.264	0.366	5.700	5.270
2-e	2:1	Type B	0.2	D	0.298	0.149	0.433	2.872	5.700	5.270
3-e	Hor.	Type A	0.2	D	0.298	0.149	0.264	0.366	5.70/19.8[1]	5.27/19.8[1]
4-e	2:1	Type A	0.2	D	0.298	0.149	0.433	0.731	5.70/19.8[1]	5.27/19.8[1]

1. Passive pressure coefficient for footing and key, respectively.
2. Shaded k_{ae} values indicate combination of slope angle, soil properties, and acceleration level that resulted in a very large inertial mass. These seismic coefficients exceed levels that can be used in design. Either an alternate method of determining k_{ae} is required or the geometry of the wall needs to be revised.

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Table 4. Summary of Analyses Results for Granular Backfill (ϕ = 33° and c = 0)

	Footing Size		Capacity/Demand: Strength I-a				Capacity/Demand: Strength I-b				Capacity/Demand: Extreme Event I			
Analysis	Width W [ft]	Toe Length C [ft]	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined
1-a	13.50	6.08	0.015	4.80	1.55	1.74	-0.103	4.28	3.35	3.86	0.318	1.58	1.14	1.21
2-a	15.00	7.58	0.114	1.28	0.89[1]	1.00	-0.092	3.29	1.95	2.23	1.008[1]	n/a[2]	0.37[1]	0.43[1]
3-a	13.50	6.08	0.015	64.44	5.14	3.41	-0.103	37.89	7.52	6.08	0.315	6.37	1.90	2.22
4-a	15.00	7.58	0.114	44.65	2.27	1.84	-0.092	34.97	4.08	3.29	0.471[1]	0.77	0.58[1]	1.25
1-b	12.00	4.58	0.066	3.37	1.52	1.74	-0.071	4.10	3.28	3.86	0.306	1.84	1.43	1.54
2-b	15.00	7.58	0.114	1.28	0.89[1]	1.00	-0.092	3.29	1.95	2.23	0.909[1]	n/a[2]	0.42[1]	0.48[1]
3-b	12.00	4.58	0.066	51.81	4.28	3.13	-0.071	37.67	7.29	5.63	0.306	6.71	2.36	2.63
4-b	15.00	7.58	0.114	44.65	2.27	1.84	-0.092	34.97	4.08	3.29	0.382[1]	3.43	1.03	1.54
1-c	12.00	4.58	0.066	3.37	1.52	1.74	-0.071	4.10	3.28	3.86	0.330	1.48	1.35	1.45
2-c	15.00	7.58	0.114	1.28	0.89[1]	1.00	-0.092	3.29	1.95	2.23	0.940[1]	n/a[2]	0.40[1]	0.46[1]
3-c	12.00	4.58	0.066	51.81	4.28	3.13	-0.071	37.67	7.29	5.63	0.330	5.83	2.08	2.47
4-c	15.00	7.58	0.114	44.65	2.27	1.84	-0.092	34.97	4.08	3.29	0.403[1]	2.77	0.91[1]	1.47
1-d	11.00	3.58	0.110	2.53	1.49	1.74	-0.042	4.01	3.23	3.87	0.259	2.61	1.89	2.03
2-d	15.00	7.58	0.114	1.28	0.89[1]	1.00	-0.092	3.29	1.95	2.23	0.477[1]	0.76[1]	0.76[1]	0.83[1]
3-d	11.00	3.58	0.110	43.12	3.71	2.95	-0.042	37.69	7.15	5.34	0.259	8.10	3.41	3.34
4-d	14.00	6.58	0.152	37.59	1.99	1.75	-0.071	34.87	4.01	3.14	0.330	5.09	1.55	1.93
1-e	11.00	3.58	0.110	2.53	1.49	1.74	-0.042	4.01	3.23	3.87	0.296	1.98	1.71	1.85
2-e	15.00	7.58	0.114	1.28	0.89[1]	1.00	-0.092	3.29	1.95	2.23	0.719[1]	n/a[2]	0.53[1]	0.59[1]
3-e	11.00	3.58	0.110	43.12	3.71	2.95	-0.042	37.69	7.15	5.34	0.296	6.80	2.83	3.02
4-e	15.00	7.58	0.114	44.65	2.27	1.84	-0.092	34.97	4.08	3.29	0.313	5.74	1.53	1.84

1. Design criteria not satisfied.
2. Bearing capacity could not be calculated due to large eccentricity.

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Table 5. Yield Acceleration for Walls with Granular Backfill ($\phi = 33°$ and $c = 0$)

Analysis	Backfill Slope	Foundation	Width W [ft]	Toe Length C [ft]	Key Depth [ft]	k_y	Newmark Displacement [in][1]
1-a[1]	Hor.	Type B	13.50	6.08	1.0	0.355	1.2
2-a[1]	2:1	Type B	15.00	7.58	1.0	0.107	27.6
3-a[1]	Hor.	Type A	13.50	6.08	1.0	0.420	0.4
4-a[1]	2:1	Type A	15.00	7.58	1.0	0.214	7.4
1-a[2]	Hor.	Type B	13.50	6.08	1.0	0.355	1.5
2-a[2]	2:1	Type B	15.00	7.58	1.0	0.107	34.7
3-a[2]	Hor.	Type A	13.50	6.08	1.0	0.420	0.5
4-a[2]	2:1	Type A	15.00	7.58	1.0	0.214	9.3
1-b	Hor.	Type B	12.00	4.58	1.0	0.359	0.0
2-b	2:1	Type B	15.00	7.58	1.0	0.107	10.4
3-b	Hor.	Type A	12.00	4.58	1.0	0.344	0.0
4-b	2:1	Type A	15.00	7.58	1.0	0.214	1.6
1-c	Hor.	Type B	12.00	4.58	1.0	0.359	0.1
2-c	2:1	Type B	15.00	7.58	1.0	0.107	15.4
3-c	Hor.	Type A	12.00	4.58	1.0	0.344	0.2
4-c	2:1	Type A	15.00	7.58	1.0	0.214	2.8
1-d	Hor.	Type B	11.00	3.58	1.0	0.360	0
2-d	2:1	Type B	15.00	7.58	1.0	0.107	0.9
3-d	Hor.	Type A	11.00	3.58	1.0	0.295	0
4-d	2:1	Type A	14.00	6.58	1.0	0.180	0.0
1-e	Hor.	Type B	11.00	3.58	1.0	0.360	0
2-e	2:1	Type B	15.00	7.58	1.0	0.107	2.4
3-e	Hor.	Type A	11.00	3.58	1.0	0.295	0.0
4-e	2:1	Type A	15.00	7.58	1.0	0.214	0.1

Site Type C. Site Type D.

[1] Accuracy of Newmark method does not support displacement estimates to less than an inch. Results are shown to 1 decimal point for comparative purposes only.

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Table 6. Summary of Analyses Results for Cases 2a to 2e, Revised Dimensions, Granular Backfill ($\phi = 33°$ and c = 0)

	Footing Size				Capacity/Demand: Strength I-a				Capacity/Demand: Strength I-b				Capacity/Demand: Extreme Event I[1]			
Analysis	Width W [ft]	Toe Length C [ft]	Key Depth [ft]	Yield Acc. k_y [g]	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined
2-a 2-b 2-c 2-d 2-e	17.00	8.58	2.00	0.114	0.056	2.06	1.01	1.20	-0.093	3.81	2.08	2.55	0.292	1.03	1.00	1.10

1. Seismic Capacity/Demand ratios calculated for yield acceleration.

Table 7. Newmark Displacement for Cases 2-a to 2-e, Revised Dimensions, Granular Backfill ($\phi = 33°$ and c = 0)

Analysis	Region	Site Soil Profile	k_{max}	F_v S_1	PGV (in/sec)	Newmark Displacement[2] (in)
2-a	WUS (Los Angeles)	C	0.600	0.677	37.24	25.1
2-a	WUS (Los Angeles)	D	0.600	0.782	43.01	31.6
2-b	WUS (Seattle)	C	0.420	0.443	24.37	9.2
2-c	WUS (Seattle)	D	0.460	0.535	29.43	13.8
2-d	CEUS (Charleston)	C	0.240	0.168	9.24	0.7
2-e	CEUS (Charleston)	D	0.298	0.237	13.04	2.0

[2] Accuracy of Newmark method does not support displacement estimates to less than an inch. Results are shown to 1 decimal point for comparative purposes only.

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Table 8. Summary of Analyses Cases for Cohesive Backfill (ϕ = 33° and c = 200)

Analysis	Backfill Slope	Foundation	PGA	Site Soil Profile	k_{max}	k_h	k_a	k_{ae}	k_p	k_{pe} [g]
2-a	2:1	Type B	0.6	C or D	0.600	0.300	0.192	4.086	10.000	9.310
4-a	2:1	Type A	0.6	C or D	0.600	0.300	0.192	0.705	10.0/19.8[1]	9.31/19.8[1]
2-b	2:1	Type B	0.4	C	0.420	0.210	0.192	0.588	10.000	9.560
4-b	2:1	Type A	0.4	C	0.420	0.210	0.192	0.523	10.0/19.8[1]	9.56/19.8[1]
2-c	2:1	Type B	0.4	D	0.460	0.230	0.192	0.714	10.000	9.500
4-c	2:1	Type A	0.4	D	0.460	0.230	0.192	0.564	10.0/19.8[1]	9.50/19.8[1]
2-d	2:1	Type B	0.2	C	0.240	0.120	0.192	0.344	10.000	9.790
4-d	2:1	Type A	0.2	C	0.240	0.120	0.192	0.344	10.0/19.8[1]	9.79/19.8[1]
2-e	2:1	Type B	0.2	D	0.298	0.149	0.192	0.401	10.000	9.710
4-e	2:1	Type A	0.2	D	0.298	0.149	0.192	0.400	10.0/19.8[1]	9.71/19.8[1]

1. Passive pressure coefficient for footing and key, respectively.
2. Shaded k_{ae} value indicates combination of slope angle, soil properties, and acceleration level that resulted in a very large inertial mass. This seismic coefficient exceeds levels that can be used in design. Either an alternate method of determining k_{ae} is required or the geometry of the wall needs to be revised.

Table 9. Summary of Analyses Results for Cohesive Backfill (ϕ = 33° and c = 200 psf)

	Footing Size		Capacity/Demand: Strength I-a				Capacity/Demand: Strength I-b				Capacity/Demand: Extreme Event I			
Analysis	Width W [ft]	Toe Length C [ft]	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined	Eccentricity e/W	Bearing	Sliding Horizontal	Sliding Inclined
2-a	15.00	7.58	-0.034	14.07	2.25	2.27	-0.138	6.99	4.57	4.88	0.838[1]	n/a[2]	0.47[1]	0.54[1]
4-a	15.00	7.58	-0.034	63.05	6.04	4.21	-0.138	34.97	8.55	7.45	0.371[1]	3.99	1.25	1.75
2-b	14.25	6.83	-0.016	13.96	2.22	2.25	-0.127	6.86	4.51	4.86	0.325	4.45	1.33	1.36
4-b	13.75	6.33	-0.003	63.15	5.93	3.94	-0.119	34.57	8.33	7.03	0.323	5.62	1.88	2.24
2-c	15.00	7.58	-0.034	14.07	2.25	2.27	-0.138	6.99	4.57	4.88	0.343[1]	3.83	1.16	1.19
4-c	14.25	6.83	-0.016	63.08	5.97	4.05	-0.127	34.71	8.42	7.19	0.324	5.61	1.76	2.12
2-d	11.00	3.58	0.097	7.59	2.07	2.22	-0.055	6.47	4.32	4.81	0.330	3.68	2.04	2.10
4-d	11.25	3.83	0.085	43.88	4.44	3.44	-0.062	34.26	7.93	6.22	0.312	5.66	2.75	3.07
2-e	12.00	4.58	0.054	9.84	2.12	2.83	-0.082	6.56	4.38	4.81	0.324	4.09	1.78	1.82
4-e	12.00	4.58	0.054	50.04	4.91	3.59	-0.082	34.27	8.04	6.46	0.323	5.42	2.34	2.72

1. Design criteria not satisfied.
2. Bearing capacity could not be calculated due to large eccentricity.

Table 10 Newmark Displacement for Cases 2-a, Cohesive Backfill (ϕ = 33° and c = 200 psf)

Analysis	Region	Site Soil Profile	k_{max}	$F_v S_1$	PGV (in/sec)	k_y	Newmark Displacement[3] (in)
2-a	WUS (Los Angeles)	C	0.600	0.677	37.24	0.24	5.4
2-a	WUS (Los Angeles)	D	0.600	0.782	43.01	0.24	6.8

[3] Accuracy of Newmark method does not support displacement estimates to less than an inch. Results are shown to 1 decimal point for comparative purposes only.

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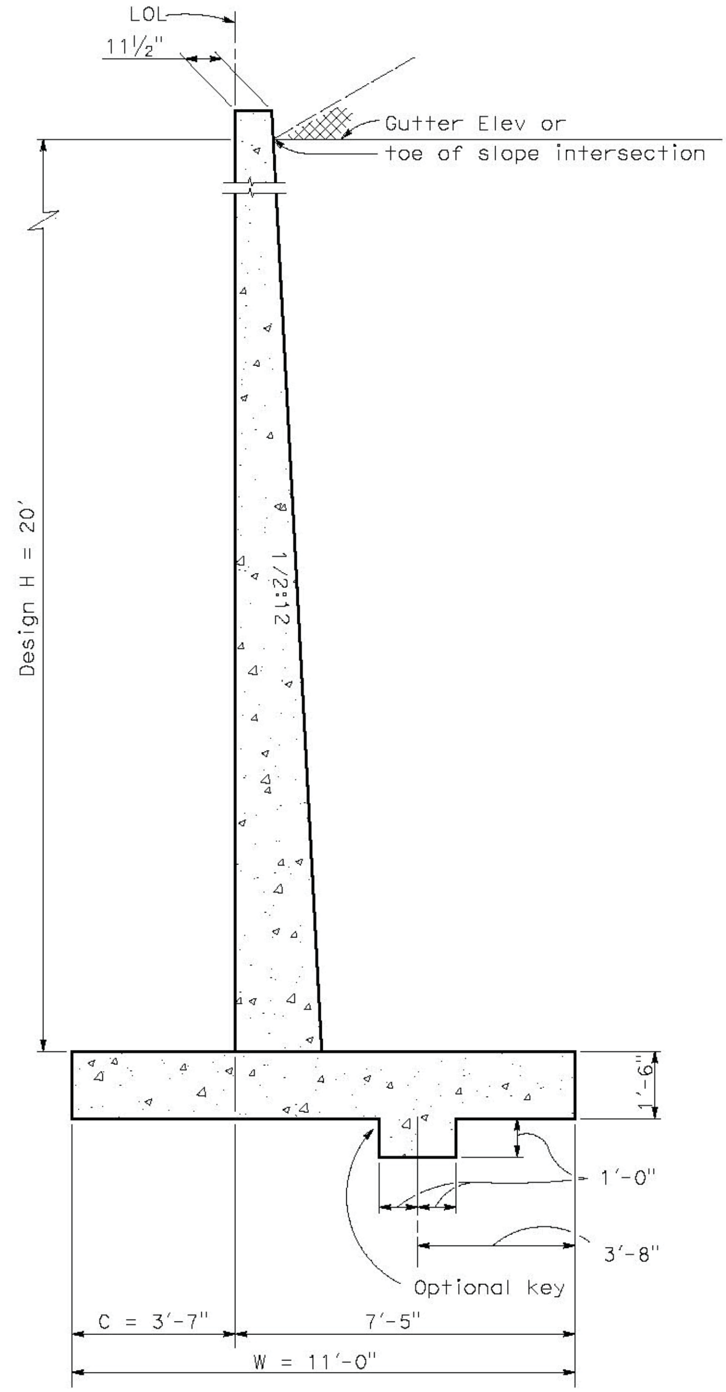

Figure 1. Basic Wall Geometry

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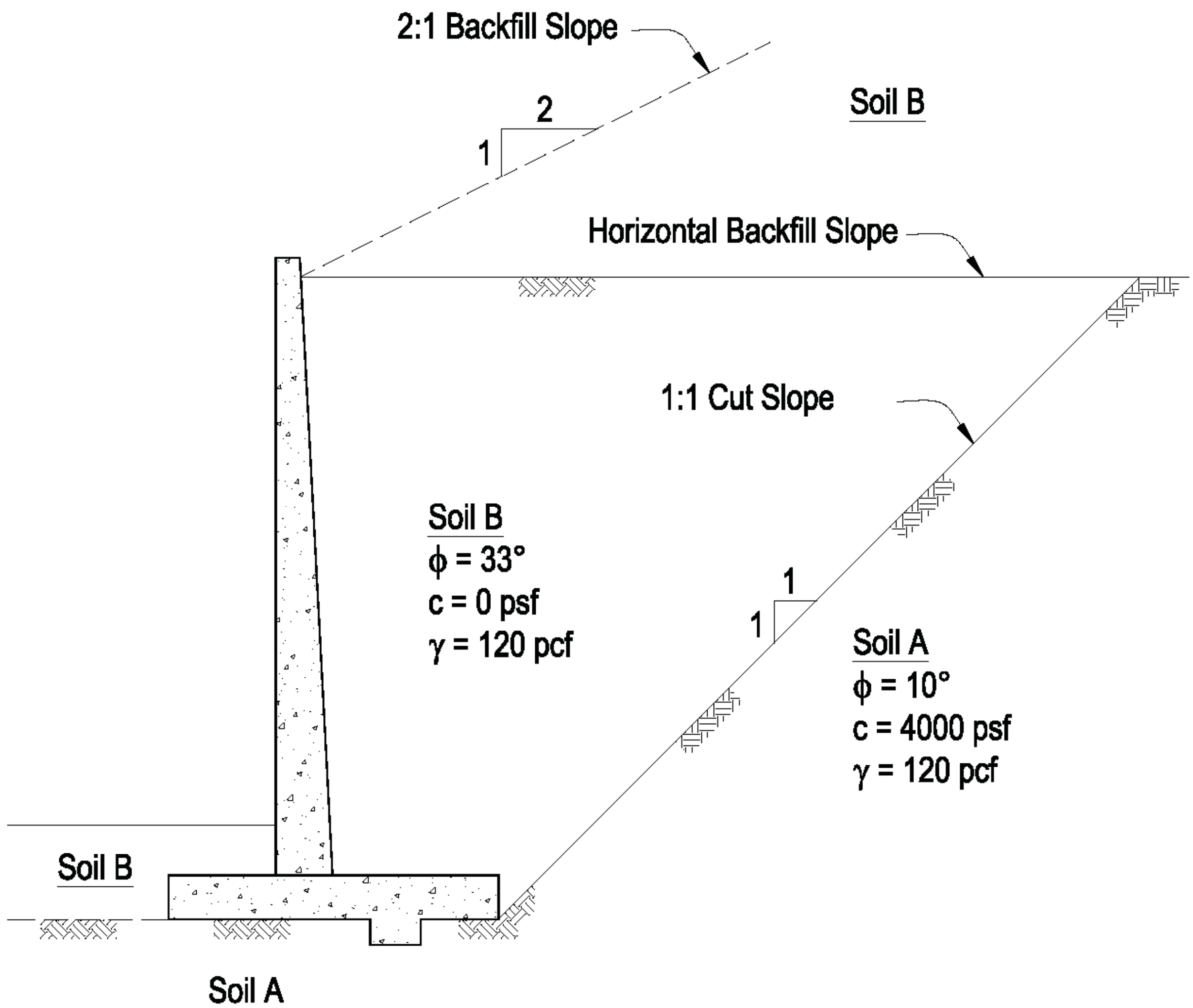

Figure 2. Backfill Configuration Type A

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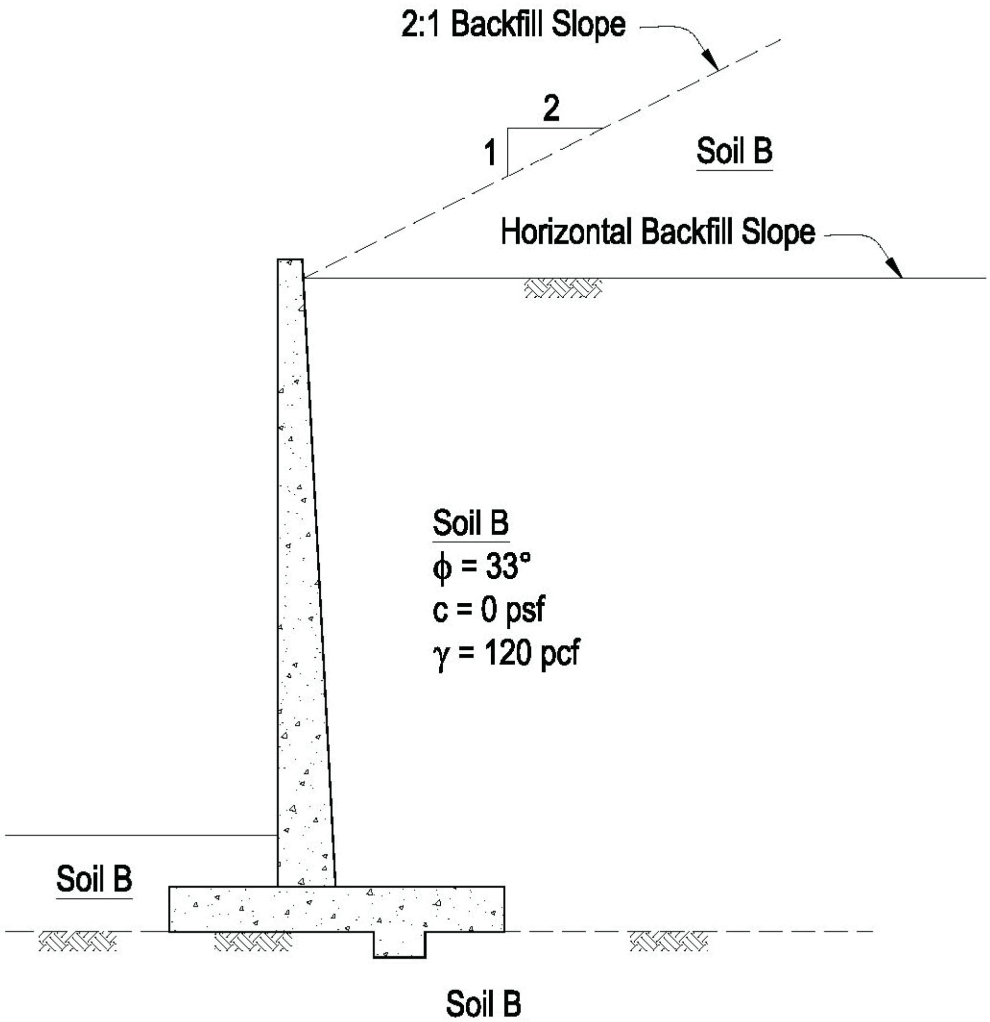

Figure 3. Backfill Configuration Type B

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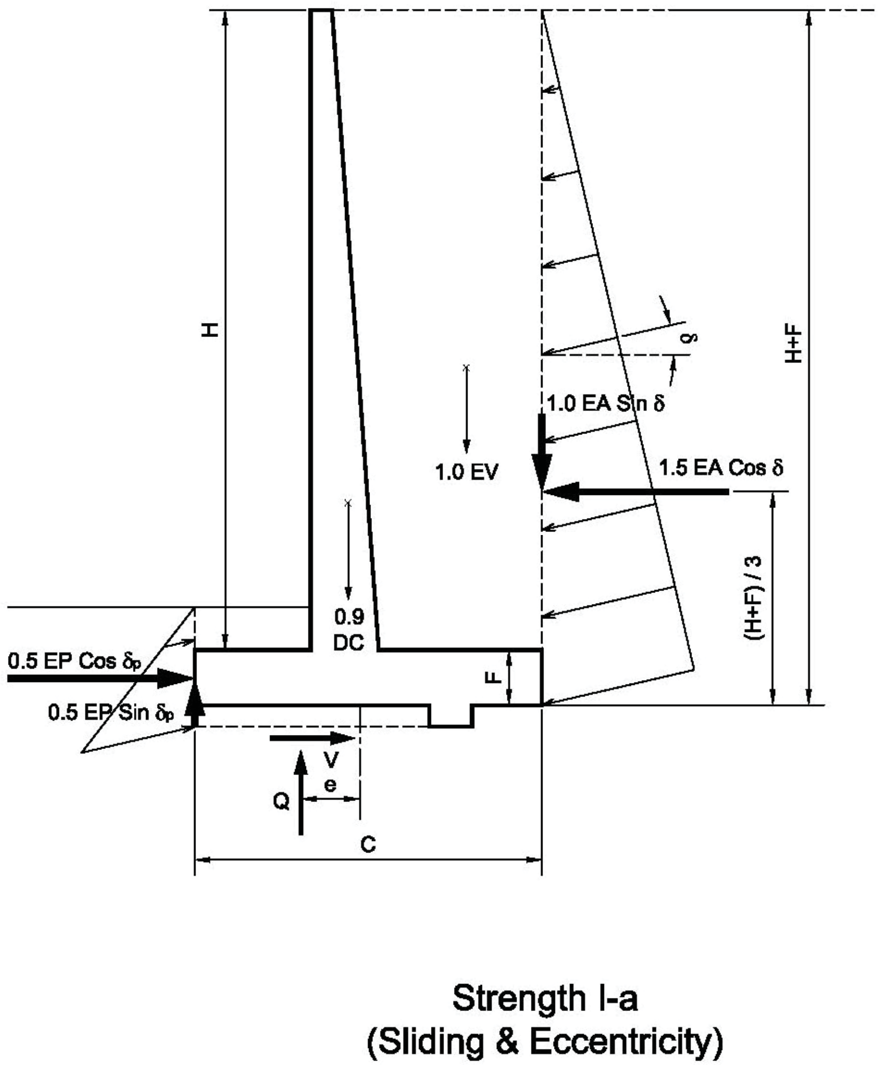

Figure 4. Load Combination Strength I-a

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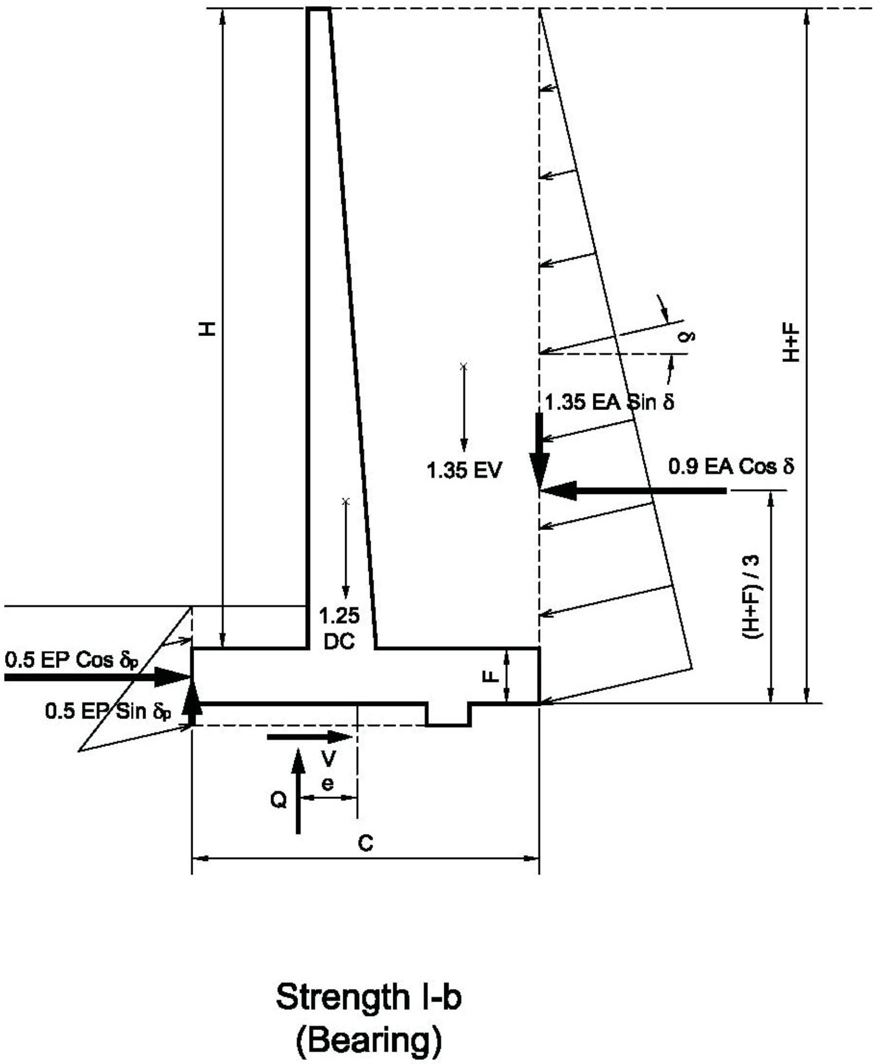

Figure 5. Load Combination Strength I-b

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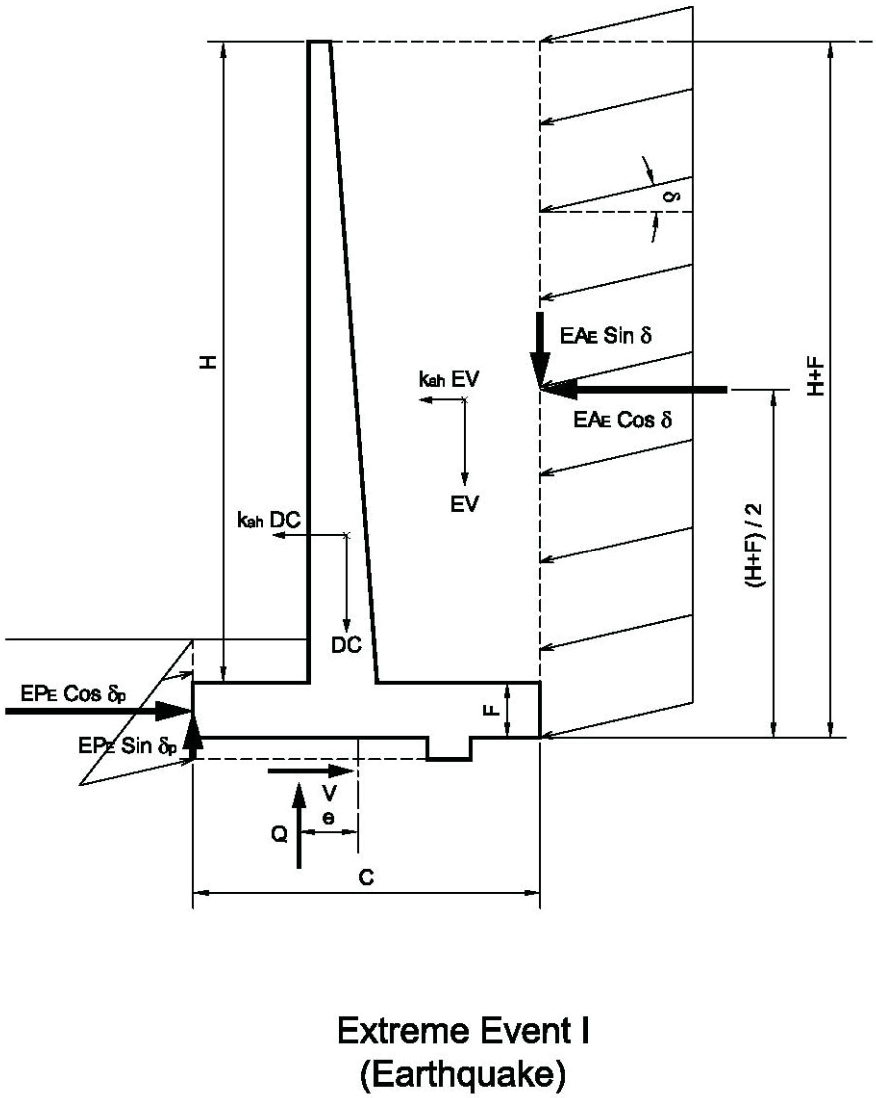

Figure 6. Load Combination Extreme Event I (earthquake)

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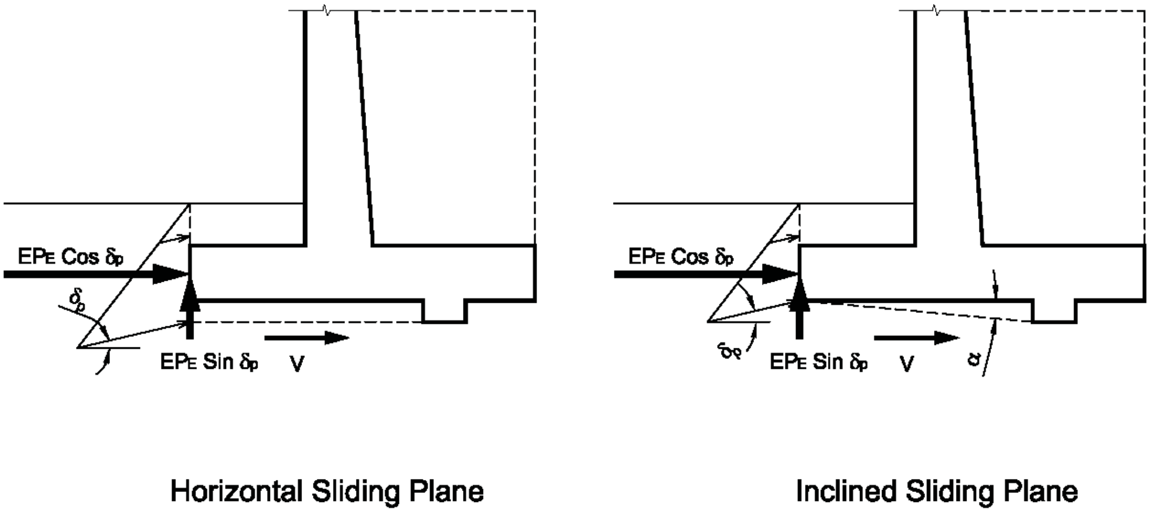

Figure 7. Passive Resistance on Footings with Key

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Appendix A: Summary of Wall Stability Calculations for Case 1-a

Earth Pressure Coefficients (from Table 3):

Analysis	k_h	k_a	k_{ae}	k_p	k_{pe}
1-a	0.300	0.264	0.518	5.700	4.750

Material Unit Weights:

Material	Unit Weight [pcf]
Backfill Soil	145
Wall (concrete)	120
Foundation Soil	120
Foundation Cover Soil	120

Other Assumptions:

- Footing cover: 2 ft
- Soil-soil friction angle (d) for active pressure: 22°
- Soil-concrete friction angle for passive pressure: 16.5°
- Soil-concrete friction angle for footing sliding: 33°

Load Case: Strength I-a:
Unfactored Weights and Pressures:

Weights			
Component	**Area [ft^2]**	**Load [lbs/ft]**	**Load Value [lbs/ft]**
Wall weight (including footing and key)	50.58	50.58×145	7,335
Backfill soil weight	120.07	120.07×120	14,408
Footing cover soil weight	12.16	12.16×120	1,459
Pressures			
Component	**Pressure Length [ft]**	**Load [lbs/ft]**	**Load Value [lbs/ft]**
Horizontal active earth pressure	21.5	$0.264\times120\times21.5^2/2\times\text{Cos } 22°$	6,789
Vertical active earth pressure	21.5	$0.264\times120\times21.5^2/2\times\text{Sin } 22°$	2,743
Horizontal passive earth pressure on footing	3.5	$5.70\times120\times3.5^2/2\times\text{Cos } 16.5°$	4,017
Vertical passive earth pressure on footing	3.5	$5.70\times120\times3.5^2/2\times\text{Sin } 16.5°$	1,190
Horizontal passive pressure on key	1.0	$5.70\times(120\times3.5\times1.0+120\times1.0^2/2)\times\text{Cos } 16.5°$	2,623
Vertical passive pressure on key	1.0	$5.70\times(120\times3.5\times1.0+120\times1.0^2/2)\times\text{Sin } 16.5°$	777

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Foundation Reactions:

Note that passive earth pressure is not used in calculating foundation reactions.

Vertical Loads				
Component	**Load [ft^2]**	**Load Factor**	**Moment Arm about Toe [ft]**	**Moment [lbs.ft/ft]**
Wall weight (including footing and key)	7,335	0.9	6.87	45,352
Backfill soil weight	14,408	1.0	10.494	151,197
Footing cover soil weight	1,459	1.0	3.04	4,435
Vertical active earth pressure	2,743	1.0	13.50	37,031
Sum of Factored Vertical Loads	**25,211**			
Horizontal Loads				
Component	**Load [ft^2]**	**Load Factor**	**Moment Arm about Toe [ft]**	**Moment [lbs.ft/ft]**
Horizontal active earth pressure	6,789	1.5	7.167	-72,980
Sum of Factored Horizontal Loads	**10,184**			
Sum of Factored Moments				**165,037**

Q = sum of vertical forces = 25,211 lb/ft
V = sum of horizontal forces = 10,184 lb/ft
M = sum of moments = 165,037 lb.ft/ft
e = foundation eccentricity = W/2-M/Q = 13.50/2-165,037/25,211 = 0.204 ft
e/W = 0.204 / 13.50 = 0.015 < 0.167

Bearing Capacity:

Using Vesic coefficients: $\phi \times q_{ult}$ = 9,239 psf
B' = B-2e = 13.5-2×0.204 = 13.09 ft
q_{demand} = 25,211 / 13.09 = 1,926 psf
Capacity / Demand =9,239 / 1,926 = 4.80 > 1.00

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Sliding Along Horizontal Plane:

Bottom of Footing Resistance					
Component	**Load [ft^2]**	**Load Factor**	**Resis-tance Factor**	**Factored Resisting Force [lbs/ft]**	**Force Value [lbs/ft]**
Vertical Load	25,211	n.a.	0.80	25211×Tan 33°×0.8	13,098
Vertical passive earth pressure on footing	1,190	0.5	0.80	-0.5×1190×Tan 33°×0.8	-309
Vertical passive pressure on key	777	0.5	0.80	-0.5×777×Tan 33°×0.8	-202
Sum of Factored Bottom of Footing Resistance					**12,587**
Passive Earth Pressure					
Component	**Pressure Length [ft]**	**Load Factor**	**Resis-tance Factor**	**Factored Passive Force [lbs/ft]**	**Force Value [lbs/ft]**
Horizontal passive earth pressure on footing	4,017	n.a.	0.5	0.5×4,017	2,009
Horizontal passive pressure on key	2,623	n.a.	0.5	0.5×2,623	1,311
Sum of Factored Passive Pressure Resistance					**3,320**

Sliding Factor of Safety:

Capacity / Demand = Sum of factored resisting forces / sum of factored driving forces
= (12587+3320)/(10184) = 1.56 > 1.00

Load Case: Strength I-b:

The calculations for this load case are identical to Strength I-a case except that different load factors shall be used.

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Load Case: Extreme Event I:
Unfactored Weights and Pressures:

Weights			
Component	**Area [ft^2]**	**Load [lbs/ft]**	**Load Value [lbs/ft]**
Wall weight (including footing and key)	50.58	50.58×145	7,335
Backfill soil weight	120.07	120.07×120	14,408
Footing cover soil weight	12.16	12.16×120	1,459
Earthquake Inertial Loads			
Component	**Area [ft^2]**	**Load [lbs/ft]**	**Load Value [lbs/ft]**
Inertial load on wall	50.58	0.3×50.58×145	2,200
Inertial load on backfill soil	120.07	0.3×120.07×120	4,322
Inertial load on footing cover soil	12.16	0.3×12.16×120	438
Pressures			
Component	**Pressure Length [ft]**	**Load [lbs/ft]**	**Load Value [lbs/ft]**
Horizontal active earth pressure	21.5	$0.518\times120\times21.5^2/2\times\text{Cos } 22°$	13,321
Vertical active earth pressure	21.5	$0.518\times120\times21.5^2/2\times\text{Sin } 22°$	5,382
Horizontal passive earth pressure on footing	3.5	$4.75\times120\times3.5^2/2\times\text{Cos } 16.5°$	3,347
Vertical passive earth pressure on footing	3.5	$4.75\times120\times3.5^2/2\times\text{Sin } 16.5°$	992
Horizontal passive pressure on key	1.0	$4.75\times(120\times3.5\times1.0+120\times1.0^2/2)\times\text{Cos } 16.5°$	2,186
Vertical passive pressure on key	1.0	$4.75\times(120\times3.5\times1.0+120\times1.0^2/2)\times\text{Sin } 16.5°$	648

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Foundation Reactions:

Note that passive earth pressure is not used in calculating foundation reactions.

Vertical Loads				
Component	**Load [ft^2]**	**Load Factor**	**Moment Arm about Toe [ft]**	**Moment [lbs.ft/ft]**
Wall weight (including footing and key)	7,335	1.0	6.87	50,391
Backfill soil weight	14,408	1.0	10.494	151,197
Footing cover soil weight	1,459	1.0	3.04	4,435
Vertical active earth pressure	5,382	1.0	13.50	72,657
Sum of Factored Vertical Loads	**28,584**			
Horizontal Loads				
Component	**Load [ft^2]**	**Load Factor**	**Moment Arm about Toe [ft]**	**Moment [lbs.ft/ft]**
Horizontal active earth pressure	13,321	1.0	10.750	-143,201
Inertial load on wall	2,200	1.0	6.173	-13,581
Inertial load on backfill soil	4,322	1.0	11.731	-50,701
Inertial load on footing cover soil	438	1.0	2.500	-1,095
Sum of Factored Horizontal Loads	**20,281**			
Sum of Factored Moments				**70,102**

Q = sum of vertical forces = 28,584 lb/ft
V = sum of horizontal forces = 20,281 lb/ft
M = sum of moments = 70,102 lb.ft/ft
e = foundation eccentricity = W/2-M/Q = 13.50/2-70,102/28,584 = 4.297 ft
e/W = 4.297 / 13.50 = 0.318 < 0.333

Bearing Capacity:

Using Vesic coefficients: $\phi \times q_{ult}$ = 9,212 psf
B' = B-2e = 13.5-2×4.297 = 4.91 ft
q_{demand} = 28,584 / 4.91 = 5,822 psf
Capacity / Demand =9,212 / 5,822 = 1.58 > 1.00

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Sliding Along Horizontal Plane:

Bottom of Footing Resistance					
Component	**Load [ft^2]**	**Load Factor**	**Resis-tance Factor**	**Factored Resisting Force [lbs/ft]**	**Force Value [lbs/ft]**
Vertical Load	28,584	n.a.	1.0	28,584×Tan 33°×1.0	18,563
Vertical passive earth pressure on footing	992	1.0	1.0	-1.0×992×Tan 33°×1.0	-644
Vertical passive pressure on key	648	1.0	1.0	-1.0×648×Tan 33°×1.0	-421
Sum of Factored Bottom of Footing Resistance					**17,498**
Passive Earth Pressure					
Component	**Pressure Length [ft]**	**Load Factor**	**Resis-tance Factor**	**Factored Passive Force [lbs/ft]**	**Force Value [lbs/ft]**
Horizontal passive earth pressure on footing	3,347	n.a.	1.0	1.0×3,347	3,347
Horizontal passive pressure on key	2,186	n.a.	1.0	1.0×2,186	2,186
Sum of Factored Passive Pressure Resistance					**5,533**

Sliding Factor of Safety:

Capacity / Demand = Sum of factored resisting forces / sum of factored driving forces
= (17,498+5,533)/(20,281) = 1.14 > 1.00

A screen shot of the spreadsheet that has been used in stability calculations is shown on the next page.

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External Stability

Calculation

Soil Parameters:

- γs = Unit weight of soil
- δ = Angle of friction range from (ϕ/2) to (2ϕ/3) for dry sand
- ϕ = Slope of ground surface behind wall (34 degrees for sand)

Seismic Parameters:

- θ = Seismic inertia angle $\theta = \tan^{-1}[Kh/(1-Kv)]$
- Kv = Coefficient of vertical acceleration
- Kh = Coefficient of Horizontal acceleration

Retaining Wall Parameters:

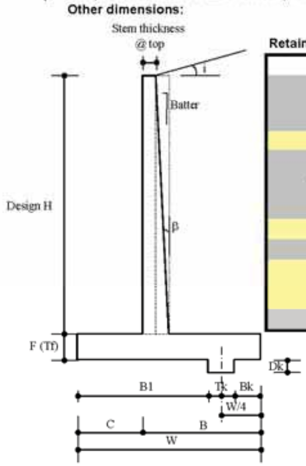
- γw = Unit weight of retaining wall
- i = Slope of ground surface behind wall (β in NCEL)
- β = Slope of back of the wall to vertical (θ in NCEL)

Other dimensions:

	LRFD Factors Load min	max	Resistance
DC	0.9	1.25	
EH	0.9	1.5	
ES	1.5	0.75	
EV	1	1.35	

	c		fi	
Passive	0.6	0.6	0.55	0.55
Bearing	0.6	0.6	0.55	0.55
Sliding	0.8	0.8	0.8	0.8
Soil on Soil	1	1	1	1
Passive Sliding	0.5	0.5	0.5	0.5

load combination *overturning*

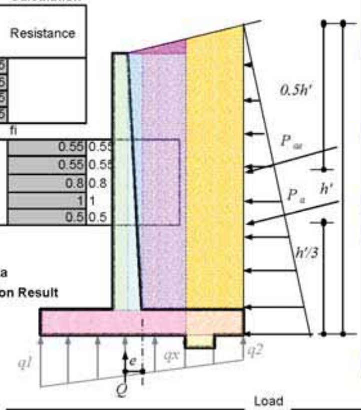


Retaining wall dimensions:

	length (ft)
Design H	20.00
W	13.50
C	6.08
B	7.42
F (Tf)	1.50
Stem thick. @ top	1.00
Batter	1/2: 12
i	0.00 deg.
$\beta = \tan^{-1}$(Batter)	2.39 deg.
Dk	1.00
Tk	2.00
Bk	3.50
B1	8.00
foundation cover	2.00 ft

Input Data
Calculation Result

passive resistance factored? *no*
passive pressure on footing: *on footing+cover*



Material properties

γ_s	120.00 pcf
γ_{cover}	120.00 pcf
γ_f	120.00 pcf
δ soil-soil*	22.00 deg.
δ soil-footing	33.00 deg.
$\delta_{passive}$	16.50 deg.
ϕ	33.00 deg.
ϕ_{cover}	33.00 deg.
ϕ_f	33.00 deg.
θ	16.70 deg.
K_v **	0.00
K_h	0.30
γ_w	145.00 pcf
i	0.00 deg.
β soil-soil	0.00 deg.
β soil-wall	2.39 deg.
c	0.00
c_f	0.00
K_a (Coulomb)	0.26
K_{ae} (Coulomb)	0.52
K_p (Coulomb)	5.70
K_{pe} (Coulomb)	4.75
ΔK_{ae}	0.25

* angle from horizontal line
** positive down

	Weight (lbs/ft)	Arm (ft)	Load Factor
W_{wall} =	7335	6.872	0.9
W_{soil} =	14408	10.494	1
Pa =	7322		
Pax =	6789	7.167	1.5
Pay =	2743	13.500	1
Pae =	14367		
Paex =	13321	10.750	1
Paey =	5382	13.500	1
Ppx =	4017	1.167	0.5
Ppy =	1190	0.000	0.5
Ppex =	3347	1.167	1
Ppey =	992	0.000	1
Ppxkey =	2623	-0.521	0.5
Ppykey =	777	0.000	0.5
Ppexkey =	2186	-0.521	1
Ppeykey =	648	0.000	1
horizontal inertial load on soil ΔFsx =	4322	11.731	1
horizontal inertial load on wall ΔFwx =	2200	6.173	1
vertical inertial load on soil ΔFsy =	0	10.494	1
vertical inertial load on wall ΔFwy =	0	6.872	1
W_{cover} =	1459.2	3.040	1
ΔW_{coverx} =	437.76	2.500	1
ΔW_{covery} =	0	3.040	1
h' =	21.50	ft	

	Static State	Seismic State	
Q =	25211	28584	lbs
e =	0.204	4.297	ft
q1 =	2037	6161	lb/ft2
q2 =	1698	-1927	lb/ft2
Lx =	8.00	7.36	ft
q1' =	2037	7770	lb/ft2
q2' =	1698	0	lb/ft2
qx =	1836	0	lb/ft2
T =	10183	13321	lbs
e/w =	0.015	0.318	
B' =	13.09	4.91	ft
Pmax =	12459	17498	lbs

$\phi q_{u,static}$ (psf)	C/D
9,239	4.80

$q_{u,seismic}$ (psf)	C/D
9,212	1.58

Dk<=Tf Wall with shallow footing key or no footing key
Bk>Dk Pressure height h' ends at bottom of wall heel

Other Loading Parameters:

Sliding stability (static state)

$$Fs,slide = \frac{Pmax+\phi_\tau Pp}{Pa} = 1.55 \quad >=1.0$$ safe

Sliding stability (seismic state)

$$Fs,slide = \frac{Pmax+\phi_\tau (Pp+\Delta Ppe)}{Pa+\Delta Pae} = 1.14 \quad >=1.0$$ safe

Overturning

Foundation type: (soil/rock) soil

W/6 =	2.25 >= e	static	safe
W/3 =	4.50 >= e	seismic	safe

α = 0.12 radians

Pmax = 12452 17513 lbs

Sliding stability (inclined sliding plane, static state)

$$Fs,slide = \frac{Pmax+\phi_\tau Pp}{Pa} = 1.74 \quad >=1.0$$ safe

Sliding stability (inclined sliding plane, seismic state)

$$Fs,slide = \frac{Pmax+\phi_\tau (Pp+\Delta Ppe)}{Pa+\Delta Pae} = 1.21 \quad >=1.0$$ safe

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Example Nongravity Cantilever Wall Problem – Limit Equilibrium Method

Introduction

This example demonstrates the application of the proposed seismic design procedure outlined in Section X.8 of the proposed Specifications for the seismic design of a cantilever sheet pile wall. The static and seismic design is established following the limit equilibrium method in AASHTO.

Wall Geometry and Soil Properties

The geometry of the wall is shown on Figure 1. The vertical element is a continuous sheet pile. Properties for the backfill and foundation soils are shown on Figure 1.

Static Design Methodology Using AASHTO LRFD Method

Figure 2 shows the AASHTO recommended factored simplified earth pressure distribution for permanent nongravity cantilevered walls with continuous vertical wall elements embedded in granular soil. The live load surcharge (LS) is added to the basic earth pressure distribution.

The embedment depth was calculated using the following procedure:

- Calculate x in Figure 2 from the following equation:

  $$x = [\gamma k_{a2} \gamma'_{s1} H] / [(\phi k_{p2} - \gamma k_{a2}) \gamma'_{s2}]$$

  where:

  γ = load factor for horizontal earth pressure, EH
  k_{a2} = active earth pressure coefficient for soil 2
  γ'_{s1} = effective soil unit weight for soil 1
  H = design height of the wall
  ϕ = resistance factor for passive resistance in front of the wall
  k_{p2} = passive earth pressure coefficient for soil 2
  γ'_{s2} = effective soil unit weight for soil 2

- Sum the moments about the point of action of F (the base of the wall) to determine the embedment (D_0) for which the net passive pressure is sufficient to provide moment equilibrium.

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- Determine the depth at which the shear in the wall is zero, i.e., the point at which the areas of driving and resisting pressure diagrams are equivalent.

- Calculate the maximum bending moment at the point of zero shear.

- Calculate the design depth, $D = 1.2D_0$, to account for errors inherent in the simplified passive pressure distribution.

Table 1 shows the load and resistance factors relevant to LRFD design of nongravity cantilevered walls.

Table 1. Load and Resistance Factors for Permanent Nongravity Cantilevered Walls

Load Combination	Load	Type	Maximum Factor	Minimum Factor
Strength I	Horizontal Earth Pressure, EH	Load	1.50	0.90
	Live Load Surcharge, LS	Load	1.75	1.75
	Passive Pressure	Resistance	0.75	0.75
Extreme Event I	Horizontal Earth Pressure, EH	Load	1.00	1.00
	Live Load Surcharge, LS	Load	γ_{EQ}[1]	γ_{EQ}
	Passive Pressure	Resistance	1.00	1.00

1) To be determined on a project-specific basis.

For a cantilevered nongravity wall, the maximum horizontal earth pressure factor (1.50) will control the design. The active earth pressure coefficients for backfill and foundation soils were calculated using the Coulomb Method. Active earth pressure coefficients of 0.283 and 0.26 were calculated for the backfill and foundation soil, respectively. Wall friction angle (δ) was assumed to be zero on the active side. For the passive side, however, a wall friction angle of 24° (2/3 ϕ) was used. The designer should evaluate these assumptions on a project-specific basis. The static passive earth pressure coefficient for the foundation soil was estimated to be 8.2 using the log-spiral method.

The following parameters were used to estimate the wall dimensions for the Strength I load case:

$\gamma =$ 1.50 (load factor for horizontal earth pressure)

$\gamma_{LS} =$ 1.75 (load factor for surcharge load)

$\phi =$ 0.75 (resistance factor for passive pressure)

$k_{a1} =$ 0.283

$k_{a2} =$ 0.260

$k_{p2} =$ 8.2

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γ'_{s1} = 120 pcf

γ'_{s2} = 125 pcf

H = 10 ft

Figure 3 shows the factored pressure distribution for the Strength I load case. Using the aforementioned procedure and the above parameters, the following dimensions were obtained: x = 0.89 ft, D_0 = 8.57 ft and D = 10.3 ft. These results were checked using the CT-Flex program (Shamsabadi, 2006). The distribution of shear force and bending moment in the vertical element was calculated by CT-Flex. Results of the CT-Flex analyses are shown in Figure 4.

Seismic Design Methodology Using AASHTO LRFD Method

Seismic earth pressures were evaluated for three levels of site-adjusted peak ground acceleration coefficient: k_{max} = 0.2, 0.4 and 0.8 where $k_{max} = F_{pga}$ PGA. It was assumed that a small amount of movement of the wall at the excavation level was permissible for the design seismic event – as long as the wall did not collapse. This assumptions allowed a seismic coefficient equal to 0.5 k_{max} (i.e., k_{max} = 0.1, 0.2 and 0.4). The methodology followed that outlined in Section X.8 of the proposed Specifications. Since the free height of the wall was less than 20 feet, the seismic acceleration coefficients were not adjusted for wall-height effects.

The Mononobe-Okabe (M-O) equation was used to estimate the seismic earth pressure for a non-cohesive backfill in view of the very simple geometry. For the seismic condition, the earth pressure distribution on the free height of the wall was assumed to be uniform. Table 2 shows the estimated active pressure coefficients and seismic earth pressures for the free height of the wall; Table 3 shows the seismic active and passive earth pressure coefficients for foundation soil 2.

Table 2. Earth Pressure for Backfill, Estimated by M-O Equation.

Seismic Coefficient, k_{max}	**Static**	**0.1**	**0.2**	**0.4**
EQ Earth Pressure Coefficient, K_{ae}	0.283	0.341	0.410	0.602
Total Seismic Earth Pressure Load on the Wall (lb/ft) = $\frac{1}{2} \gamma H^2 K_{ae}$ (above excavation depth)	n/a	2,046	2,460	3,612
Distributed Uniform Pressure (psf)	n/a	205	246	361

Table 3. Earth Pressure for Foundation Soil.

Seismic Coefficient, k_{max}	**Static**	**0.1**	**0.2**	**0.4**
EQ Earth Pressure Coefficient, K_{ae}	0.260	0.315	0.381	0.561
EQ Passive Earth Pressure Coefficient, K_{pe}	8.2	7.67	7.21	6.15

The seismic lateral earth pressure above the excavation depth was applied as a uniform load, acting on the design height of the wall (top 10 feet), as shown in Figures 5 to 7. As the form of seismic active and passive pressure distributions below the excavation level is uncertain, it was

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simply assumed they are similar to static pressure distributions. The procedure used for the Strength I load condition was used for the seismic case except that the load and resistance factors were changed to 1.0 for the seismic condition. According to AASHTO, the percentage of the live load surcharge to be used in seismic design (γ_{EQ}) should be determined on a project-specific basis. For this example 100% of the live load surcharge was assumed to be present under the seismic condition (γ_{EQ} = 1.0).

The following parameters are used to estimate the wall dimensions for the Extreme Event I load case:

γ =	1.0 (load factor for horizontal earth pressure)
γ_{EQ}=	1.0 (load factor for surcharge load)
ϕ =	1.0 (resistance factor for passive pressure)
k_{a1} =	from Table 2
k_{a2} =	from Table 3
k_{p2} =	from Table 3
γ'_{s1} =	120 pcf
γ'_{s2} =	125 pcf
H =	10 ft

Based on the above parameters, the wall dimensions were obtained for each of three seismic cases. The results were also checked independently using the CT-Flex program. The distribution of shear force and bending moment in the vertical element was also calculated for each acceleration level using CT-Flex.

Table 4 shows the summary of wall dimensions and maximum factored shear force and moment in the vertical element. The results are shown graphically in Figures 8 to 10. Based on these results, the Strength I load case controls the design for the first two seismic cases (k_{max} = 0.1 and 0.2), while the Extreme Event I load case is more critical for k_{max} = 0.4 case, resulting in a deeper embedment depth and a stronger cross section for the sheet pile, compared to the static design.

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Table 4. Summary of Wall Dimensions and Shear-Moment Demand for Static and Seismic Conditions

Load Combination	**Strength I (Static)**	**Extreme Event I (k_{max} = 0.1)**	**Extreme Event I (k_{max} = 0.2)**	**Extreme Event I (k_{max} = 0.4)**
x (ft)	0.89	0.53	0.67	1.16
D_0 (ft)	8.57	6.71	7.47	9.79
D (ft)	10.3	8.05	8.96	11.75
Factored Moment M_{max} (lb-ft)	27,272	20,088	24,339	38,905
Max. Moment Location	4.41 feet below excavation	3.14 feet below excavation	3.60 feet below excavation	5.03 feet below excavation
Factored Shear V_{max} (lbs)	15,030	13,137	14,574	18,764

Global Stability for Static Load Conditions

The global stability of the cantilever wall for the static condition was evaluated using the computer program SLIDE (RocScience, 2007). . The Spencer method was used to evaluate the minimum factor of safety for planar and circular failure surfaces. The results are shown in Figures 11 and 12 for planar and circular failure planes, respectively.

Recommendations in Section 11.6.2.3 of the AASHTO *LRFD Bridge Design Specifications* were followed for global stability:

> *"The overall stability of the retaining walls, retained slopes and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis. The overall stability of temporary cut slopes to facilitate construction shall also be evaluated. Special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft deposits.*
>
> *The evaluation of overall stability of earth slopes with or without a foundation unit should be investigated at Service 1 Load Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, ϕ, may be taken as:*
>
> - *Where the geotechnical parameters are well defined, and the slope does not support or contain a structural element: 0.75*

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- *Where the geotechnical parameters are based on limited information, or the slope contains or supports a structural element: 0.65"*

Based on these recommendations, a resistance factor of 0.65 was applied to the soil strength parameters in the global slope stability analyses. This approach is somewhat different than is often followed for global stability analyses where the soil properties are not modified and results are considered acceptable if the total factor of safety is equal to the inverse of the resistance factor (i.e., FS = 1/0.65 = 1.538). However, the procedure used here allows for use of partial resistance factors on different resistive components (e.g., friction and cohesion) and agrees better with LRFD philosophy.

Following this approach, the strength parameters for backfill soil were calculated as:

- $0.65 \times \tan(\phi) = 0.65 \times \tan(34) = 0.4384 \rightarrow$ use a friction angle of 23.7°.

The strength parameters for foundation soil were calculated as:

- $0.65 \times \tan(\phi) = 0.65 \times \tan(36) = 0.4868 \rightarrow$ use a friction angle of 26.0°.

As shown in Figures 11 and 12, the minimum factor of safety for the static condition using the factor friction angle was 2.06 and 2.93 for planar and circular failure planes, respectively.

Global Stability under Seismic Loading

The global stability of the cantilevered wall for seismic conditions was evaluated using the program SLIDE. The Spencer method was used to evaluate the minimum factor of safety for planar and circular failure surfaces. The original material properties (ϕ = 34° for backfill and ϕ = 36° for foundation soil) were used in the seismic slope stability analyses. This is equivalent to using a resistance factors of one. In addition, the larger of the embedment depths from static and seismic analyses were used in the slope stability models. Table 5 shows a summary of minimum capacity to demand ratios (factors of safety). The analysis results for k_{max} = 0.4 are shown in Figures 13 and 14, for planar and circular failure planes, respectively.

Table 5. Earth Pressure for Foundation Soil, Estimated by M-O Equation.

Seismic Coefficient, k_h	Static	0.1	0.2	0.4
Embedment Depth in Model (ft)	10.3	10.3	10.3	11.75
Minimum F.S. (planar Failure Plane)	2.06	2.54	2.05	1.53
Minimum F.S. (circular Failure Plane)	2.93	3.09	2.32	1.61

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Concluding Comments

These results show that cantilevered walls designed using guidelines in the AASHTO *LRFD Bridge Design Specifications* for static pressure distributions should perform very well during ground motions that can be encountered in seismically active areas. However, design for the seismic loading condition needs to be checked, as shown in this design example. For large PGA values, the seismic design is likely to govern the design of the wall. The global stability of the wall should also be checked for static and seismic conditions.

References

AASHTO (2007), "AASHTO LRFD Bridge Design Specifications, Customary U.S. Units," 4th Edition.

RocScience (2007). "SLIDE: Stability Analysis for Soil and Rock Slopes." www.rocscience.com.

Shamsabadi, A. (2006). *CT-FLEX – Computer Manual*, Office of Earthquake Engineering, California Department of Transportation.

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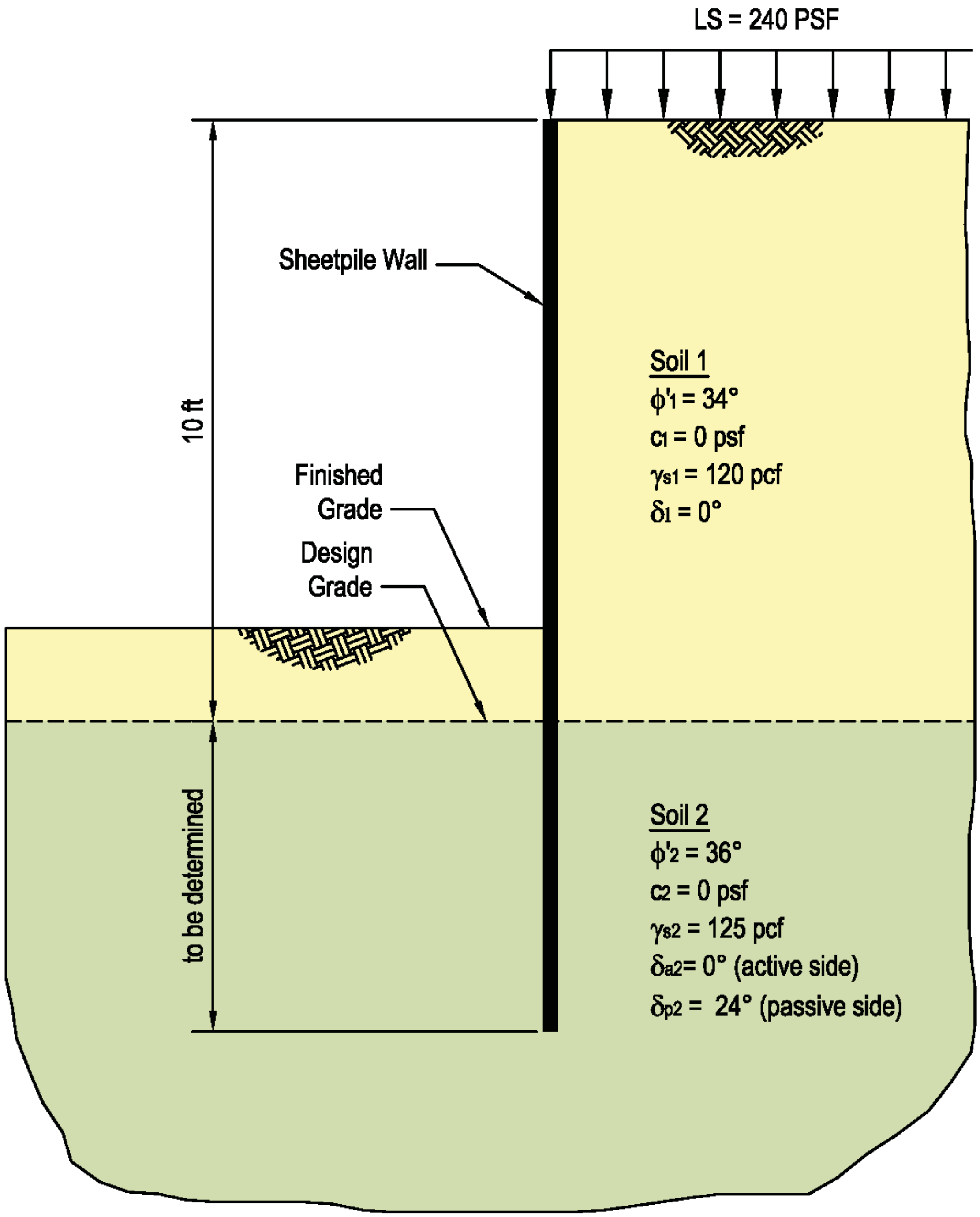

Figure 1. Cantilever Sheet pile Wall Geometry

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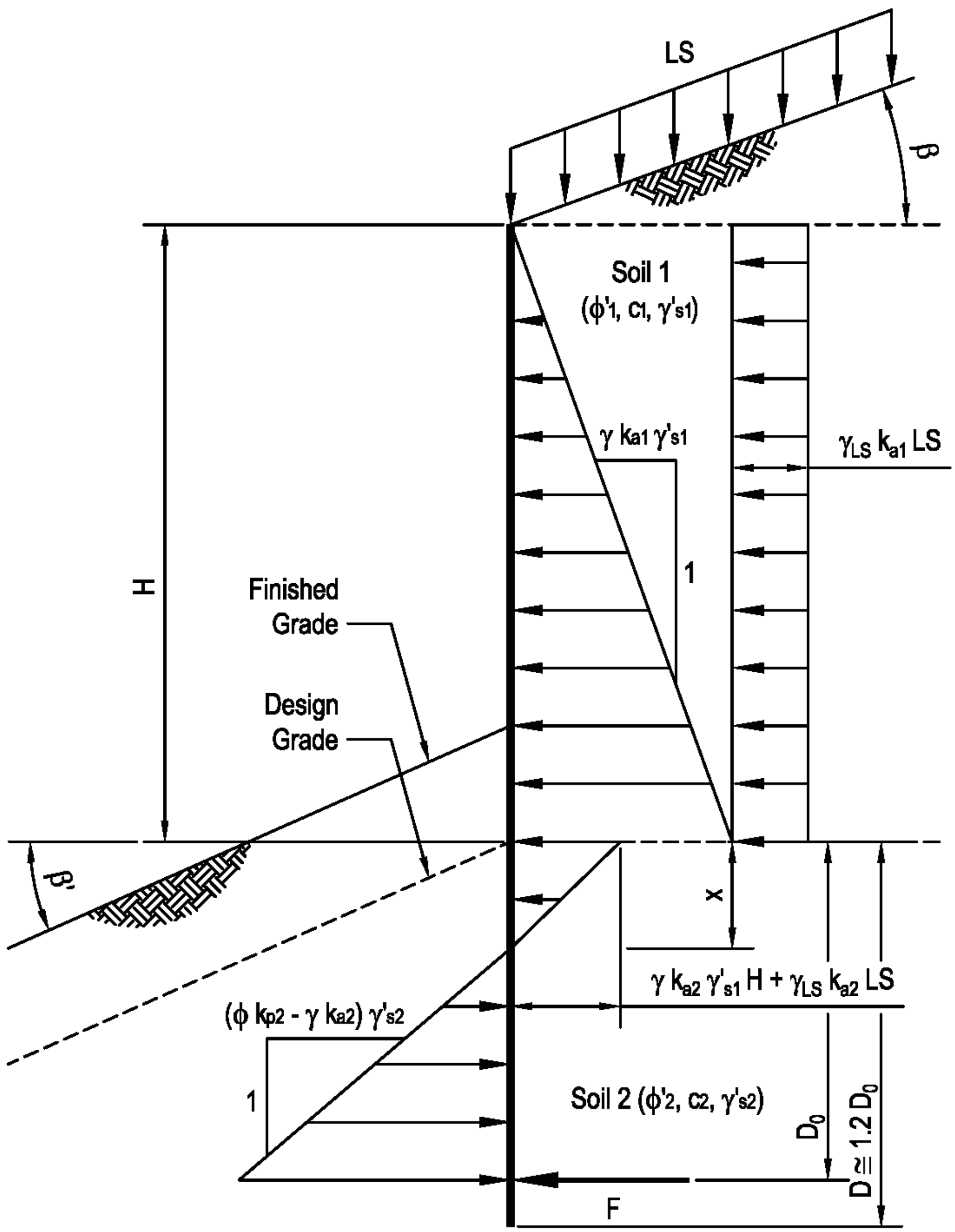

Figure 2. Factored Simplified Earth Pressure Distribution for Permanent Nongravity Cantilevered Walls with Continuous Vertical Wall Elements Embedded in Granular Soil (after AASHTO)

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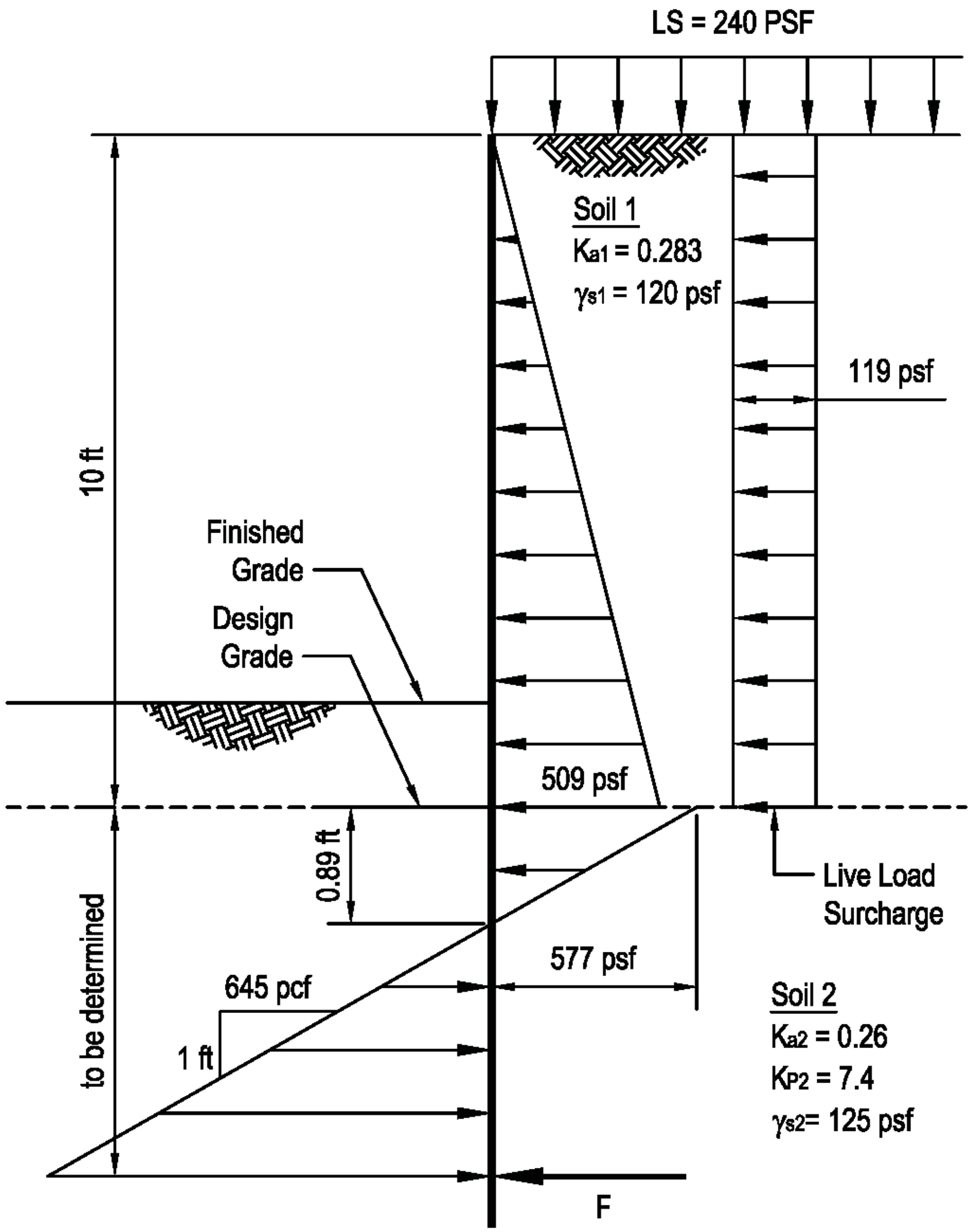

Figure 3. Factored Earth Pressure Distribution for Strength I Load Case

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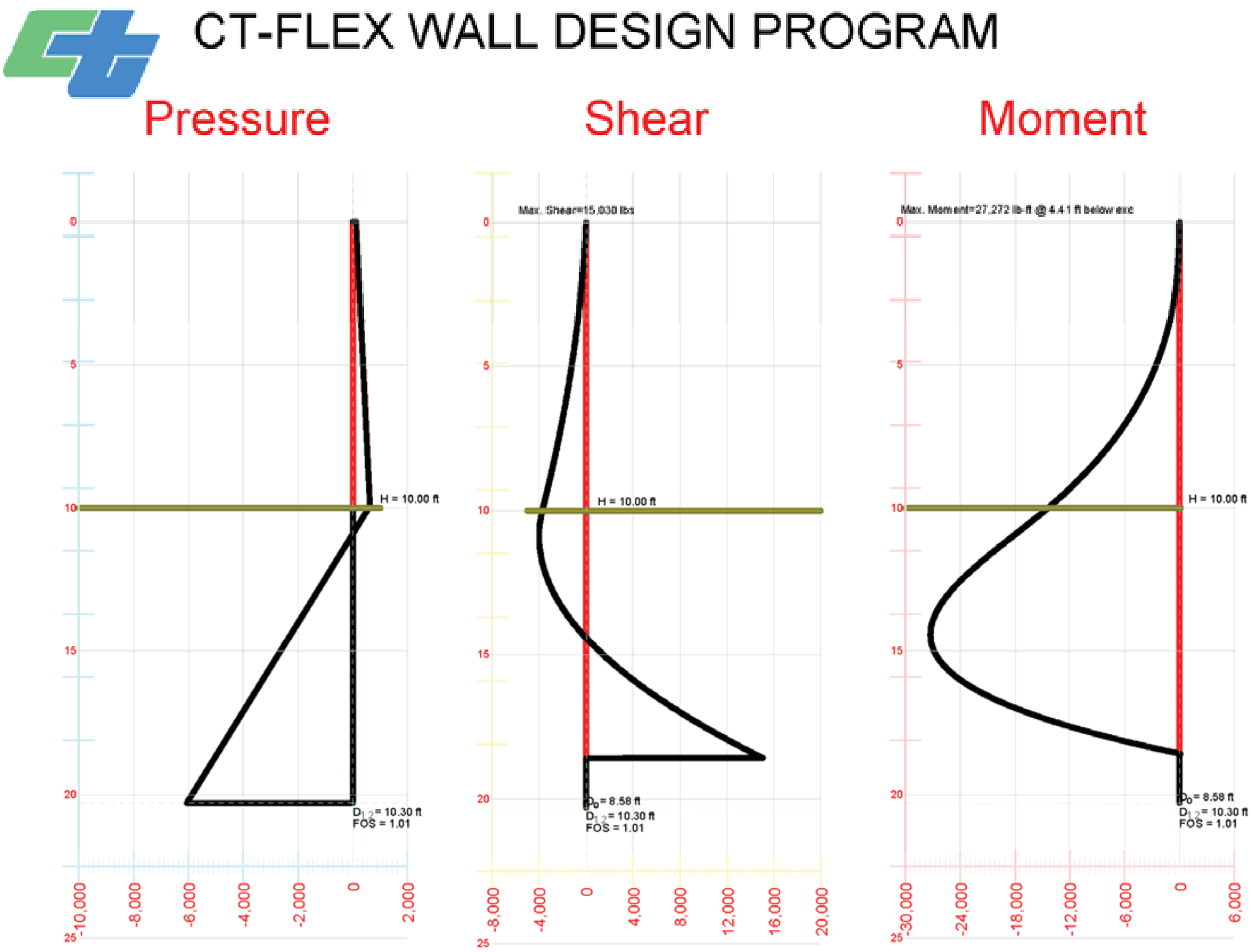

Figure 4. Pressure, Shear Force and Bending Moment Diagrams for Strength I Load Case

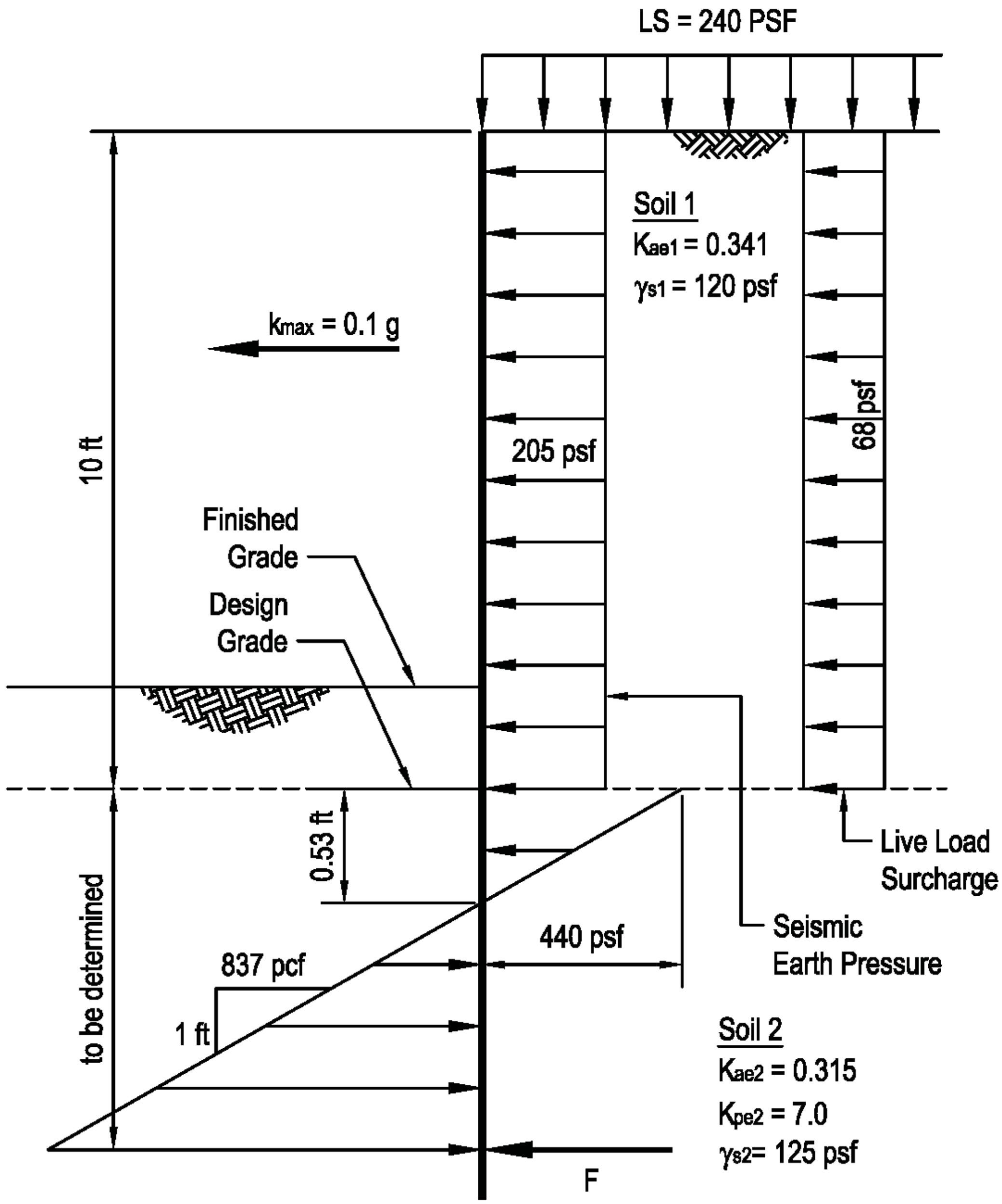

Figure 5. Factored Earth Pressure Distribution for Extreme Event I Load Case (k_{max} = 0.1)

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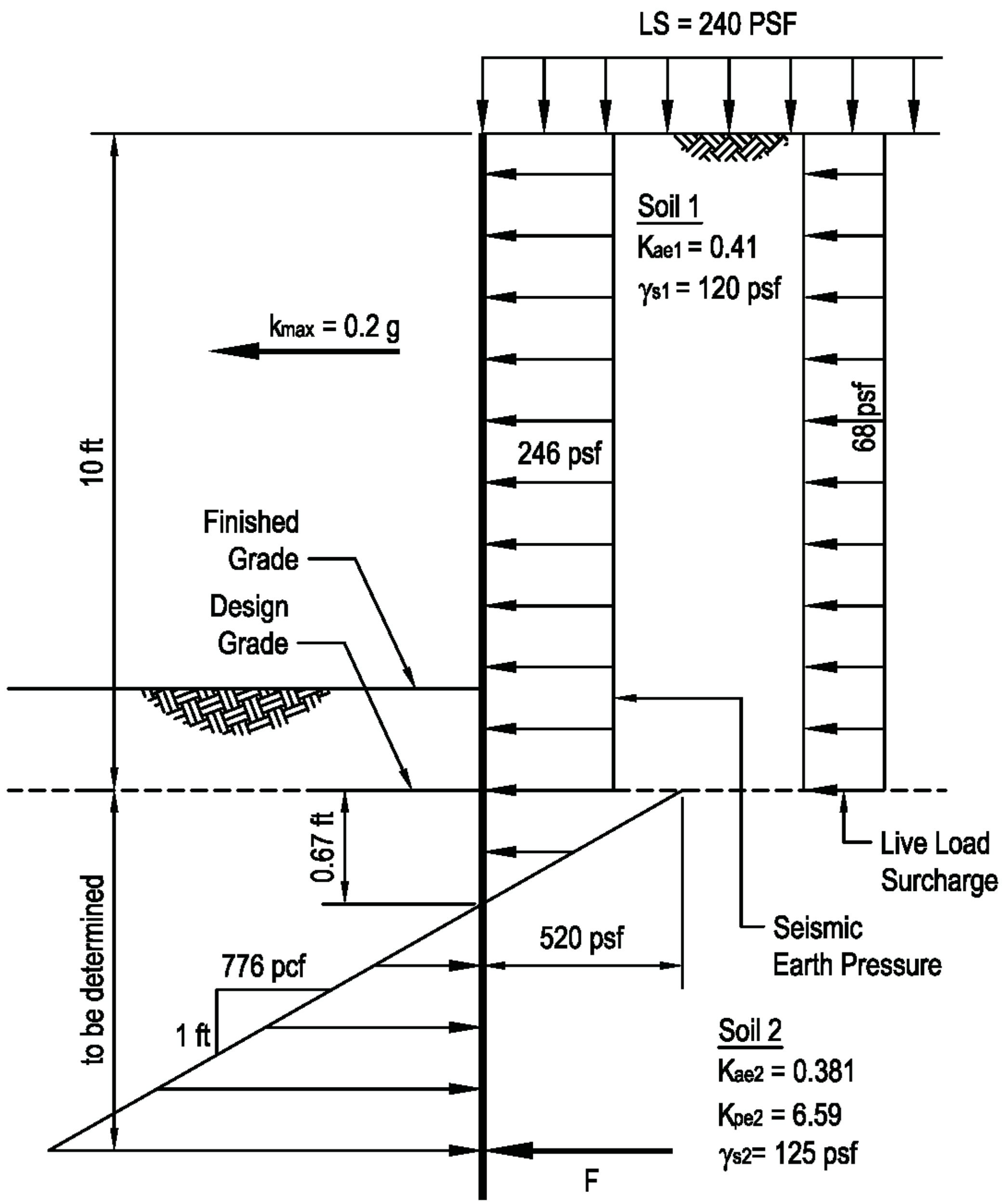

Figure 6. Factored Earth Pressure Distribution for Extreme Event I Load Case (k_{max} = 0.2)

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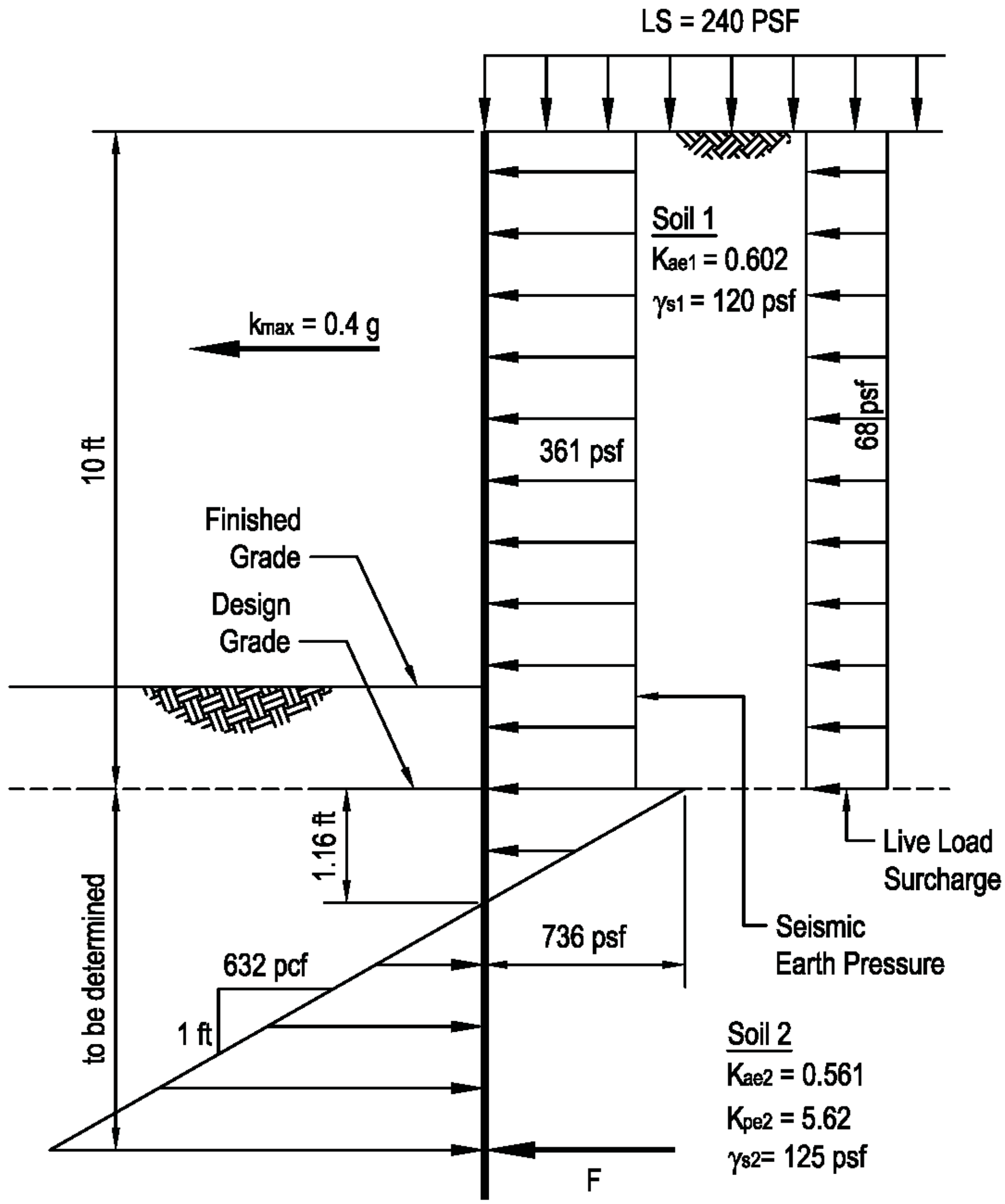

Figure 7. Factored Earth Pressure Distribution for Extreme Event I Load Case (k_{max} = 0.4)

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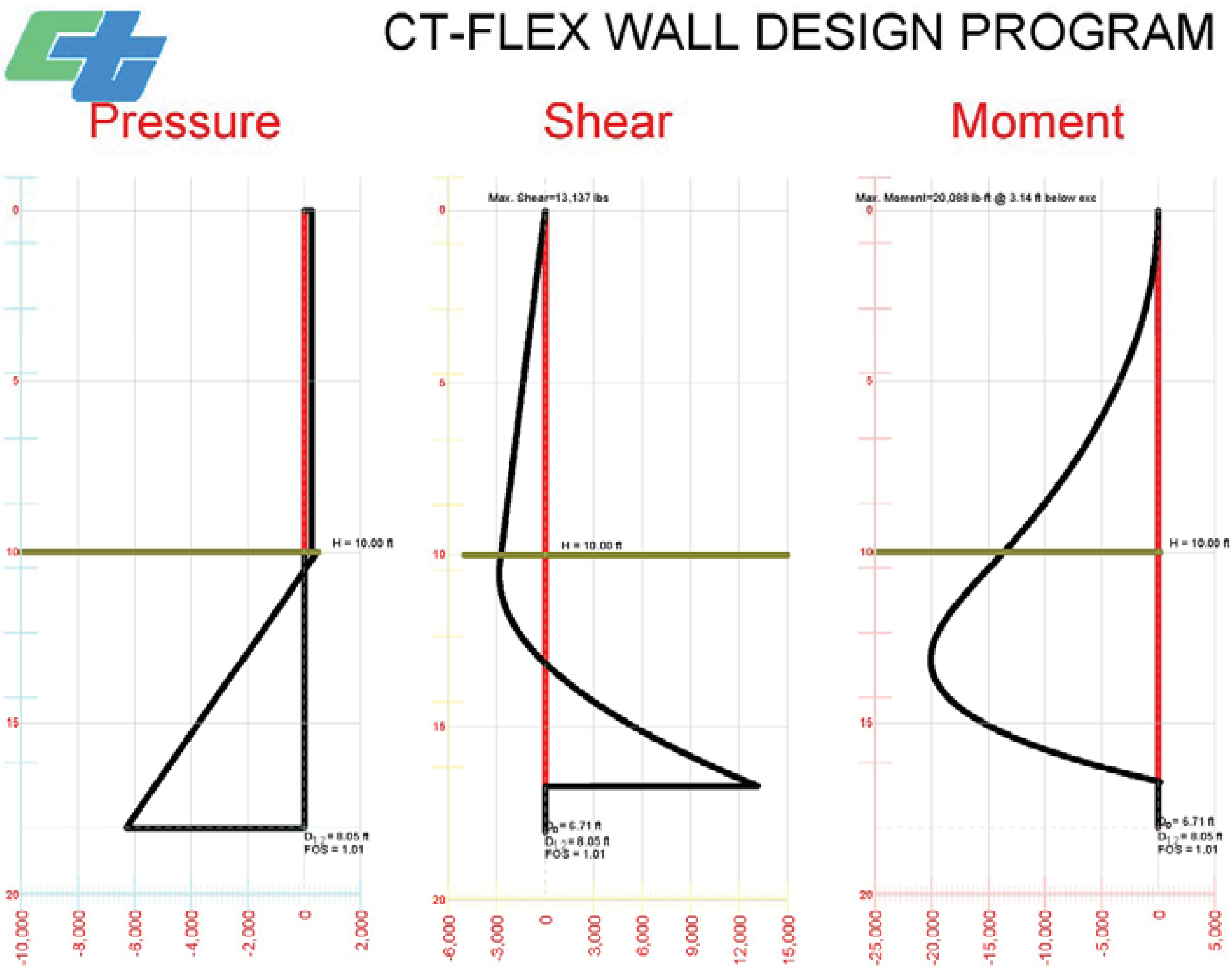

Figure 8. Pressure, Shear Force and Bending Moment Diagrams for Extreme Event I Load Case

(k_{max} = 0.1)

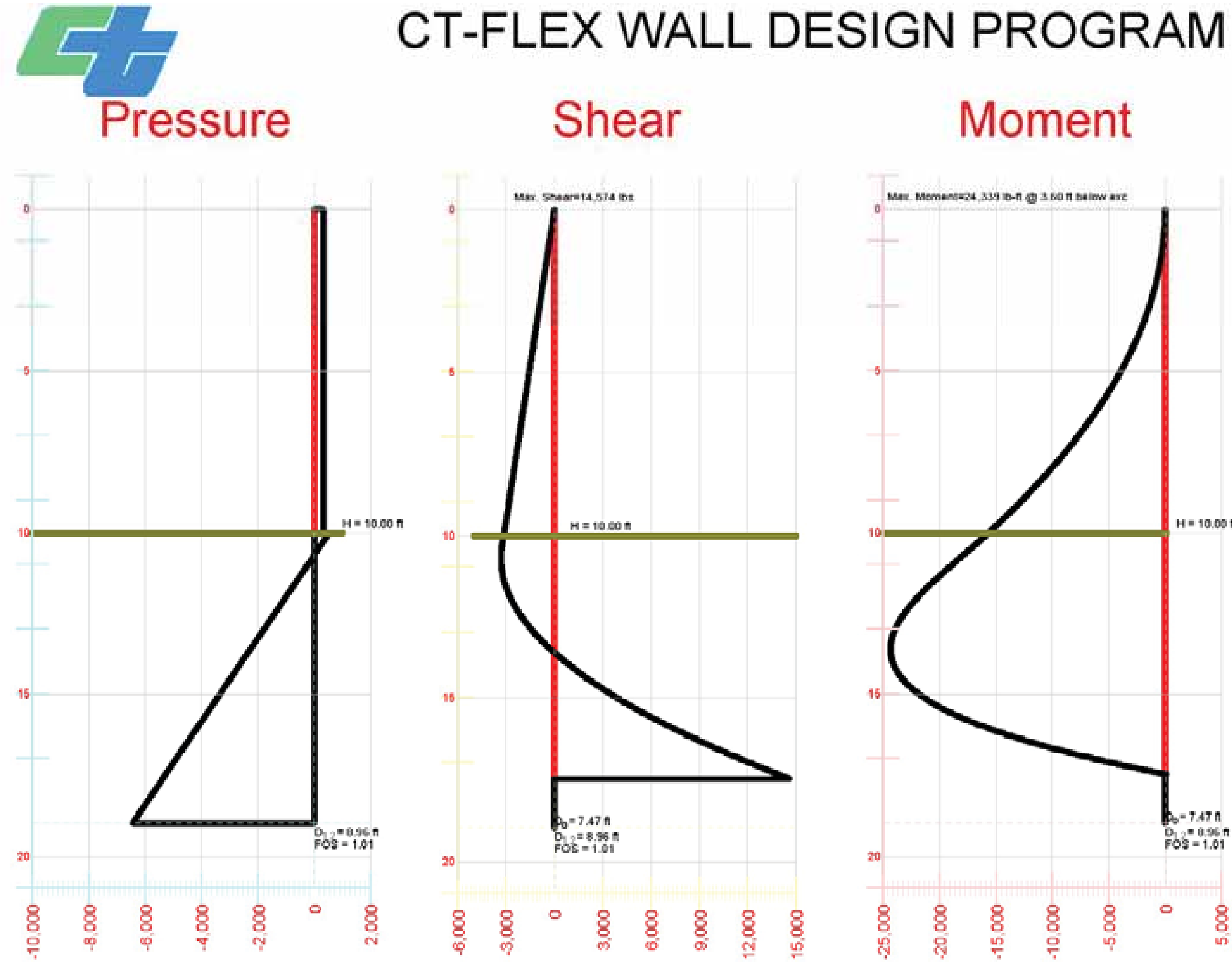

Figure 9. Pressure, Shear Force and Bending Moment Diagrams for Extreme Event I Load Case

(k_{max} = 0.2)

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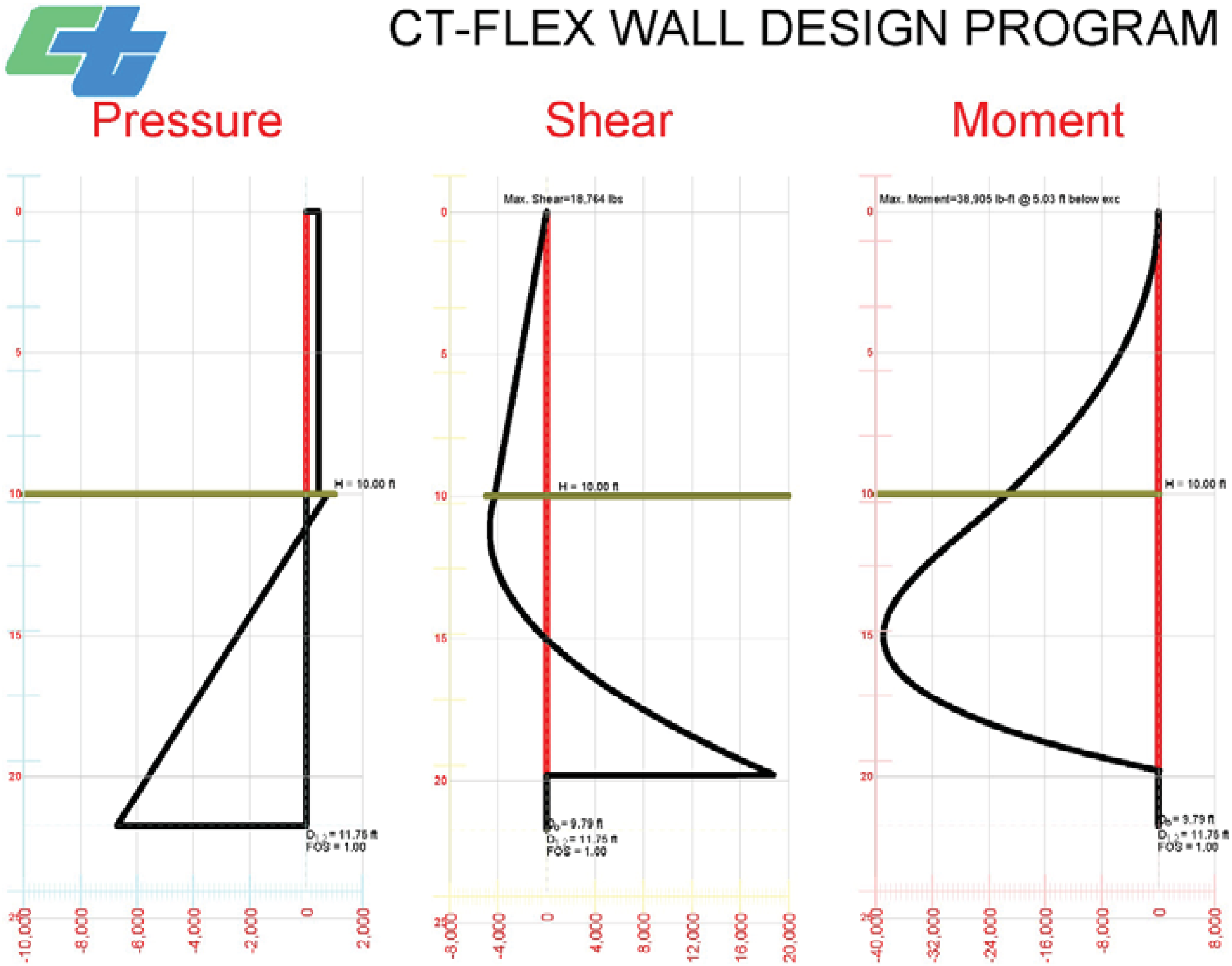

Figure 10. Pressure, Shear Force and Bending Moment Diagrams for Extreme Event I Load Case

(k_{max} = 0.4)

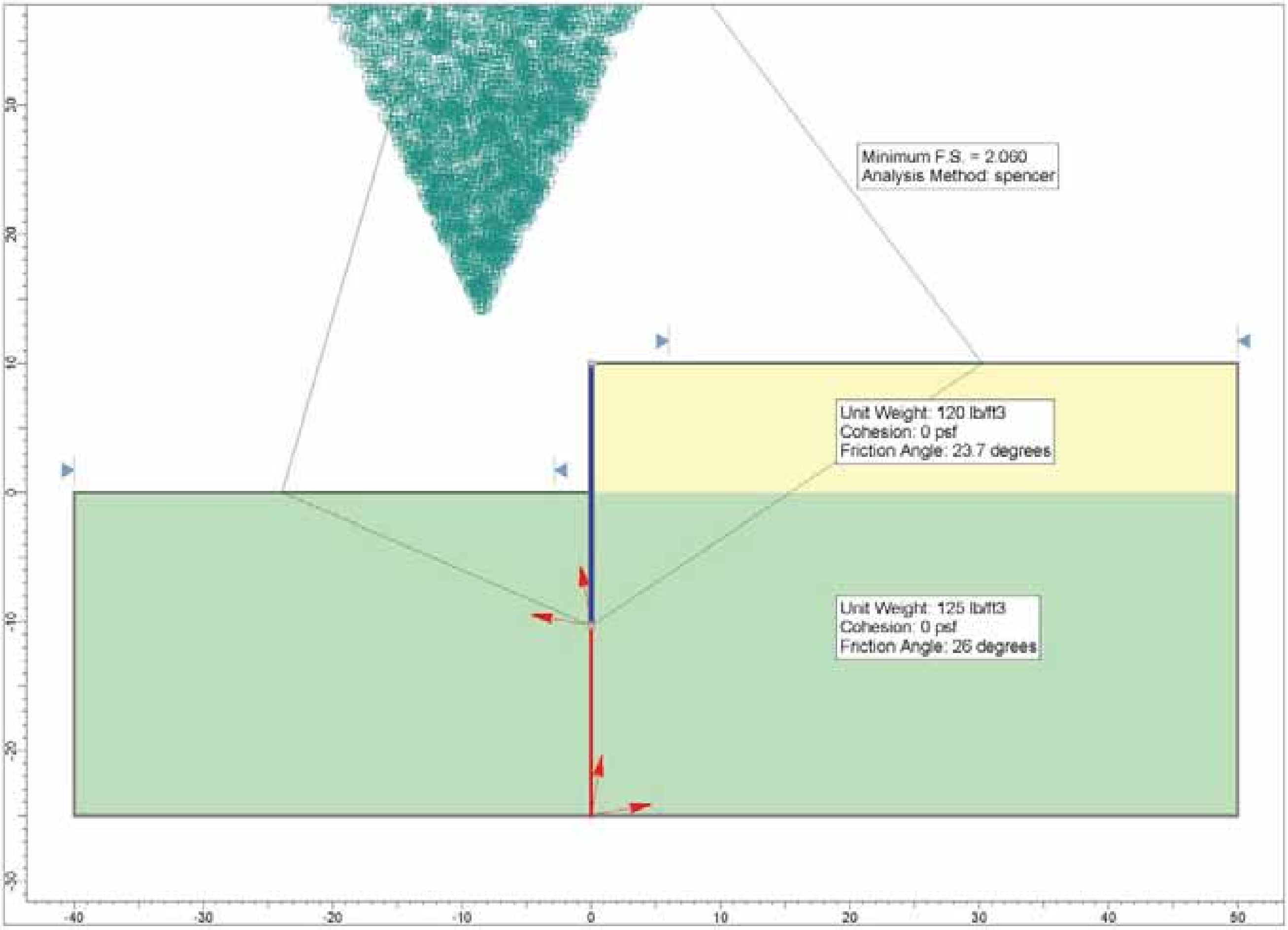

Figure 11. Slope Stability Analysis for Strength I Load Case (Planar Failure Plane)

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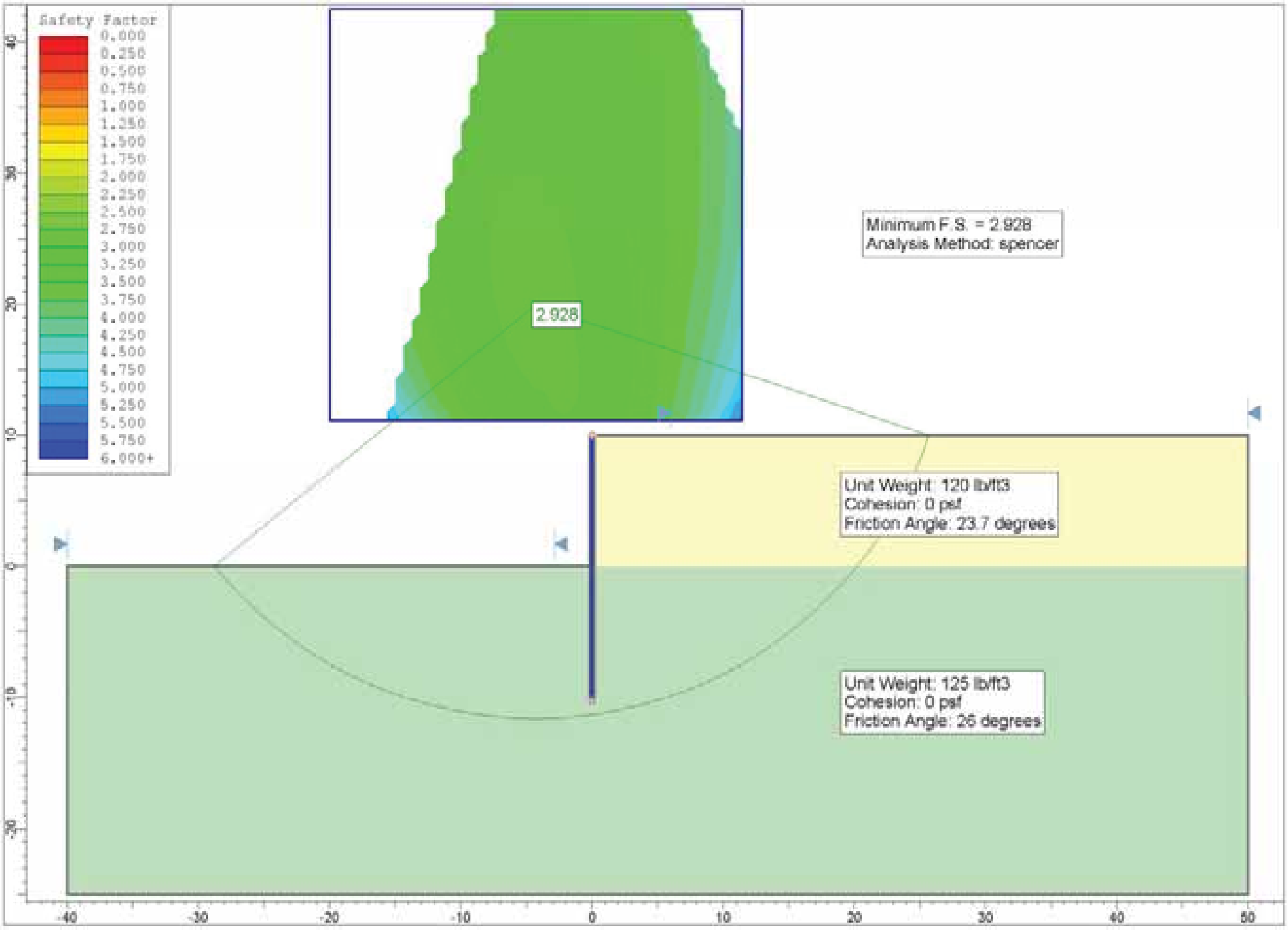

Figure 12. Slope Stability Analysis for Strength I Load Case (Circular Failure Plane)

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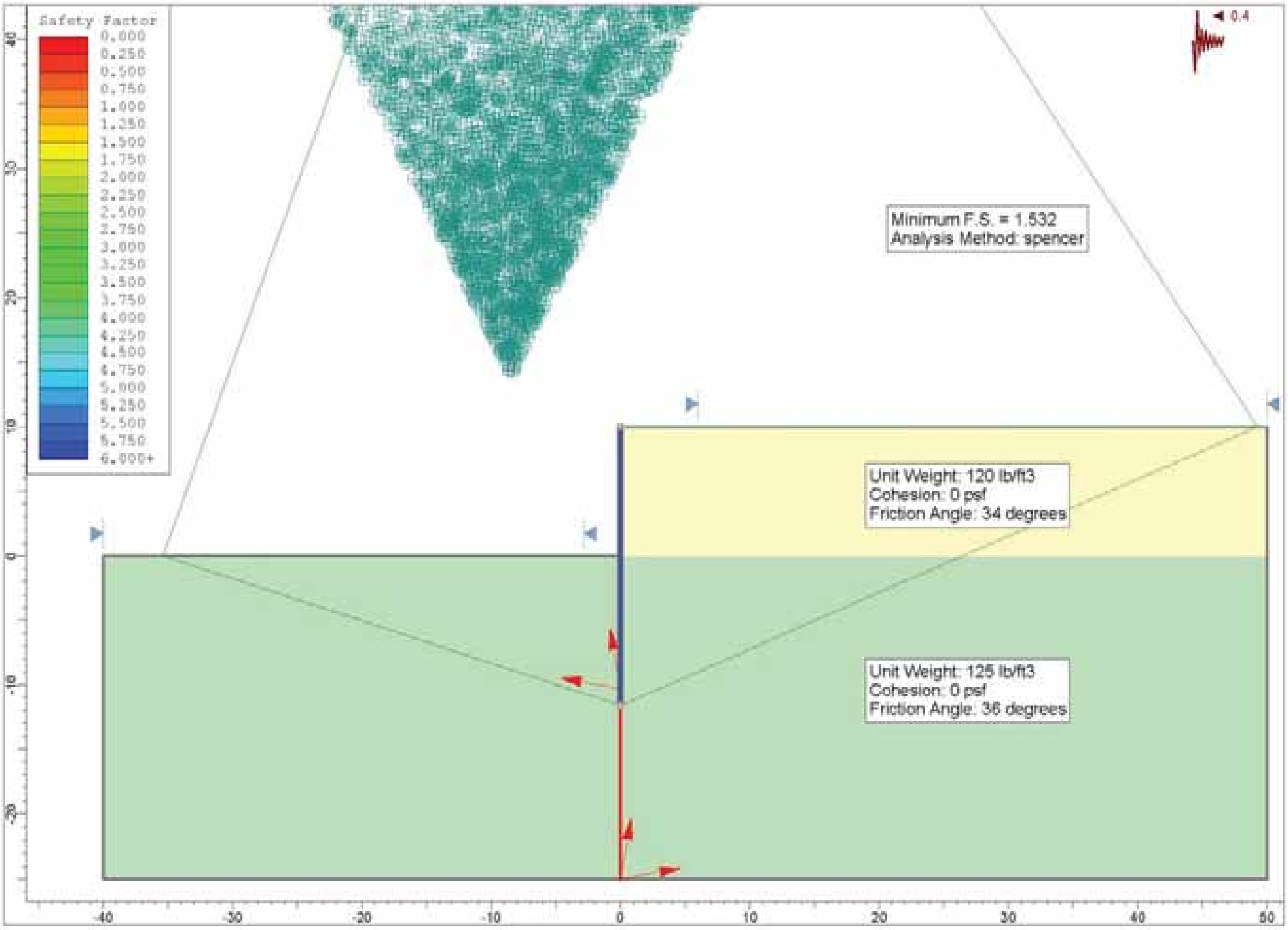

Figure 13. Slope Stability Analysis for Extreme Event I Load Case (Planar Failure Plane, k_{max} = 0.4)

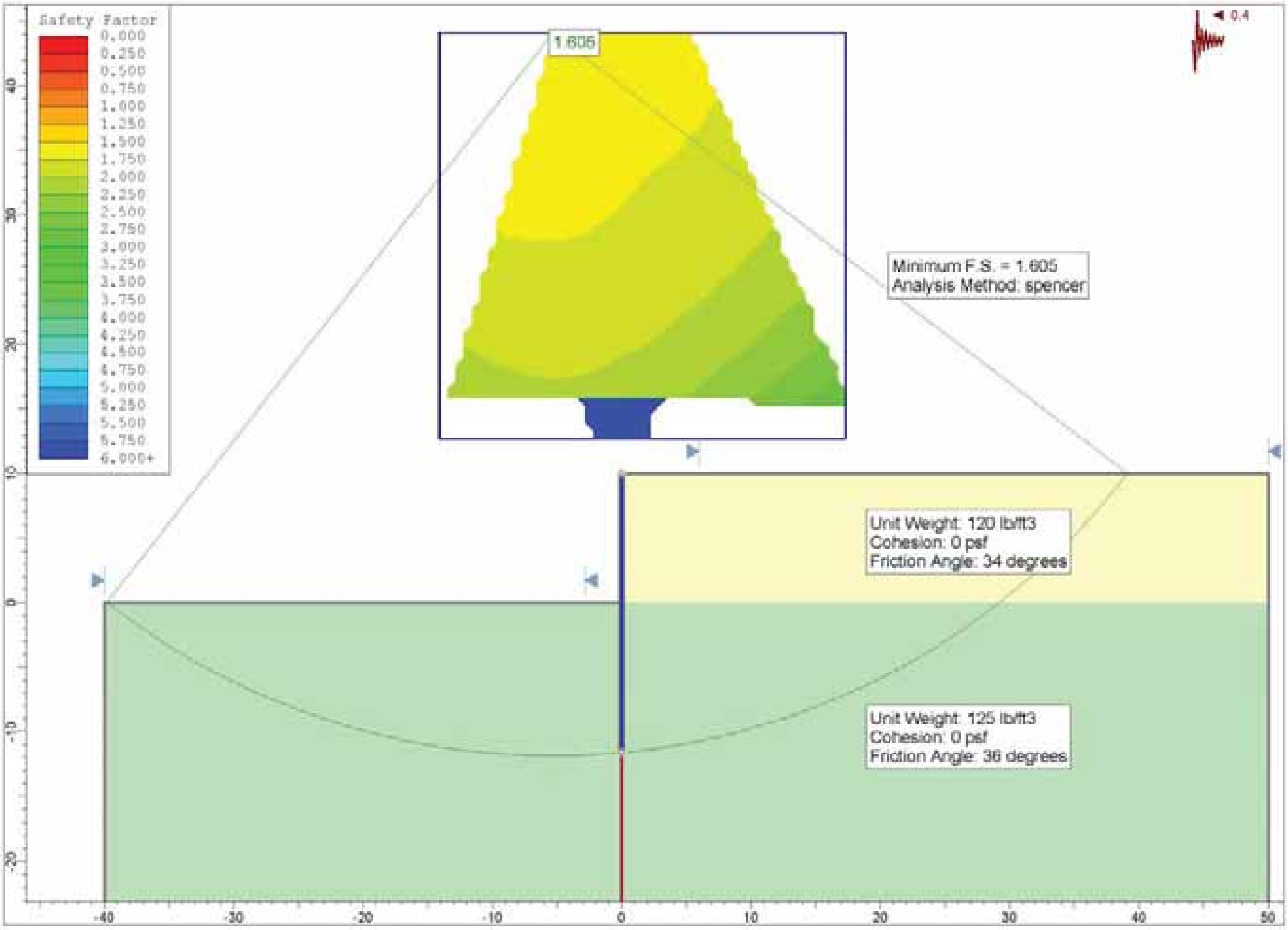

Figure 14. Slope Stability Analysis for Extreme Event I Load Case (Circular Failure Plane, k_{max} = 0.4)

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Example Nongravity Cantilever Wall Problems – P-Y Method

Introduction

Three examples were developed showing the design of a nongravity cantilever wall using the p-y method of design. The three example problems involved

- A cantilever sheet pile wall at the top of a slope (Example 1). This problem represents a typical road widening project, where changes in grade were accomplished by filling behind a cantilever wall.

- A cantilever sheet pile wall with a level ground at the base of the wall (Example 2). The demands on this wall are much lower because of the higher passive pressures developed at the base of the wall.

- A soldier pile cantilever wall (Example 3). In this example problem soldier piles rather than a sheet pile were used for the below grade portion of the wall. The use of the soldier piles changes the methods used when estimating the passive pressure in front of the below-grade portion of the wall.

A conventional limit equilibrium design was performed before conducting the p-y analysis. The limit equilibrium method was similar to the method given in the AASHTO *LRFD Bridge Design Specifications* but used a factor of safety approach when evaluating the stability of the wall. This approach is consistent with the method used by Caltrans for their designs in seismically active areas, and was thought to provide a good baseline evaluation. The p-y approach involved representing the wall by linear elastic beam elements and soil springs, similar to the approach commonly used to evaluate the response of piles to lateral loading. Results from the p-y analysis allowed direct determination of displacements, moments, and shears for the wall while accounting for the nonlinear characteristics of the soil. Comparisons were then made between designs based on the limit equilibrium method and the p-y (or displacement-based) approach.

The three wall examples were evaluated for three levels of seismic loading: k_{max} = 0.1, 0.2, and 0.3. Since the focus of these examples was on showing the benefits of the displacement-based approach compared to the limit equilibrium analysis, the seismic earth pressure was estimated using the Mononobe-Okabe method. The benefits of soil cohesion in the fill behind the wall were not accounted for directly in the example. However, as discussed in the proposed Specification, the benefits of cohesion, whether from fines in the soil or soil capillarity, can be significant. Appendix B_X includes charts for estimating the seismic earth pressure coefficients for active and passive earth pressures, and these could have been used in the design. The consequence of the cohesion in the soil is that it effectively reduces the active earth pressure in an amount comparable to the 50% reduction used for retaining walls that are able to slide several inches or rotate during seismic loading. As discussed in the conclusions for these examples, in many cases the seismic coefficient values of 0.1, 0.2, and 0.3 will be equivalent to peak ground acceleration values that are twice these amounts.

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A slightly different approach was taken to represent the seismic earth pressure in this set of problems than described for the previous nongravity wall. In these problems the seismic earth pressure was represented by a uniform pressure equal to the increment of seismic earth loading rather then the total seismic load. The incremental method of representing the seismic earth pressure is commonly used within the consulting practice. As noted elsewhere in this draft final report, the actual distribution of seismic earth pressure is still poorly understood within the profession, partly because of the limited amount of experimental data demonstrating the appropriate distribution. The same p-y method could also be used to evaluate wall response assuming a uniform load for the total seismic earth pressure.

The key conclusion from the examples presented in this section is that the displacement-based approach provides a better understanding of the moments and shears that will develop during seismic loading. This conclusion applies whether incremental or total seismic earth pressures are used in the p-y representation.

Example 1: Cantilever Wall on Sloping Ground

A sensitivity analysis, using the example problem shown in Figure 1, was conducted. The retaining wall is a sheet pile pile wall constructed above a slope. The wall will support both earth and traffic loads. The traffic loads are represented by 2 feet of soil, which will be equivalent to a uniform pressure of approximately 250 psf.

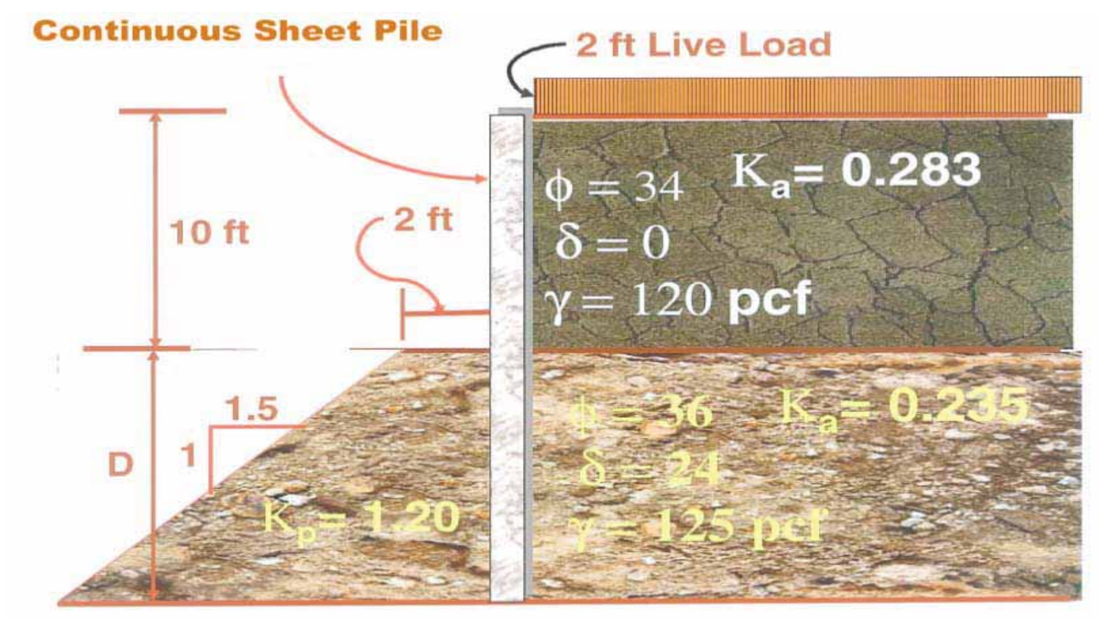

Figure 1. Example Problem of a Short Sheet pile Cantilever Retaining Wall

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Earth pressures were determined following methods given in Section 3 of AASHTO. However, rather than using load and resistance factors given in Section 3, a factor of safety was applied to the demand side of the moment equilibrium equation, consistent with methods used by Caltrans for the evaluation of nongravity cantilever retaining walls. This approach appears to result in a somewhat lower capacity to demand (C/D) ratio (lower factor of safety) than the equivalent method used in AASHTO and is consistent with methods often used in consulting practice.

The following sections describe (1) the static design methodology using limit equilibrium methods, (2) static design using the p-y method, and (3) seismic design using the p-y method.

Static Design Methodology Using Limit Equilibrium Method

The following step-by-step process was used in designing the above retaining wall for static loads:

1) **Embedment Depth:** Step 1 in the overall design process is to determine the embedment depth of the retaining wall. Conventional design procedures developed by various Department of Transportations, such as Caltrans, can be used to determine the embedment depth of the cantilever wall.

   The design process is initiated by designing for the basic static load case with conventionally adopted factors of safety (or C/D ratios). The conventional design process involves solving for the required depth of wall embedment (D) of the cantilever wall so that the moment capacity versus the moment load demand has the required factor of safety.

   Figure 2 presents the loading condition on the right side of the retaining wall associated with the active earth pressure condition, and the soil capacity on the left side of the wall associated with the passive earth pressure condition. The design parameters defined in Figure 1 (i.e., active earth pressure coefficient of 0.283 and 0.235 above and below the grade of excavation, and a passive pressure coefficient of 1.2) were used to determine the forces.

   The embedment depth is determined by equating moment load versus moment capacity about the embedded pile tip for varying tip depths until the resultant factor of safety (C/D ratio) meets the minimum value of 1.5 (i.e., moment capacity is 1.5 times the moment demand). This computation gives an embedment depth (D) of 22.5 feet.

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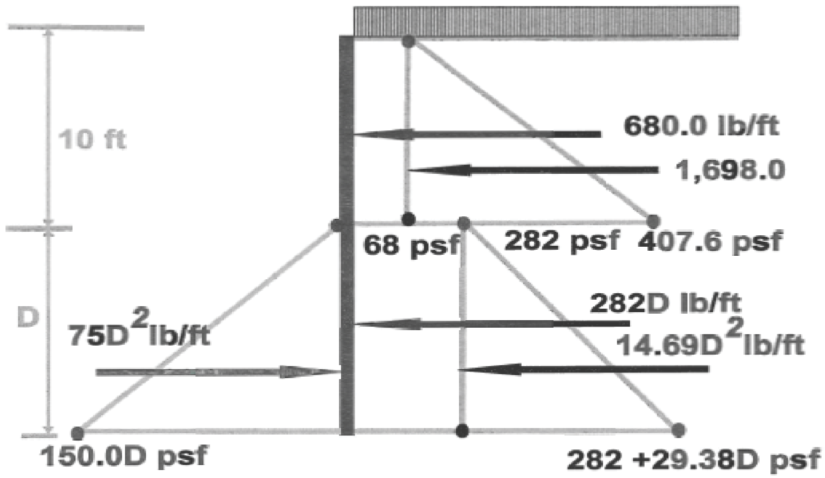

Figure 2. Moment Load vs. Moment Capacity for the Required Penetration Depth (D)

2. **Moment and Shear Determination:** Step 2 in the design process involves determining the moment and shear loads required for the cantilever retaining wall. Figure 3 depicts a common procedure used in limit equilibrium approaches to develop the maximum moment and shear load for design. This procedure involves the following:

   a. The earth pressure acting on the wall is assumed to be equal to the ultimate active and passive earth pressure conditions, as shown by the earth pressure diagram in Figure 3.

   b. The earth pressure loads are integrated to define the shear distribution acting on the retaining wall, shown in the middle of Figure 3.

   c. The shear diagram is integrated to define the moment distribution diagram as shown in the right of Figure 3.

   Shear and moment distributions from Figure 3 are then used to determine the required structural section parameters.

   The following observations can be made regarding the shear and moment distribution developed from the above limit equilibrium analysis procedure:

   a. The shear and moment loads above the point of excavation shown in Figure 3 are the ultimate active pressure condition, which will be developed for a very small degree of wall movement. The portion of the assumed earth pressure loading and wall response in shear and moment loads above the excavation level is unchanged for design, irrespective of the analysis procedure (i.e., limit equilibrium or other more advanced load-deformation procedures).

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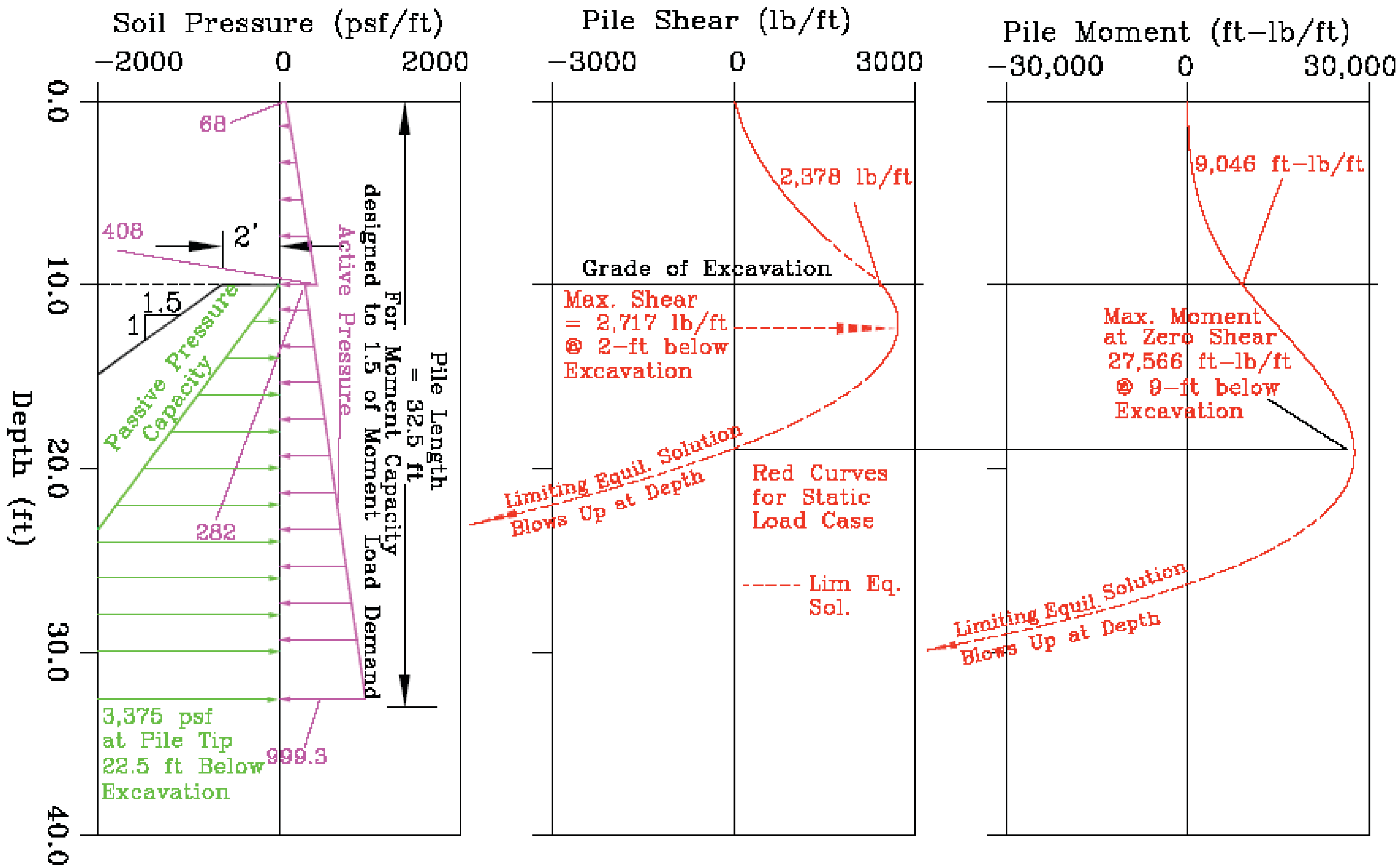

Figure 3. Conventional Limiting Equilibrium Method to Determine Design Pile Moment

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b. The net moments and shears below the excavation level increase with depth, and the design basis is not as apparent. This can lead to an implied error that grows with the assumed mobilized passive pressure value. At some depths, the moment and shear load values from this limit equilibrium analysis become unrealistic and would be meaningless. Typically, the first relative positive peak shear and moment are used for design, and the moments and shears at deeper depths from the limit equilibrium solution would be ignored.

3. **Structural Section Determination:** Step 3 involves selection of the structural section that will meet moment and shear requirements. Conventional design procedures can be used to determine the required sectional properties.

Based on this limit equilibrium solution, the maximum moment is computed to be 27,566 ft-lb per foot of wall width at a 9-foot depth beneath the excavation level, and the maximum shear is computed to be 2,717 lb per foot of wall width at 2 feet beneath the excavation level. The following calculations were conducted to determine the sectional modulus required assuming a 12-inch, 50 ksi steel sheet pile.

- Allowable stress

  = 0.55 Fy
  = 0.55 * 50 ksi
  = 27.5 ksi

- M_{max}

  = 27,566 ft-lb/ft
  = 27,566 * 12/1000
  = 330.8 in-kip/ft

- Required sectional moment of inertia, I

  = M_{max} * r / σ
  = 330.8 * 6 /27.5
  = 72.17 in^4/ft

- Bending stiffness, EI

  = 29 x 10^6 psi * 72.17 in^4/ft
  = 2.093 x 10^9 in^2-lb/ft
  = 1.453 x 10^7 ft^2-lb/ft.

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From the above calculations and using conventional design methods, the sheet pile wall for the problem depicted in Figure 1 requires use of a steel sheet pile with minimum design parameters as listed below:

- Required sheet pile length

  = 32.5 feet total, with 22.5 feet embedded length below the grade of excavation

- Required sectional moment of inertia, I

  = 72.17 in^4/ft.

- Required bending stiffness of the sheet pile, EI

  = 2.093×10^9 in^2-lb/ft.

Static Design Methodology Using P-Y Method

The p-y approach for analyzing the load-deformation response of a nongravity cantilever retaining wall is not intended to change the basic design of the cantilever wall from conventional design practice, so far as determination of the required penetration length or the choice of the sectional modulus is concerned. In practice, this design decision is normally based on the static load case, such as from the above presented design calculations, with the margin of safety associated with the static loading condition.

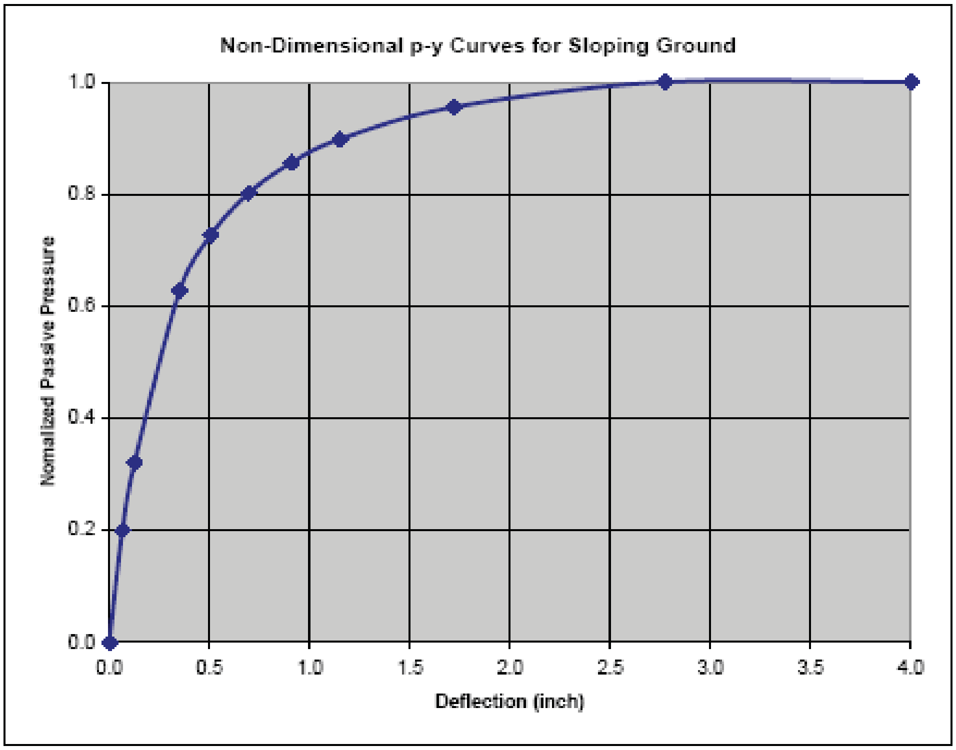
The p-y methodology provides a way to aid the structural designer in obtaining a more refined evaluation of the actual performance of a nongravity cantilever retaining wall, especially for the seismic load case. The seismic evaluation is particularly important for high seismicity areas where there may be a need to more rationally check the structural integrity of the designed wall section. Results of this load-deformation analysis can also be used to support better structural detailing required for improved seismic performance.

The same example problem shown above, but based on the p-y curve methodology, was used. In this analysis, the sheet pile wall has a 10-foot cantilevered length and a 22.5-foot embedded length. The active pressure (right side of wall in Figure 3) was prescribed as the loading condition on the pile, and nonlinear p-y curves are input along the 22.5 foot embedded portion of the pile.

The following two steps were followed for this analysis:

1. **P-y Curve Development**: In Step 1, the ultimate capacity of the p-y curves was calculated based on the passive pressure capacity solution used in the prior limit equilibrium solution (left side of wall below excavation level in Figure 3). Simplified symmetric p-y curves were adopted for the analysis. The normalized p-y curve shape was

developed using strain-wedge methods for a sloping ground configuration, as shown in Figure 4. Note that the ultimate passive pressure is not reduced by a factor of safety, which is sometimes done during limiting equilibrium analysis to account for large deformations required to mobilize passive earth pressures.



Y (inch)	0.00	0.07	0.13	0.35	0.51	0.69	0.91	1.15	1.72	2.77	4.00
p/p_{ult}	0.00	0.20	0.32	0.63	0.73	0.80	0.86	0.90	0.96	1.00	1.00

Figure 4. Normalized p-y Curve Shape for Sloping Ground Configuration as shown in Figure 1

2. **Method of Analysis:** In Step 2, displacements, shears, and moments for the sheet pile wall were determined using the computer program BMCOL which models the pile by linear elastic beam elements. [Note that BMCOL is similar to programs such as COM 624 and L-PILE.] The active earth pressure diagram shown in Figure 3 was input as external loads, identical to the basis of the limit equilibrium method. The BMCOL (or referred to as the p-y curve load-deflection method) basically differs from the limit equilibrium method in modeling the soil resistance. The limit equilibrium method assumes that the full passive pressure capacity is mobilized in the solution, irrespective of the loading condition, while the BMCOL solution models the soil resistance as p-y curves using the ultimate passive pressure capacity value to define the ultimate capacity of the p-y curves. For the BMCOL method the mobilized soil resistance is iterated in the analysis until resistance is compatible to what is required to resist the applied load (to satisfy both shear and moment equilibrium) and compatible to the deformation of the pile.

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Figure 5 presents the comparison between the conventional limit equilibrium and the mechanistic p-y method (load-deformation) for static loading condition. These results show the following:

- Limit equilibrium solutions apparently do a good job in designing for the conventional static load case in terms of estimating the maximum moment demand for evaluation of the required sectional modulus. The comparison shows a design moment within 5% of the load-deflection solution. It should be pointed out that this may be from the fact that the loading condition is unchanged between both approaches for the wall above the excavation level, as depicted in Figure 5. Probably this condition remains reasonably valid within the first 10 feet of the embedded portion of the nongravity sheet pile wall, when the pile deflection is sufficient (over 1 inch) to mobilize up to 90% of the ultimate passive pressure capacity in the BMCOL solution.

- Error in the limit equilibrium solution increases with depth, when the resultant wall response would be increasingly affected by the assumed passive earth pressure response. The limit equilibrium solution assumes that the ultimate passive pressure is fully mobilized along the entire embedded wall. Such an assumption may be reasonable up to about a 10-foot depth below the excavation grade where sufficiently large (say about 1-inch wall deflection) was calculated from the p-y model as shown in the presented load-deflection solutions. However, error increases rapidly with depth when the wall deflection becomes very small due to the deformability of the wall.

- The error in the limit equilibrium solution discussed above can lead to erroneous design shear loads, for even the static loading condition. It can be seen from Figure 5 that designers often assume a 2,717 lb per foot shear load for design, while the more accurate load-deflection solution shows that the maximum shear would occur at a deeper portion of the wall, with a correct maximum negative shear value of 3,415 lb per foot (or over 25% in error relative to the limit equilibrium analysis).

- Generally, the limit equilibrium approach does not provide for a rational basis for a performance-based design process, which fundamentally requires predicting deformation or strain in the design. Also, the ductility design principal is fundamental in designing for the earthquake loading conditions (as opposed to the conventional static dead load) within the structural engineering community. There is no provision for the limit equilibrium procedure to support structural engineers as they analyze nongravity cantilever walls beyond the linear range of the wall and soil behavior. Refinement towards the proposed p-y curve and load-deflection basis will be necessary for defining the nonlinear ductility design range.

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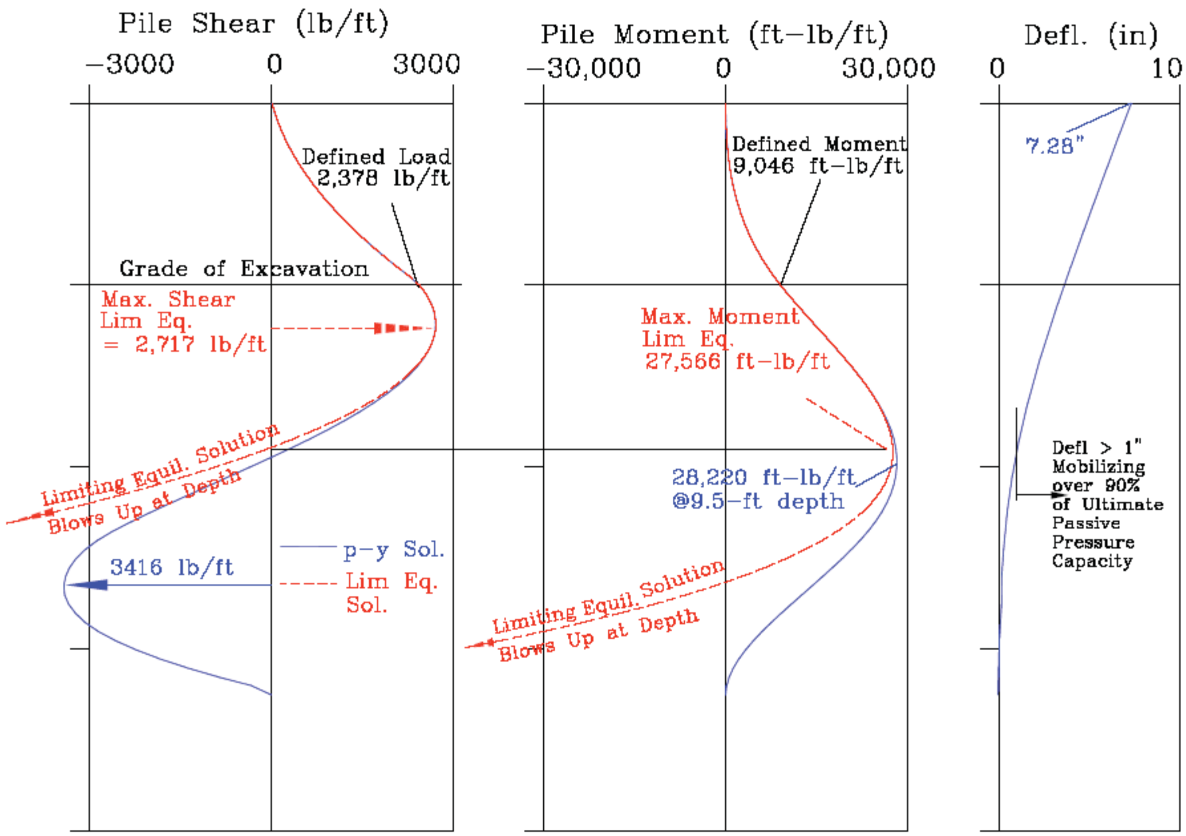

Figure 5. Comparison Between p-y Curve and Limiting Equilibrium for Static Load Case

Seismic Design Methodology Using P-Y Method

In addition to the static load case, additional solutions were developed to provide examples for the earthquake design problem. For these examples seismic ground acceleration coefficients of 0.1, 0.2, and 0.4 were used to develop pseudo-static, horizontal loads that were applied to the upper 10 feet of wall above the excavation level. This pseudo-static load (treated as a uniform pressure distributed over the 10-foot cantilever wall height) is superimposed on the static active earth pressure load defined in Figure 3.

For this analyses the net pseudo-static dynamic (earthquake) load effect in the Mononobe-Okabe solution was isolated from the inherent static earth pressure load. Figure 6 has been extracted from Appendix C in the proposed Specifications to develop the net seismic coefficient (the lower of the three lines) implied by the conventional total load Mononobe Okabe earth pressure theory. The net seismic coefficient was obtained by subtracting the static load coefficient from the total load coefficient. As shown in the following figure, the net dynamic earth pressure coefficient would be 0.04, 0.1, and 0.3, respectively, for a seismic coefficient k_{max} = 0.1, 0.2, and 0.4.

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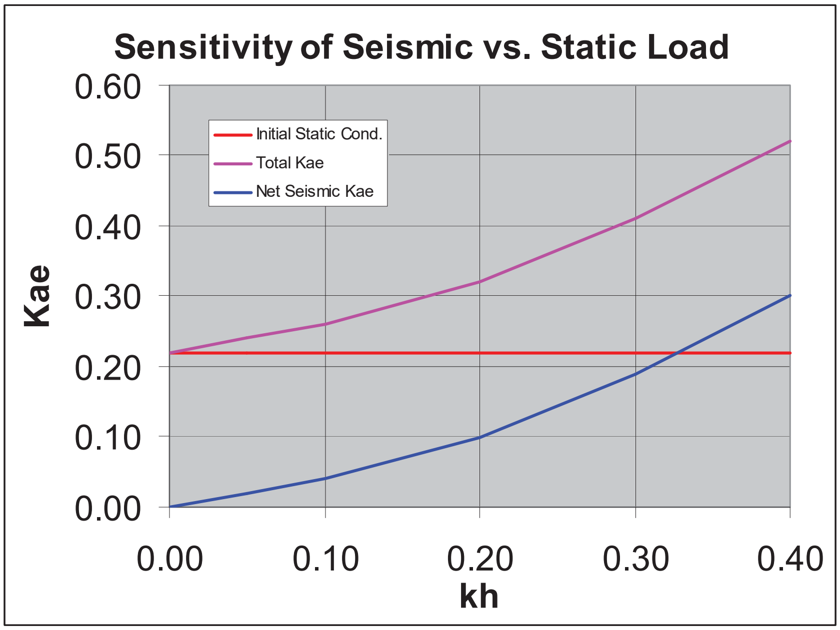

Figure 6. Net Seismic Load implicit in the Mononobe-Okabe's Earth Pressure Coefficient ($k_h = k_{max}$)

Table 1. Tabulation of Dynamic EQ Load Case Calculations

Seismic Coefficient, k_{max}	**0.1**	**0.2**	**0.4**
Net EQ Earth Pressure Coefficient, $K_{ae\ (Net\ EQ)}$ from Figure 6	0.04	0.1	0.3
Total EQ Load on Wall (lb/ft) = ½ $\gamma H^2 K_{ae\ (Net\ EQ)}$	240	600	1800
Distributed Pressure (psf)	24	60	180

The total incremental dynamic earthquake loads tabulated in Table 1 were superimposed on the static earth pressure diagram shown in Figure 3. The incremental dynamic earthquake earth pressure load was modeled as a uniform pressure distribution acting over the 10-foot cantilever wall height as shown in Figure 7. The resultant solutions for the k_{max} of 0.2 and 0.4 load cases were plotted in Figure 8. These results can be compared to the static earth pressure load case. The figure also provides comparison between the p-y solutions and the corresponding limit equilibrium solutions.

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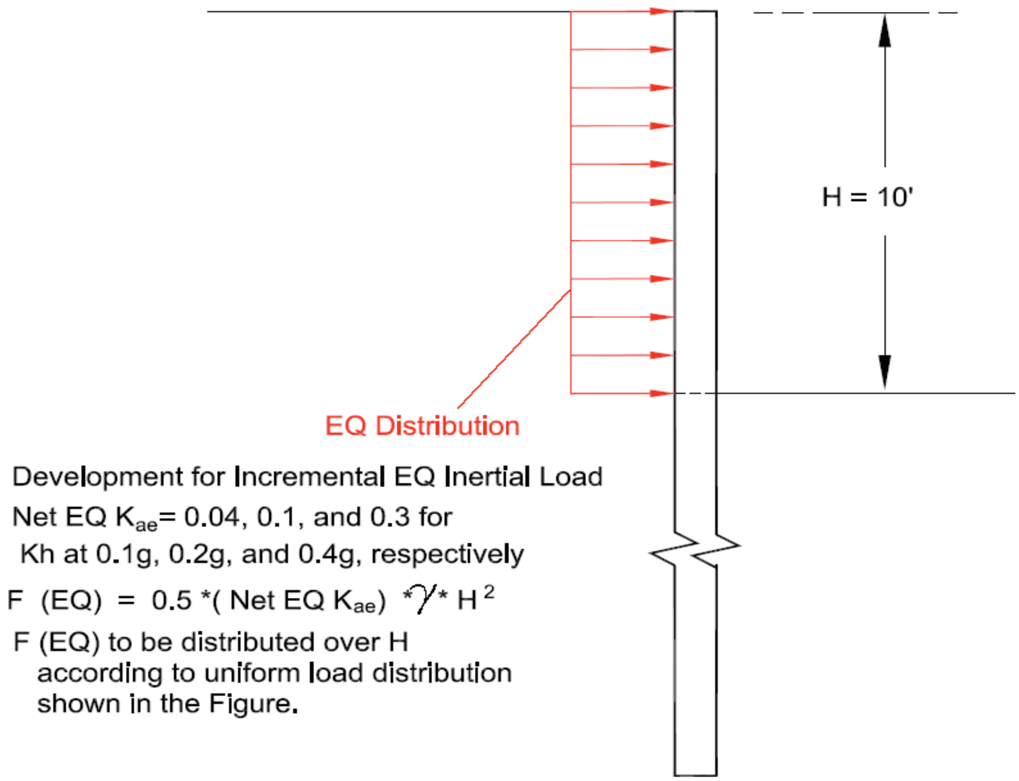

Figure 7. Dynamic Load Case Superimposed on the Static Earth Pressure Load Case

The following observations were made from the results presented in Figure 8.

- The static load case constituted a significant part of the overall load, inducing higher moment than the earthquake load case for a k_{max} of up to 0.2 (which might corresponds to a site-adjusted PGA coefficient of up to 0.4 under an assumption that k_{max} = 0.5 F_{pga} PGA).

- There is a rather high degree of conservatism in the structural design practice for defining the allowable stress (typically at about 0.55 to 0.6 of ultimate stress). In the above example of $F_{allowable}$ = 0.55 $F_{y,}$, there is a margin of reserve in the structural capacity of 1.82 times the allowable stress value. This reserve capacity appears to provide for adequate performance of the cantilever wall structurally to a very high earthquake load, well above the projected demand for most of the high seismicity regions of the U.S. For the example problem, the designed wall section modulus will only start to approach an ultimate yield stress at a pseudo-static seismic coefficient approaching 0.4 (which may correspond to a PGA of close to 0.8 if there is cohesion in the soil, as would often be the case for this type of wall). However, it can be observed that wall displacement can become rather high from the presented solution. This displacement would decrease if cohesion were also included in the passive earth pressure determination.

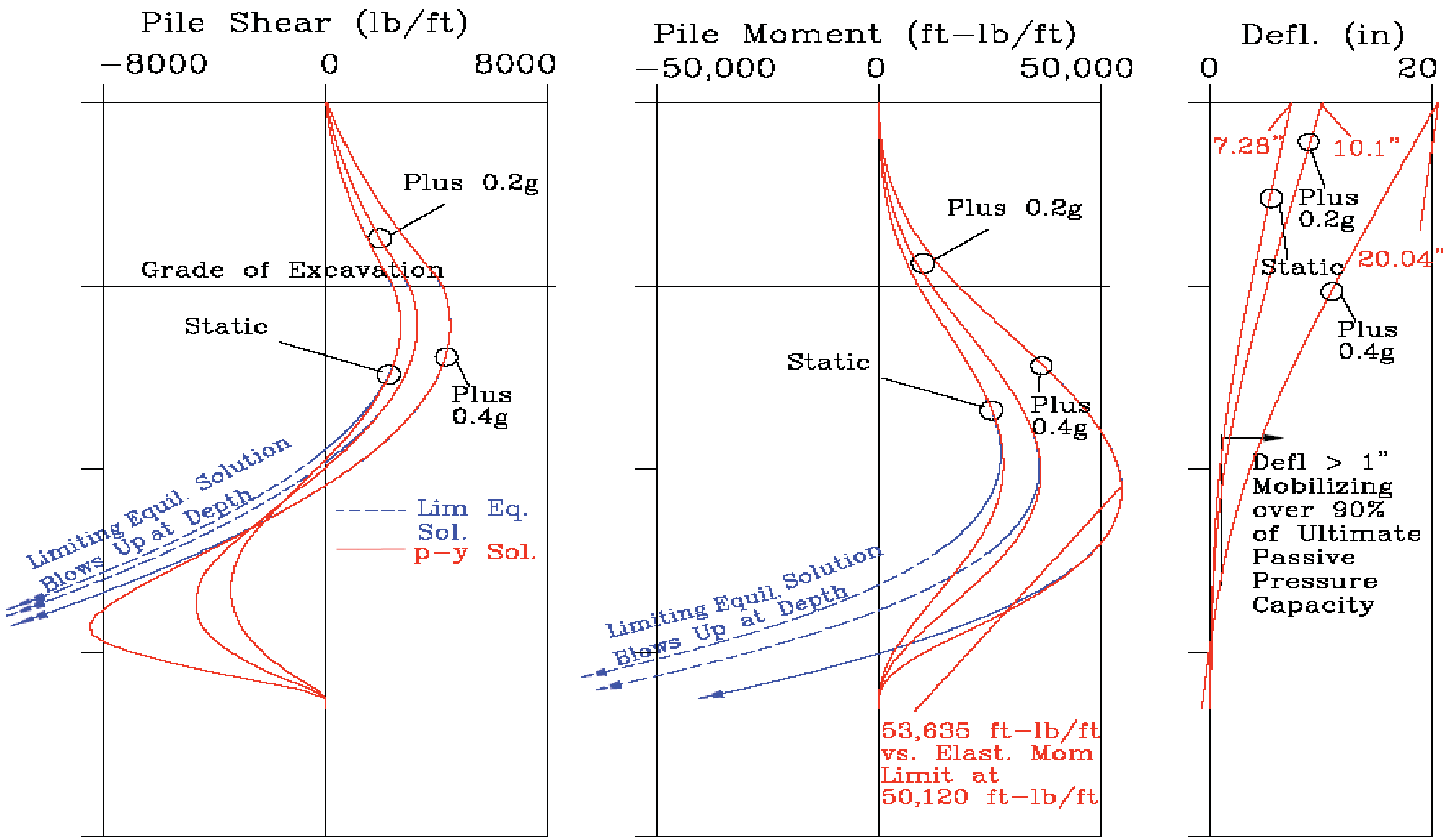

Figure 8. Comparison Between p-y Curve and Limiting Equilibrium for k_{max} = 0.4.

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Example No. 2: Cantilever Sheet Pile Wall On Level Ground

This example is essentially the same as Example No. 1 except that the sloping ground configuration (commonly encountered in road widening projects) is changed to a classical level ground configuration. Other parameters remain unchanged except that the passive pressure capacity of the soil below the excavation level becomes much larger; i.e., the passive pressure coefficient (K_p) changes from 1.2 for the sloping ground condition to a K_p equal to 8.26 for the level ground configuration.

Static Design Methodology

The design process for this level ground sheet pile problem follows the same steps as described for Example No. 1. The higher passive pressure capacity leads to a much shorter sheet pile embedment depth. Repeating the steps in the moment equilibrium calculation with the much higher K_p (using 8.26 as oppose to 1.2) leads to a required sheet pile embedment depth of 6.55 feet below the excavation level. This depth satisfies the requirement that the ultimate passive pressure capacity is 1.5 times the active pressure load based on moment equilibrium about the sheet pile tip. For design the sheet pile embedment depth is rounded up to 7 feet in the example problem.

Based on a limit equilibrium solution, the maximum moment is computed to be 13,371 ft-lb per foot of wall width at 2.5 feet beneath the excavation level, and the maximum shear is computed to be 2,449 lb per foot of wall width at the excavation level. Assuming a 12-inch, 50 ksi steel sheet pile, the following calculations were conducted to solve for the required sheet pile sections:

- M_{max}

  = 13,371 ft-lb/ft
  = 13,371*12/1000
  = 160.5 in-kip/ft

- Required sectional moment of inertia, I

  = M_{max} * r / σ
  = 160.5 * 6 /27.5
  = 35 in^4/ft

- Bending stiffness, EI

  = 29 x 10^6 psi * 35 in4/ft
  = 1.015 x 10^9 in^2-lb/ft
  = 7.049 x 10^6 ft^2-lb/ft.

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From the above calculations and using conventional design methods, the sheet pile wall requires use of a steel sheet pile with minimum design parameters as listed below:

- Required sheet pile length

  = 17 feet total with 7 feet of embedded length below the level of excavation.

- The required sectional moment of inertia, I

  = 35 in^4/ft.

- The required bending stiffness of the steel beam, EI

  = 1.015 x 10^9 in^2-lb/ft.

Design Using P-Y Methodology

The beam-column analyses were conducted with the normalized p-y curves defined in Figure 4, but the resistance on the p-y curves was scaled to reflect the much higher passive pressure capacity. The resultant p-y curves have a capacity increasing linearly from zero at the excavation level to an ultimate capacity equal to 7225.5 psf at the pile tip at 7-foot depth below the excavation grade.

Figure 9 presents the beam-column solution for the level ground design problem. The much larger passive pressure capacity is shown on the left side of the pressure diagram, and pile shear, pile moment and deflection solutions are depicted in the same figure, with the solutions for static loading shown as the minimum in each data set, followed by successive incremental earthquake-induced earth pressure loading (using uniform pressure distribution) for k_{max} = 0.1, 0.2 and 0.24.

Results of the sensitivity study shows that there is insufficient soil capacity for this 7-foot penetration to withstand an earthquake load above 0.24g (corresponding to 84 psf uniform incremental pressure above the static loading condition) based on a pseudo-static representation (i.e., the earthquake load is sustained with time). Again, comparison between the beam-column (p-y) solution and the conventional limit equilibrium solution shows that the conventional solution leads to reasonable maximum pile moment load for design. However, this solution again shows that the conventional limit equilibrium method may not provide correct maximum shear values for design. A maximum negative shear of 5,235 lb per foot should be used for design as opposed to the maximum positive shear value at 2,449 lb per foot that might be adopted in practice from the limit equilibrium approach.

When the deflection profile in Figure 9 is compared to Figure 8 for the deeper sheet pile problem for sloping ground, one can observe the sheetpiles approach a rigid pile solution for the 7-foot embedment depth condition as opposed to the 22.5-foot embedment depth problem. There is a much more noticeable toe kick-out behavior for the shorter pile problem. Also the 1.5 factor

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safety in static design appears to imply a lower reserve so far as global equilibrium is concerned. The example problem in Figure 9 shows that if the sheet pile wall is designed for a factor safety of 1.5 for the static gravity load, the resultant design can only sustain a horizontal seismic coefficient k_{max} of up to about 0.24, as oppose to over 0.4 for the earlier sheet pile wall embedded to 22.5-foot depth.

It should also be pointed out that it is common practice to account for the above potential problems associated with short embedded sheet pile walls by various rules such as increasing the embedment depth of short walls by a 1.2 length factor, or also by specifying minimum default active earth pressure loads for design. Unfortunately, there does not appear to be clear guidance on what should be defined as short versus long walls and when the length factor of 1.2 should be applied. In contrast the beam column solution provides a good means for actually inputting all the relevant parameters (i.e., soil pressure, passive pressure capacity along with the elastic properties of the pile itself) to solve for the deflection profile, which should give the designer a better appreciation for whether the pile-soil system will be closer to a rigid pile or a long flexible pile.

Example No. 3: Soldier Pile Wall Problem

So far, the two examples involved continuous sheet pile walls. Another commonly encountered condition is the soldier pile problem as shown in Figure 10 below.

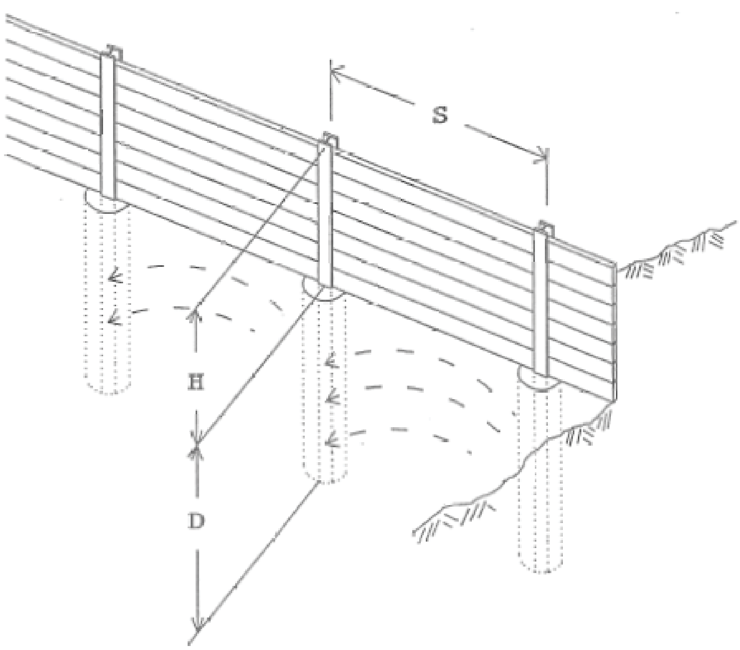

Figure 10. Configuration of the Soldier Pile Problem

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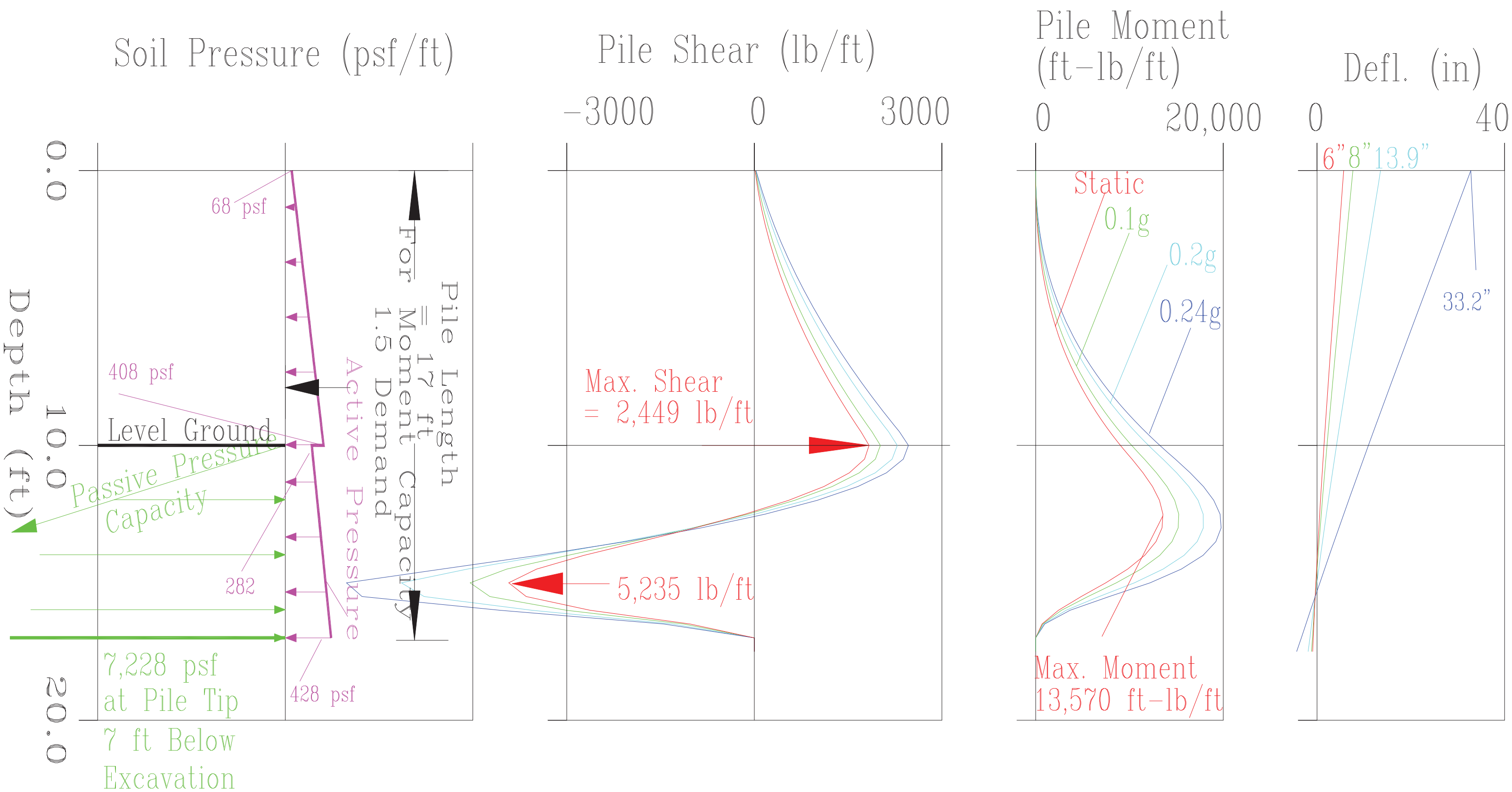

Figure 9. Beam-Column Solution for Sheet Pile Wall Embedded in Level Ground

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The soldier pile wall as shown in Figure 10 is normally designed with some form of lagging – normally timber but sometimes steel plate or concrete. The soldier piles can consist of steel HP or wide flange sections, driven concrete piles, or cast-in-drilled-hole (CIDH) piles. Again, this design problem differs from the two previous example problems in regards to how to model the passive soil pressure below the excavation grade.

Static Design Using P-Y Methodology

The appropriate earth pressure theory for solving for the passive pressure capacity involves accounting for the 3-dimensional pile-soil configuration as opposed to the previous plane-strain configuration. The state-of-the-practice in pile design for the above circular or rectangular pile configuration generally follows the practice in the offshore oil industry. The passive soil reaction for the soil-pile system generally begins with p-y curves developed from empirical, single solitary pile load tests. P-y curves for sands and clays developed by Reese and Matlock are the most widely adopted criteria for pile design as documented in American Petroleum Institute pile design guidelines (API, 1993). This approach is followed in programs such as L-PILE or COM 624.

As shown in Figure 10, when a row of soldier piles is constructed, greater soil resistance is realized than the soil resistance directly in front of the pile. This additional component of soil resistance is often referred as the soil arching effect. To account for this additional component of soil resistance, above what is expected from a single isolated pile, a so called effective pile width greater than the actual pile dimension is used. This is commonly referred as the arching factor (or arching capacity factor). Figure 11 presents arching factors from Caltrans Trenching and Shoring Manual for illustration.

It is necessary to estimate the appropriate p-y curves in this soldier pile problem. The following methodology was used for constructing the p-y curves in the beam-column analysis for this soldier pile wall problem.

- The foundation soils consist of sand with a 36-degree friction angle.
- Following Figure 11, the arching factor was initially calculated based on the equation 0.08 ϕ = 0.08 x 36 = 2.88. Alternately, one can scale the pile width by this 2.88 factor in calculating the passive soil pressure capacity.
- Following common practice, a 24-inch CIDH pile spaced 8-feet apart (center-to-center) was assumed for the problem.
- The first trial involved following the API code for developing p-y curves for a 24 x 2.88 = 69-inch diameter pile.

SOLDIER PILES

GUIDELINES FOR REVIEW OF SOLDIER PILES PASSIVE ARCHING CAPACITIES

GRANULAR SOILS

COMPACTEDNESS	VERY LOOSE		LOOSE		MEDIUM		DENSE		VERY DENSE
Relative Density, D		15%		35%		65%		85%	
Standard Penetration Resistance, N = Blows/ft		4		10		30		50	
Angle of Internal Angle, ϕ (Degree)		28		30		36		41	
Unit Weight (pcf) Moist	100		95-125		110-130		110-140		130+
Submerged	60		55-65		60-70		65-85		75+
Arching Capacity	0.08 ϕ		0.08 ϕ		0.08 ϕ		0.08 ϕ		0.08 ϕ

COHESIVE SOILS

CONSISTENCY	VERY SOFT		SOFT		MEDIUM		STIFF		VERY STIFF		HARD
Unconfined Comp. Strength, q_u (psf)		500		1000		2000		4000		8000	
Unit Weight (pcf) Saturated		100-120			110-130			120-140			130+
Arching Capacity		1 to 2		1 to 2		2		2		2	

Figure 11. Arching Factor Extracted from Caltrans Trenching and Shoring Manual

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- The ultimate passive pressure capacity based on the p-y procedure in the API code is then plotted in Figure 12. Note that the passive pressure value of the p-y curve (p_{ult}) shown in the figure is in FL^{-1} (lb/in) representing the lumped soil resistance acting over the overall pile diameter. The ultimate passive pressure capacity is based on an arching factor of 2.88.

Results in Figure 12 show that at a 7-foot depth (the depth required for static global stability in the earlier level ground sheet pile wall problem) the ultimate passive pressure capacity p_{ult} value would be equal to 6,354 lb/in. After distributing this to the 8-foot pile spacing, the corresponding ultimate passive pressure capacity expressed in terms of pressure would be 66.2 psi, or 9,531 psf.

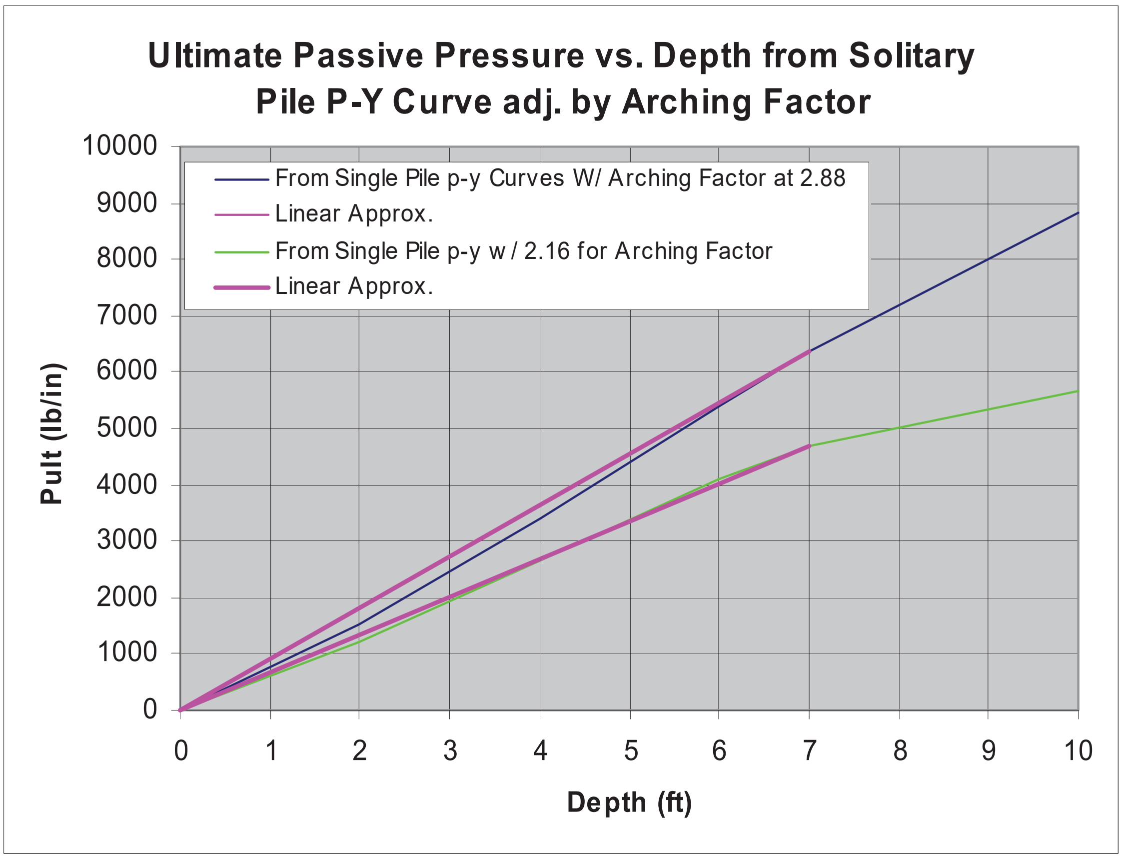

Figure 12. Ultimate Passive Pressure versus Depth of Penetration from API p-y curve Procedures

From Figure 9 the theoretical ultimate passive pressure capacity would only be 7,226 psf if the piles are at the closest spacing (i.e., continuous). This comparison shows that there is tremendous uncertainty in the Caltrans' arching factor procedure. From a theoretical point of view, the resultant ultimate passive pressure capacity should not be higher than the capacity associated with the theoretical limit corresponding plane-strain theory.

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The above comparison suggests that the assumed arching factor of 2.88 might be overly optimistic. Hence, the arching factor was reduced by the ratio of 7226/9531 = 0.75. The revised arching factor would then be 2.88 x 0.75 = 2.16, which leads to an effective pile width of 24 x 2.16 = 51.84 inch. The revised calculation used the 51.84 inch pile diameter, which shows a reduction in ultimate passive pressure roughly equal to the theoretical plane strain solution. This revised ultimate passive pressure capacity value is shown in the lower line in Figure 12. The resultant single pile p-y curves are then divided by the 8-foot pile spacing for a beam column solution which models the pile-soil system curve on a per foot.

In addition to the p-y curve model, the bending pile stiffness (EI) in the pile model beneath the excavation level also needs to be changed based on the 24-inch CIDH pile. The following documents the EI value used in the beam-column analysis.

- Moment of inertia (I) for 24-inch circular CIDH pile

  $= \pi R^4/4 = 16{,}286 \text{ in}^4$

- Youngs modulus of concrete (E) for concrete strength, f_c'

  = 5,000 psi,

- Youngs modulus of concrete, E

  $= 59{,}000\ \theta\ f_c' = 4.172 \times 10^6$ psi

- Resultant effective EI

  $= 0.5$ Gross EI $= 0.5 \times 4.172 \times 10^6$ psi $\times$ $16{,}286 \text{ in}^4$

  $= 3.4 \times 10^{10}$ in^2-lb per pile

  Normalizing EI for the 8-foot spacing

  $= 3.4 \times 10^{10}/8 = 4.25 \times 10^9$ in^2-lb per ft wall width

The above soldier pile EI can be compared to the corresponding EI for the sheet pile wall problem in Example 2 where the EI = 1.015×10^9 in^2-lb per ft wall width. For this problem the soldier pile wall has stiffer piles on a per unit width basis than the sheet pile wall.

Figure 13 presents the beam-column solution for a level ground design condition. As described earlier, the ultimate passive pressure capacity, after accounting for the soil arching effect, would be approximately the same as the sheet pile wall (plane-strain) passive pressure theory in Example 2. Hence, the same 7-foot pile penetration length below the excavation level is adopted in this example. The Example 3 model is changed from the earlier Example 2 problem in terms of the p-y curves and the pile bending stiffness EI. Details for the calculations have been presented earlier.

Results of P-Y Analyses

The solutions for static loading are shown as the lowest set of curves in Figure 13, followed by successive incremental earthquake-induced earth pressure loading for 0.1g and 0.2g. Comparison between the beam-column (p-y) solution with the conventional limit equilibrium solution shows that the conventional solution leads to reasonable maximum pile moment load for design. However, the results again show that the conventional limit equilibrium method may not provide correct maximum shear values for design. A maximum negative shear of 5,216 lb per foot should be used for design as opposed to the maximum positive shear value at 2,449 lb per foot commonly adopted in practice from limit equilibrium approaches.

When compared to Figure 9, the deflections for this soldier pile problem are lower for the static load case and for k_{max} = 0.1 seismic load case. However, the deflections become larger for k_{max} = 0.2g seismic load case. This response is due to the higher equivalent EI value for the soldier pile compared to the sheet pile wall in Example 2, which leads to a lower deflection for the cases involving lower load values. However, at the higher load values, the passive pressure for the soldier pile problem is slightly lower than the sheet pile wall problem, which leads to higher deflection, and eventually to a global overturning failure for seismic coefficients above k_{max} = 0.2 (corresponding to uniform pressure of 84 psf added to the static load case).

Again, the example problem illustrates how beam-column load-deflection solutions (which have become standard geotechnical analysis practice) can provide better insight to designers about how different design parameters affect the overall design.

Concluding Comments

The solutions for these examples start to explain why free-standing retaining walls have performed well in past earthquakes, even though in many cases there were no provisions for earthquake loading. Most cantilever walls will have an adequate margin of capacity from static design to resist moderate levels of loading, particularly when the effects of soil cohesion on active and passive earth pressures are considered. From these analyses, there is just cause to assume that conventionally designed, free-standing retaining walls for the static load case have sufficient reserve for many earthquake load cases, and complex earthquake design analyses for retaining walls are usually unnecessary.

References

API (1993). "Recommended Practice for Planning, Design and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design," American Petroleum Institute, Recommended Practice 2A-LRFD, First Edition, July.

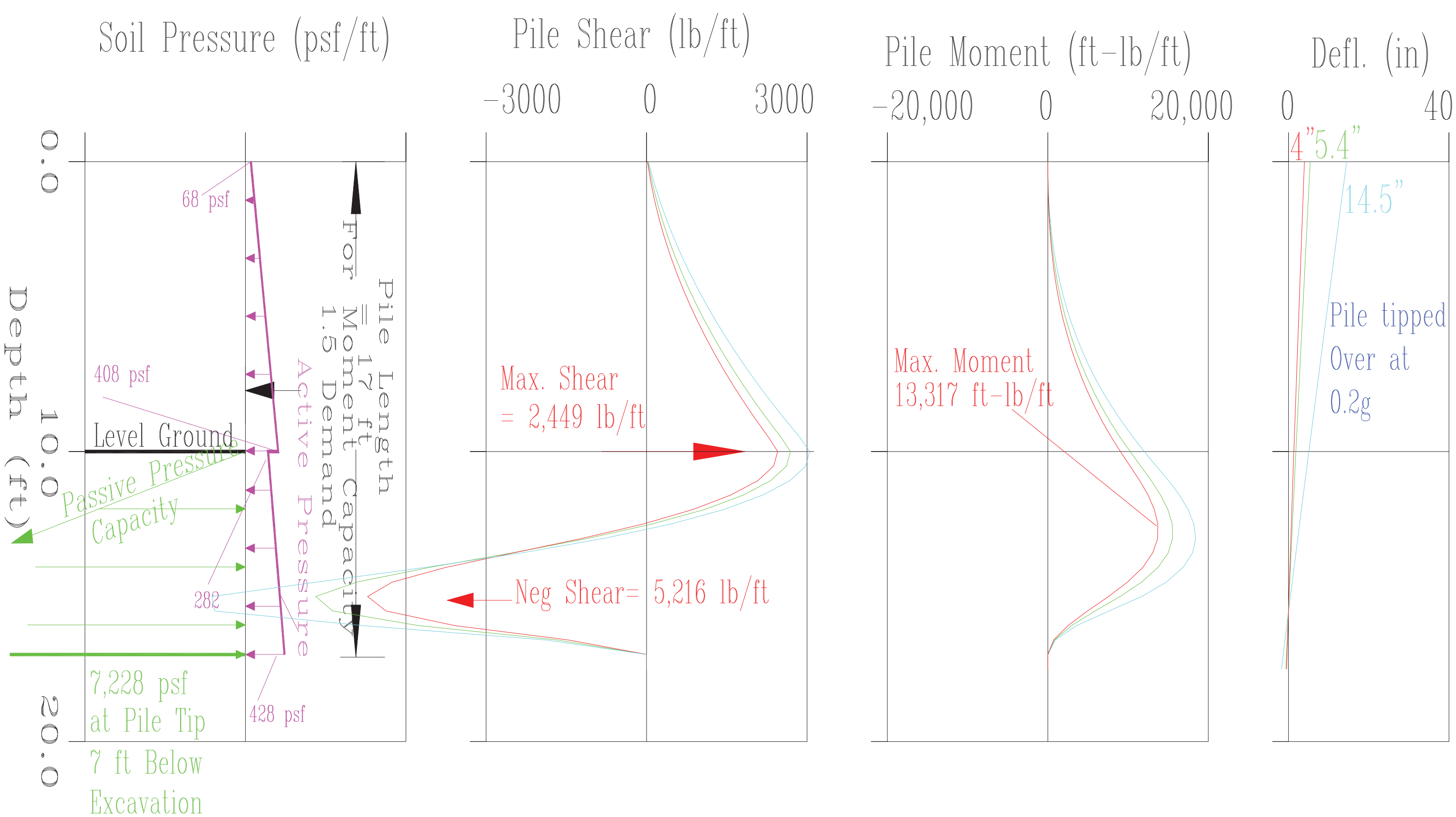

Figure 13. Beam-Column Solution for Soldier Pile Wall in Level Ground

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Example Anchored Wall Problem – Limit Equilibrium Method

Introduction

This example demonstrates application of procedures outlined in Section X.9 of the proposed Specifications for the seismic design of an anchored soldier pile wall. The example focuses on calculation of tieback loads and location of the anchor zone. The static design follows methods in the 4th Edition of the AASHTO *LRFD Bridge Design Specifications*. The following subsections summarize (1) the wall geometry and soil properties used in the example, (2) the seismicity for the site considered, (3) the general methodology followed, (4) the results of the analyses, and (5) concluding comments about these analyses. The next problem in this set of example problems demonstrates the use of p-y approach for evaluating the response of an anchored wall.

Wall Geometry and Soil Properties

The geometry of the wall is shown on Figure 1. As shown, the wall height at the face of the wall is 50 feet, but a very steep 9-foot high slope occurs above the wall, and a heavily traveled roadway occurs immediately next to the slope. It is assumed that the use of the roadway is such that the live load should be considered in design.

The native silty sand in the cut profile has a friction angle of 32°, cohesion of 200 psf, and unit weight of 120 pcf. This soil layer overlies a dense alluvium with a friction angle of 40° and unit weight of 130 pcf. A 6-foot deep tension crack zone was assumed to occur at the top of the slope.

The soil properties assigned to the soil behind the wall represent conditions associated with a partially cemented silty sand. The effect of the cohesion component of the native soil is particularly significant during seismic loading.

Seismicity

The site-adjusted Peak Ground Acceleration coefficient (PGA) for the site (assumed Site Class D) was 0.50, and $k_{max} = F_{pga}$ PGA = 1.0 * 0.5 = 0.5. The corresponding value of $F_vS_1 = 0.5$.

These seismic conditions were assumed. For an actual site the PGA and S_1 values would be obtained from the AASHTO ground motion hazard maps using the interactive CD. The AASHTO maps provide ground motion on rock (Site Class B). Since the assumed Site Class is D, the PGA and S_1 values need to be adjusted by F_a and F_v given in the 2008 Interim Revisions to the AASHTO Specifications.

Methodology

Before applying the seismic design evaluation, an initial static design was established following AASHTO Specifications, as required by Section X.9 of the proposed Specifications. Once the

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static design was completed, a check on the wall performance during seismic loading was performed. This check included determining the seismic active earth pressure, checking whether the forces from the seismic earth pressure would exceed the anchor capacities, and then performing an external stability analyses to confirm that the capacity to demand (C/D) ratio was greater than 1.0.

Since the wall height was greater than 20 feet, the PGA was adjusted for wall-height effects using the procedure recommended in the proposed Specifications. Based on these recommendations, the seismic acceleration was adjusted using the following equation:

$$k_{av} = \alpha\, k_{max} = \alpha\, F_{pga}\, PGA$$

$$\alpha = 1 + 0.01\, H\, [0.5\, \beta) - 1]$$

where H is the wall height in feet and β is calculated from the equation:

$$\beta = F_v\, S_1 / k_{max} = 0.5/0.5 = 1.0$$

For H = 59 feet and $\alpha = 0.7$ the resulting seismic coefficient, $k_{av} = \alpha * k_{max} = 0.35g$ was used in pseudo-static seismic limit equilibrium analyses as allowed in the proposed Specifications.

Results of Analyses – Static Design

Prior to evaluating the seismic design, an initial static design was established as outlined below. The static design followed the requirements in Sections 3 and 11 of the AASHTO *LRFD Bridge Design Specifications.*

Static Earth Pressure and Anchor Design for Static Loading

Development of static earth pressures behind tieback walls is complex and is affected by various parameters such as the method and sequence of construction, number of tiebacks, the relative stiffness of soil, wall and tieback system, and anchor preloads. Due to this complexity tieback walls are commonly designed using prescribed static earth pressure distributions. Some of these pressure distributions are based on measurement of earth pressure on anchored or strutted walls. Others are obtained by analytical means or scaled models.

The common theme among these pressure distributions is the presence of higher lateral pressures near the top of the wall compared to active earth pressure distributions. AASHTO recommends prescribed apparent earth pressure distributions for single-anchor and multi-anchor walls in cohesionless and cohesive soils. For this example the pressure distribution recommended by AASHTO for a multi-anchor wall in cohesionless soil was used. The resulting pressure distribution is shown in Figure 2.

The maximum ordinate of the pressure distribution in Figure 2 was estimated from the following equation:

$$p_a = \frac{k_a \gamma_s' H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}}$$

where:

p_a = maximum ordinate of pressure diagram
k_a = active earth pressure coefficient
γ'_s = effective unit weight of soil
H = total excavation depth
H_1 = distance from ground surface to uppermost ground anchor
H_{n+1} = distance from base of excavation to lowermost ground anchor

The active earth pressure coefficient for simple cases can be calculated using either Coulomb or Rankine equations. For a general case with surcharge loading, tension cracks, and a frictional-cohesive soil, as occurs in this example, the trial wedge method can be used to estimate the total active pressure force (P_a). This method is discussed in Appendix B_X of the proposed Specifications. Alternatively, the generalized limit equilibrium method using a slope stability analysis program can be used to estimate total active earth pressure.

For this example, the trial wedge method was used to calculate P_a. The trial wedge method for a general static case is schematically illustrated in Figure 3. The total earth pressure force on the wall (P_A) was calculated from the following equation:

$$P_A = \frac{W \cdot \tan(\phi - \alpha) - c \cdot L_C \left[\tan(\phi - \alpha) \cdot \sin\alpha + \cos\alpha\right] - c_a \cdot L_a \left[\tan(\alpha - \phi) \cdot \cos\omega - \sin\omega\right]}{\left[1 + \tan(\delta + \omega) \cdot \tan(\phi + \alpha)\right] \cdot \cos(\delta + \omega)}$$

where:

W = total weight of the active wedge, including surcharge
ϕ = soil internal friction angle
c = soil cohesion
c_a = soil / wall adhesion
δ = soil / wall friction angle
α = angle of failure plane measured from horizontal plane
ω = angle of wall face measured from vertical plane

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L_a = length of wall face below tension crack

L_c = length of the failure plane

P_A is calculated for different values of angle α. The maximum P_A corresponds to the total active earth pressure force on the wall.

Using this method, the total active pressure force was calculated as P_a = 52.5 kips. A modified form of AASHTO equation was used to estimate the maximum ordinate of the pressure distribution:

$$p_a = \frac{2 \times P_a}{1.5H - 0.5H_1 - 0.5H_{n+1}}$$

Using this equation and dimensions shown in Figure 1, the maximum ordinate of the static pressure distribution was evaluated as:

$$p_a = \frac{2 \times 52.5}{1.5 \times 50 - 0.5 \times 8 - 0.5 \times 12} = 1.62 \cdot ksf$$

The horizontal component of design load at each tieback can be calculated from the Tributary Area Method or Hinge Method, as explained in AASHTO (2007). The hinge method is used here to determine the anchor loads and minimum embedment depth of the wall. In determining the minimum embedment depth of the wall for the static case, a load factor of 1.35 was applied for the active earth pressure, and a resistance factor of 0.75 was used for the passive pressure, as shown on Figure 4. In the Hinge Method, it was assumed that there is a hinge in the wall at all anchor locations except the top one (Figure 4). The load in Anchor 1 is calculated by taking the bending moment about Point 2 and setting the moment of all forces above this point to zero. Then the load in Anchor 2 is calculated by setting the bending moment about Point 3 to zero. The minimum embedment depth is calculated by balancing the bending moment of all forces about Point 3. Finally, the load in the last anchor (Row 3) is calculated by setting the sum of horizontal forces to zero.

Using this method, the corresponding unfactored loads in the tiebacks for unit width of the wall are calculated as:

t_1 = 22.46 kip
t_2 = 22.63 kip
t_3 = 23.70 kip

The minimum required embedment depth is 10.9 feet. An embedment depth of 12 feet was used in the subsequent calculations, anticipating additional depth for seismic loads. Assuming the anchors are 8 feet apart in the horizontal direction:

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$T_{h1} = 8 \times t_1 = 180$ kip ➔ $T_1 = 180 / \cos 15° = 186$ kip
$T_{h2} = 8 \times t_2 = 181$ kip ➔ $T_2 = 181 / \cos 15° = 187$ kip
$T_{h3} = 8 \times t_3 = 190$ kip ➔ $T_3 = 190 / \cos 15° = 197$ kip

Anchor Test Loads

All anchors should be tested to a load equal to or larger than factored anchor loads. AASHTO specifies a test load factor of 1.35 for apparent earth pressure on anchored walls; therefore, the minimum test load of anchors is 135% of the lock off load (normally the calculated design load). In practice, however, for permanent walls a larger factor of 1.5 is usually used for testing.

Using a load factor of 1.5, the factored loads are calculated as:

$T_1 = 279$ kip
$T_2 = 281$ kip
$T_3 = 296$ kip

The corresponding vertical component of the anchor loads are calculated as:

$T_{v1} = 279 \times \sin 15° = 72$ kip
$T_{v2} = 281 \times \sin 15° = 73$ kip
$T_{v3} = 296 \times \sin 15° = 77$ kip

Total vertical load in the soldier pile is therefore the sum of these vertical components, which is equal to 222 kips. This vertical load is assumed to be resisted by the embedded section of the soldier pile. It is important to check the capacity of the soldier pile to ensure that it is larger than this demand because any vertical settlement of the soldier pile will reduce the tension in the anchors. The details of this calculation, however, are outside of scope of this example.

Anchor Length

The critical active wedge for static conditions has an angle of approximately 60° with the horizontal plane. AASHTO requires at least the greater of 5 feet or H/5 (50 / 5 = 10 ft) between the bond zone and active wedge. Therefore the minimum unbonded length of the tiebacks can be estimated as:

$L_{ub1} = 42 / (\tan 60° \cos 15° + \sin 15°) + 10 \text{ ft} = 31.8 \text{ ft}$
$L_{ub2} = 27 / (\tan 60° \cos 15° + \sin 15°) + 10 \text{ ft} = 24.0 \text{ ft}$
$L_{ub3} = 12 / (\tan 60° \cos 15° + \sin 15°) + 10 \text{ ft} = 16.2 \text{ ft}$

The bonded length is calculated based on a hole diameter of 8 inch and ultimate unit bond stress of 6 ksf for the silty sand native soil. Using a resistance factor of 1.0 (assuming the anchors will be load tested), the minimum bonded length for each anchor was calculated as:

L_{b1} = 279 / (1.0×6.0×3.1416×8/12) = 22.2 ft
L_{b2} = 281 / (1.0×6.0×3.1416×8/12) = 22.4 ft
L_{b3} = 296 / (1.0×6.0×3.1416×8/12) = 23.8 ft

A uniform bonded length of 25 feet was used for all three anchors. Therefore the minimum length of tiebacks was calculated as:

L_1 = 31.8 + 25 = 56.8 ft → use 60 feet anchor
L_2 = 24.0 + 25 = 49 ft → use 50 feet anchor
L_3 = 16.2 + 25 = 41.2 ft → use 50 feet anchor

Shear and Moment Diagram for Static Condition

Based on the above loading conditions, the soldier pile moment and shear distribution may be determined. For this example, the shear and moment diagrams in the soldier piles were calculated using the Caltrans CT-Flex (Shamsabadi, 2006) computer program, which uses a similar design procedure to that of the AASHTO Specifications. Using this program, an identical embedment depth of 10.85 feet was calculated under static loading.

Results from CT-Flex analyses for static loading are shown in Figures 5 through 8. These results can be used to design the soldier piles for static load conditions.

Global Stability for Static Load Conditions

The global stability of the anchored wall for the static condition was evaluated using the program SLIDE (Rocscience, 2005). The Spencer method was used to evaluate the minimum factor of safety for planar and general failure surfaces. The results are shown in Figures 9 and 10 for planar and general failure planes, respectively. It was assumed that the anchors can carry the estimated ultimate load (e.g., 279, 281, and 296 kips, for rows 1 to 3 respectively).

AASHTO recommendations for global stability were used. ASSHTO Section 11.6.2.3 states that:

> *"The overall stability of the retaining walls, retained slopes and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis. The overall stability of temporary cut slopes to facilitate construction shall also be evaluated. Special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft deposits.*
>
> *The evaluation of overall stability of earth slopes with or without a foundation unit should be investigated at Service 1 Load Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, ϕ, may be taken as:*

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- *Where the geotechnical parameters are well defined, and the slope does not support or contain a structural element: 0.75*

- *Where the geotechnical parameters are based on limited information, or the slope contains or supports a structural element: 0.65"*

Based on these recommendations a resistance factor of 0.65 was applied to the soil strength parameters in the global slope stability analyses. The strength parameters for backfill soil were calculated as:

$0.65\times\tan(\phi) = 0.65\times\tan(32) = 0.406165 \rightarrow$ use a friction angle of 22.1°.

$0.65\times c = 0.65\times 200 = 130$ psf

The strength parameters for foundation soil were calculated as:

$0.65\times\tan(\phi) = 0.65\times\tan(40) = 0.5454 \rightarrow$ use a friction angle of 28.6°.

$0.65\times c = 0.65 \times 0 = 0$ psf

Theoretically, the used of reduced soil properties is equivalent to conducting the slope stability using original material properties (i.e., not applying a resistance factor to c and ϕ) and confirming that the reciprocal of the resulting factor of safety (in this case FS = 1.538) is equal to or less than 0.65. However, the procedure used here allows for use of partial resistance factors on different resistive components (e.g., friction and cohesion) and agrees better with LRFD philosophy. The use of different resistance factors for different resistive components is consistent with the partial factor of safety approach that has been applied for over 30 years in some codes.[1]

As shown in Figures 9 and 10, the minimum factor of safety for the static condition was 0.938 and 1.015 for planar and general failure planes, respectively. The factor of safety for planar failure surfaces was smaller than 1.0; therefore, the anchor loads are increased. The anchor loads were increased by 15%, and a bonded length of 30 feet was used because of the low factors of safety. The revised properties of anchors are as follows:

Anchor 1: Test Load = 320 kip, Lock off load = 213 kip
Bonded length = 30 feet, Total Length = 65 feet

[1] From discussions with one of the developers of the AASHTO Specifications during review of the proposed Specifications, it is understood that current methods identified in the AASHTO Specifications are based on applying the resistance factor to the foundation capacity and not the soil properties. The implied benefit of the AASHTO approach is that uncertainties in both the soil properties and the capacity predictive equation are included in the resistance factor. Future revisions to the AASHTO Specifications may want to clarify the intended approach. No changes were made to the example problems to be consistent with this methodology, as the change would not alter the overall design approach.

Anchor 2: Test Load = 323 kip, Lock off load = 215 kip
Bonded length = 30 feet, Total Length = 55 feet

Anchor 3: Test Load = 340 kip, Lock off load = 227 kip
Bonded length = 30 feet, Total Length = 55 feet

Using these properties, the global slope stability analyses were performed again. The minimum factors of safety for the new analyses were 1.065 and 1.15, for planar and general failure planes as shown on Figures 11 and 12.

Results of Analyses – Seismic Design Check

The following paragraphs summarize the design check that was performed for seismic loading. The revised properties of the anchors described above were used as the basis for this check.

Seismic Earth Pressure and Anchor Design for Seismic Loading

Unlike the static condition, where the earth pressure behind anchor walls is affected by various construction-related parameters including preloading the anchors, the seismic earth pressure for earthquake loading (which may govern the design of the wall) is controlled by limit equilibrium of the retained soil mass. The distribution of earth pressure under seismic loading is not well-known for anchored walls. It can be argued that the pressure distribution could differ from that of cantilever walls as the anchors could increase the integrity of the backfill. In lieu of more complex numerical analyses, the seismic pressures were assumed to have the same distribution as the initial static pressures.

In order to estimate the increment in earth pressure under seismic loading and associated peak anchor load demands, a limit equilibrium analysis was performed. Such analysis can be performed by the trial wedge method, as discussed in Appendix B_X of the proposed Specifications. Alternatively, a slope stability program can be used to evaluate the lateral pressure under seismic loading.

For this example the trial wedge method was implemented using an Excel spreadsheet. This method is schematically for the general case in Figure 13. The total seismic earth pressure force on the wall (P_{AE}) was calculated from the following equation:

$$P_{AE} = \frac{W\left[(1-k_v)\cdot\tan(\phi-\alpha)-k_h\right]-c\cdot L_c\left[\tan(\phi-\alpha)\cdot\sin\alpha+\cos\alpha\right]-c_a\cdot L_a\left[\tan(\alpha-\phi)\cdot\cos\omega-\sin\omega\right]}{\left[1+\tan(\delta+\omega)\cdot\tan(\phi+\alpha)\right]\cdot\cos(\delta+\omega)}$$

where:

W = total weight of the active wedge, including surcharge

k_h = horizontal earthquake acceleration coefficient

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k_v = vertical earthquake acceleration coefficient
ϕ = soil internal friction angle
c = soil cohesion
c_a = soil / wall adhesion
δ = soil / wall friction angle
α = angle of failure plane measured from horizontal plane
ω = angle of wall face measured from vertical plane
L_a = length of wall face below tension crack
L_c = length of the failure plane

P_{AE} was calculated for different values of angle α. The maximum P_{AE} corresponds to the total seismic active earth pressure force on the wall. The vertical acceleration coefficient (k_v) was assumed to be 0, for the reasons discussed in the proposed Specifications.

Using this method, a total horizontal earth pressure force P_{AE} equal to 112.4 kips was obtained using k_{av} = 0.35 for active wedges exiting at the excavation level. Resistance from the soldier piles was neglected. The computer program SLIDE was utilized for the slope stability method. Only planar failure surfaces were examined. Spencer's slope stability analysis method was used to calculate the factor of safety. A detailed discussion of the assumptions in Spencer's method can be found in Abramson et al. (2001). The results of the evaluation determined that a horizontal load equal to 135 kips was needed to stabilize the wall under seismic loading.

A modified form of AASHTO's equation for the static trapezoidal pressure diagram was used for the seismic case:

$$p_{ae} = \frac{112.4}{50 - \frac{1}{3}\times 8 - \frac{1}{3}\times 12} = 2.60 \cdot ksf$$

The resulting earth pressure diagram under seismic loading is shown in Figure 14. For static loading, the apparent earth pressure was 1.62 ksf, indicating that the seismic load resulted in a 60% increase in the pressure.

Using the hinge method, the corresponding horizontal anchor loads were calculated as:

$T_{h1eq} = 8\times 36.04 = 288$ kip ➔ $T_{1eq} = 288 / \cos 15° = 298$ kip
$T_{h2eq} = 8\times 36.33 = 291$ kip ➔ $T_{2eq} = 391 / \cos 15° = 301$ kip
$T_{h3eq} = 8\times 39.55 = 316$ kip ➔ $T_{3eq} = 316 / \cos 15° = 328$ kip

The minimum embedment depth for seismic conditions was computed as 12.2 feet. Hence, the 12-foot embedment depth that was used for the static case was adequate for seismic case.

A load factor of 1.0 was used for the seismic loading. For this load factor the anchor loads calculated under seismic condition are smaller than the test loads used for static conditions. Therefore, the bonded length of 30 feet that was calculated based on the test loads will be adequate under seismic loading. If seismic loads are larger than the test loads, the bonded length should be revised at this stage of design.

Anchor Length

Figure 15 compares the active wedge calculated for seismic loading conditions with the active wedge for static loading. The critical active wedge for seismic conditions has an angle of approximately 42° with the horizontal plane, versus 60° for static conditions. AASHTO requires at least the greater of 5 feet or H/5 (50 / 5 = 10 ft) between the bond zone and active wedge for static design. This requirement was applied for static loading; however, for seismic loading this additional unbonded length seems to be excessive, because of the temporary nature of the load. Therefore the minimum unbonded length of the tiebacks for seismic loading conditions was estimated as:

$L_{ub1} = 42 / (\tan 42° \cos 15° + \sin 15°) = 37.2$ ft
$L_{ub2} = 27 / (\tan 42° \cos 15° + \sin 15°) = 23.9$ ft
$L_{ub3} = 12 / (\tan 42° \cos 15° + \sin 15°) = 10.6$ ft

Based on these values the unbonded length for anchor 1 should be increased from the current value of 35 feet. An unbonded length of 40 feet was specified for this anchor. The unbonded length of 25 feet for rows 2 and 3 was adequate for seismic loading conditions.

Anchor Elongation

The elongation of anchors due to seismic loading was used to estimate the deformation of the wall during the design earthquake. Assuming only the unbonded length of the anchors will stretch, the elongation of the anchor was calculated as:

$$\Delta L = \frac{\Delta T_{eq} \cdot L_u}{E_s \cdot A}$$

The area of anchors is calculated from factored loads using a resistance factor of 0.9 for mild steel and 0.8 for high strength steel. The minimum area, $A_{s,min}$, for this example was calculated based on grade 270 ASTM A416 strands with $F_{pu} = 270$ ksi:

Row 1: $A_{s,min1} = 320 / (0.8 \times 270) = 1.48$ in^2
Row 2: $A_{s,min2} = 323 / (0.8 \times 270) = 1.50$ in^2
Row 3: $A_{s,min3} = 340 / (0.8 \times 270) = 1.57$ in^2

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The elongation for each level of anchors (these are approximate because the actual area is determined by availability of the strands) was computed as shown below:

Row 1: $\Delta L_1 = (298 – 213)\times(40\times12) / (29{,}000\times1.48) = 0.95$ inch
Row 2: $\Delta L_2 = (301 – 215)\times(25\times12) / (29{,}000\times1.50) = 0.59$ inch
Row 3: $\Delta L_3 = (328 – 227)\times(25\times12) / (29{,}000\times1.57) = 0.67$ inch

These deformations are judged to be sufficient to mobilize the active pressure condition assumed in the seismic load calculations.

Shear and Moment Diagram for Seismic Condition

The shear and moment diagrams under seismic loading were calculated using the Caltrans CT-Flex computer program (Shamsabadi, 2006) in a similar manner to the static loading case. Using this program, an identical embedment depth of 12.2 feet was calculated under seismic loading.

Results from the CT-Flex analyses for seismic condition are shown in Figures 16 through 19. These results can be used to check the soldier pile static design for seismic loading conditions. Note that the deflection diagram shown in Figure 19 does not include the anchor deflections calculated above, which should be superimposed on the soldier pile deflections shown to obtain the actual deflection profile. Improvements in deflection and bending and shear distributions can be obtained by using p-y soil and anchor springs in design, as illustrated in following design example.

Global Stability Under Seismic Loading

The global stability of the anchored wall for static and seismic conditions was evaluated using the program SLIDE. The Spencer method was used to evaluate the minimum factors of safety for planar and circular failure surfaces. The results are shown in Figures 20 and 21, for static and seismic conditions, respectively.

It was assumed that the anchors carry the test load (e.g., 320, 323 and 340 kips, for the rows 1 to 3 respectively). A strength factor of 1.0 was used for these seismic analyses; therefore, no reduction in soil strength parameters was needed. A seismic coefficient of $k_{av} = 0.35$ was applied in seismic global stability evaluations.

As shown in Figures 20 and 21, the minimum factors of safety for seismic loading conditions were 1.03 for planar failure surfaces and 1.24 for general failure surfaces.

Concluding Comments

These results show that anchored walls designed using static pressure distributions should perform very well during ground motions that can be encountered in seismically active areas. However, the design should be checked for the seismic loading condition, as shown in this

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design example. A review of slope stability analyses for seismic cases in general indicates that the failure wedge under seismic loading is larger than the static condition. The length of the anchors, therefore, may need to be increased to increase the stability for seismic loading condition. The global stability of the wall also needs to be checked for static and seismic conditions, as shown in this example.

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Abramson, L.W., Lee, T.S., Sharma, S. and Boyce, G.M. (2001). *Slope Stability and Stabilization Methods*, 2nd Edition, John Wiley & Sons.

RocScience (2007). “SLIDE: Stability Analysis for Soil and Rock Slopes.” www.rocscience.com.

Shamsabadi, A. (2006). *CT-FLEX – Computer Manual*, Office of Earthquake Engineering, California Department of Transportation.

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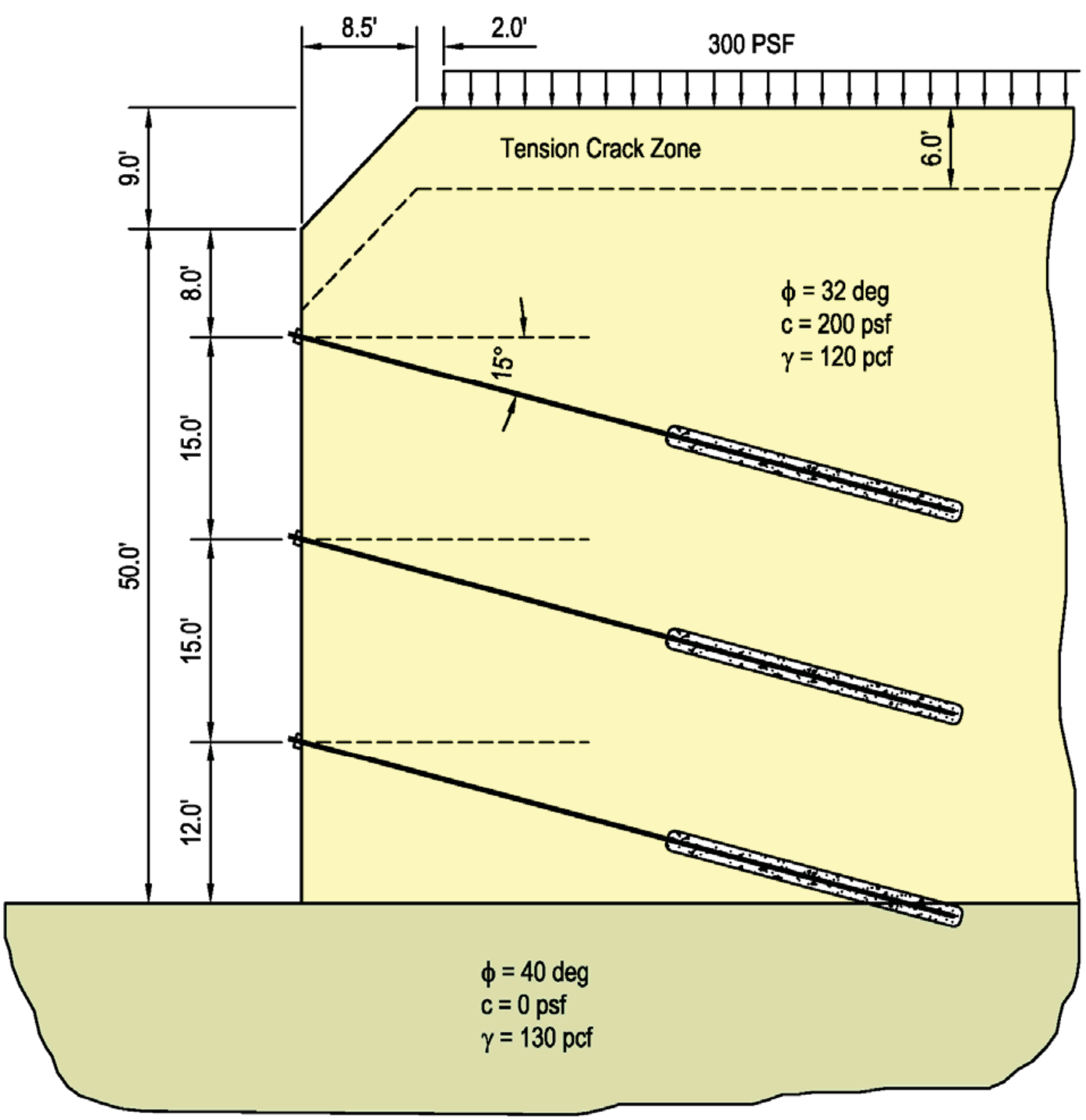

Figure 1. Wall Geometry

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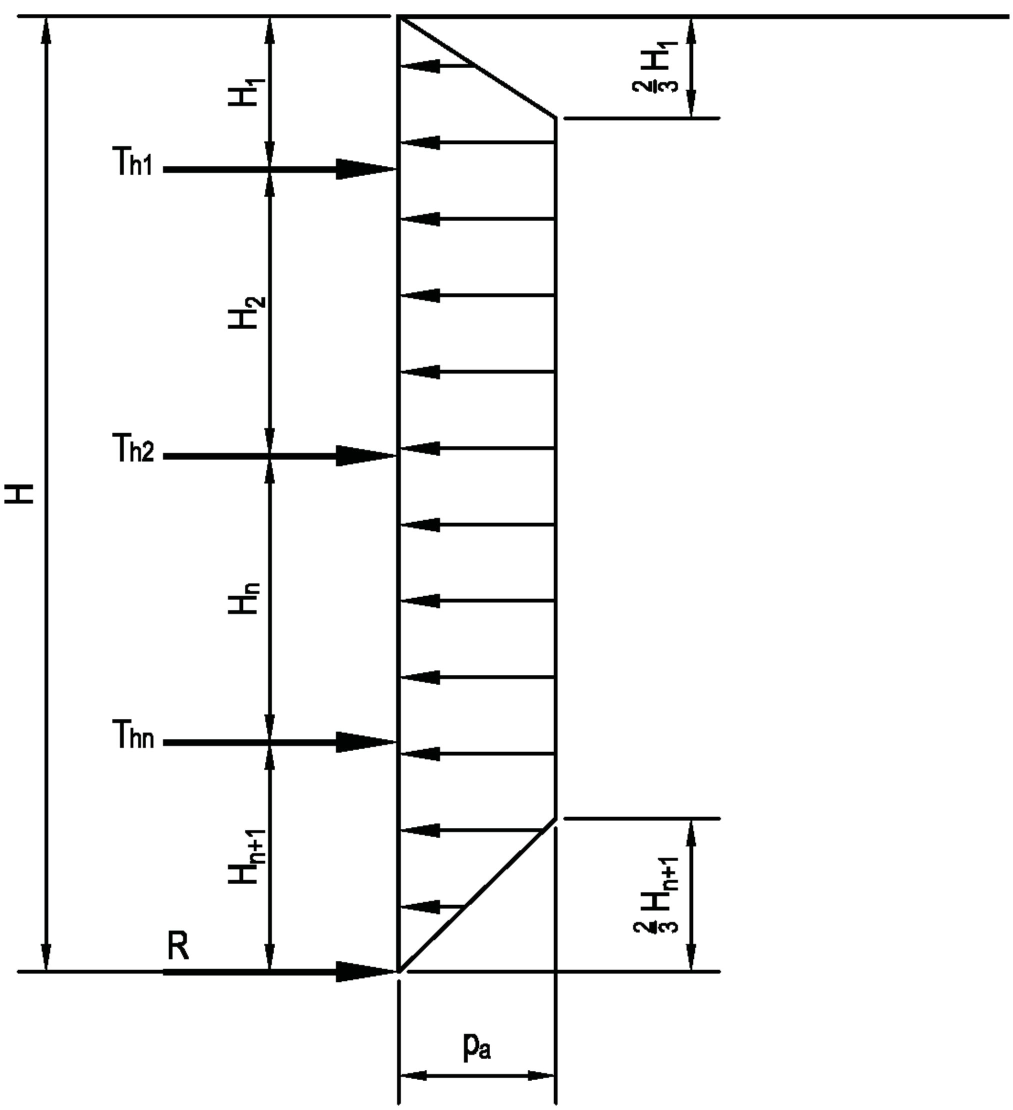

Figure 2. Apparent Earth Pressure Distribution for Multi-Anchored Walls Constructed from Top Down in Cohesionless Soil under Static Condition (after AASHTO, 2007)

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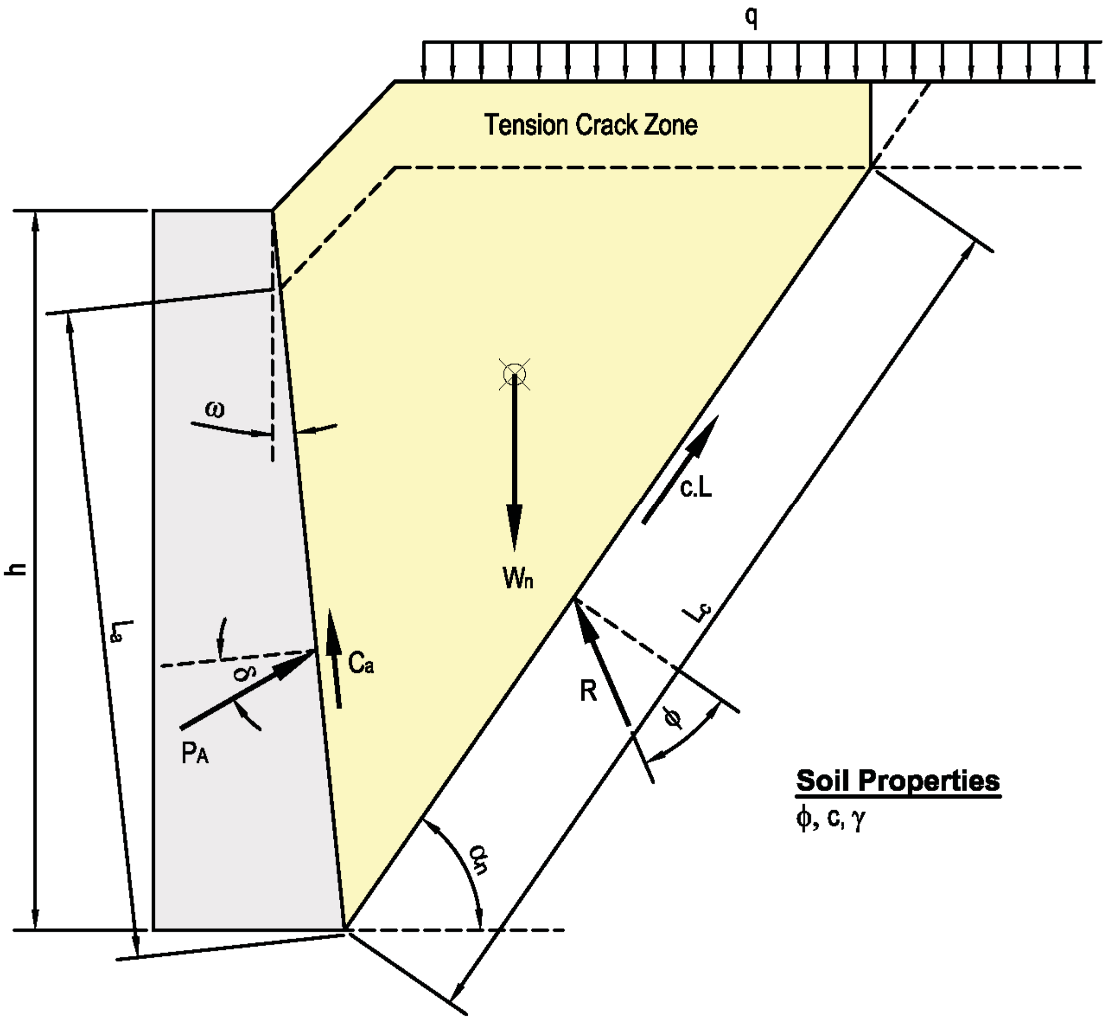

Figure 3. Trial Wedge Method under Static Loading for Retaining Walls

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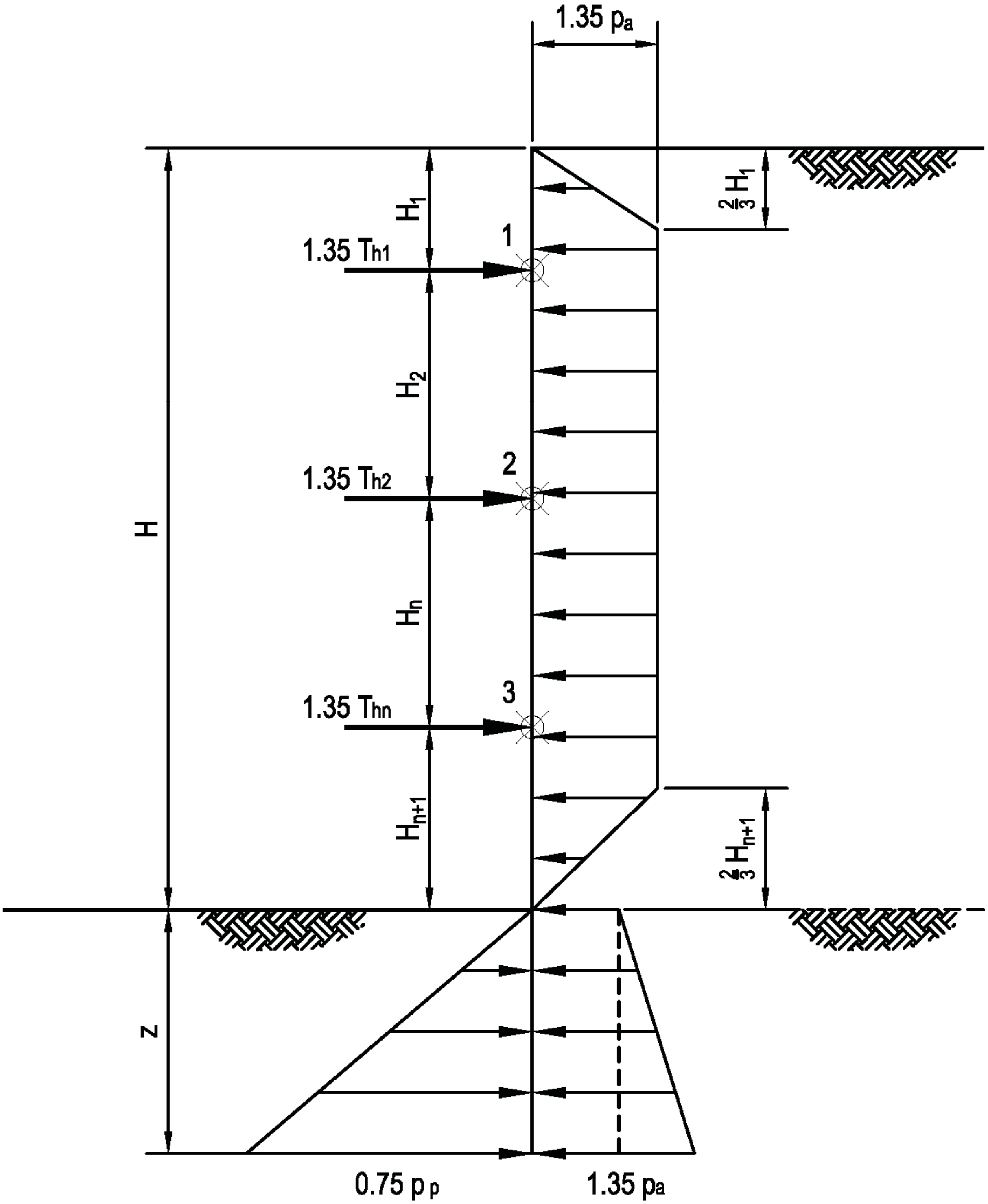

Figure 4. Determining Minimum Embedment Depth for Static Condition (Strength I Load Case)

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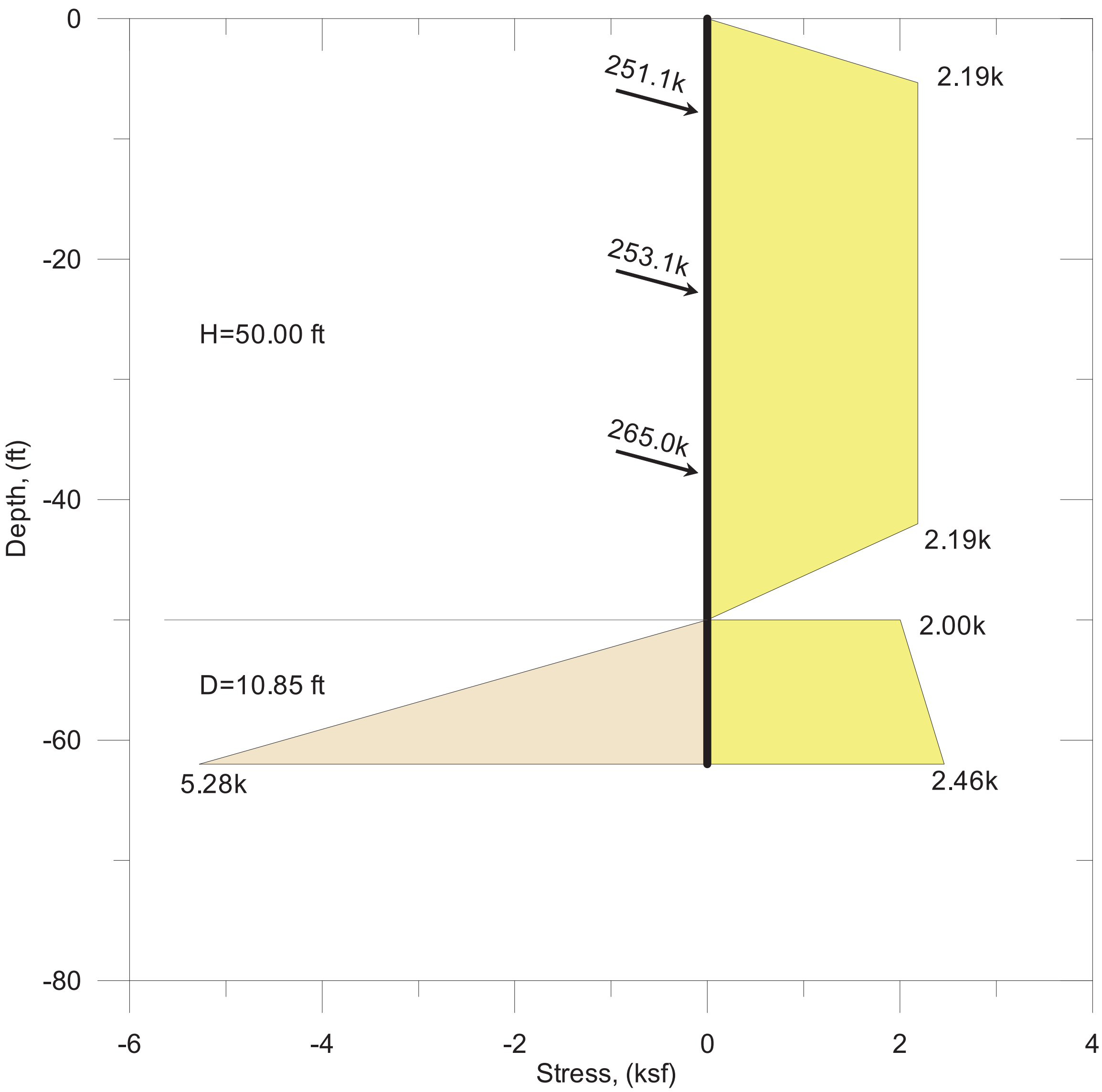

Figure 5. Soldier Pile Loading Diagram for Static Condition (CT-Flex 2006)

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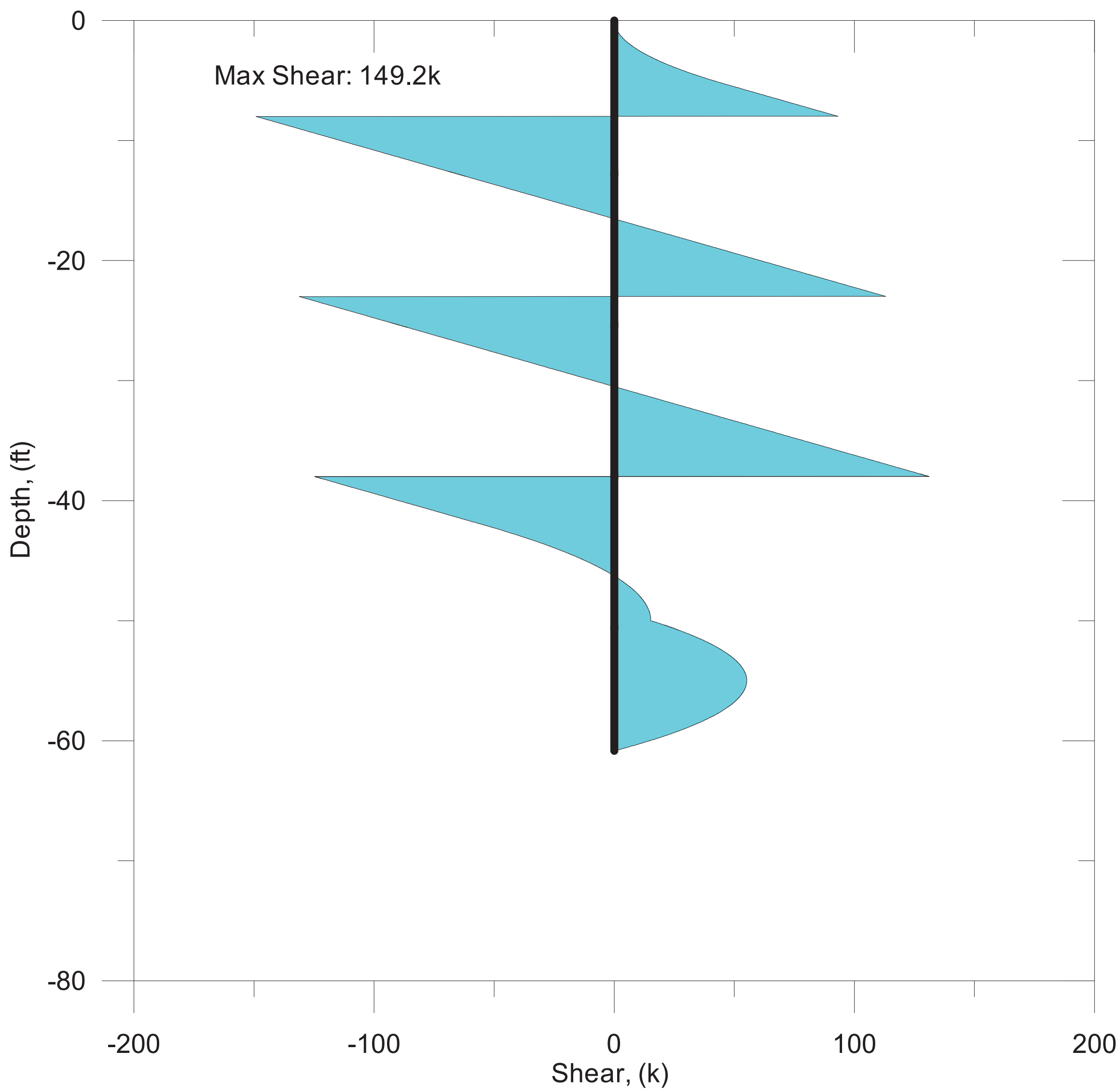

Figure 6. Soldier Pile Shear Force Diagram for Static Condition (CT-Flex 2006)

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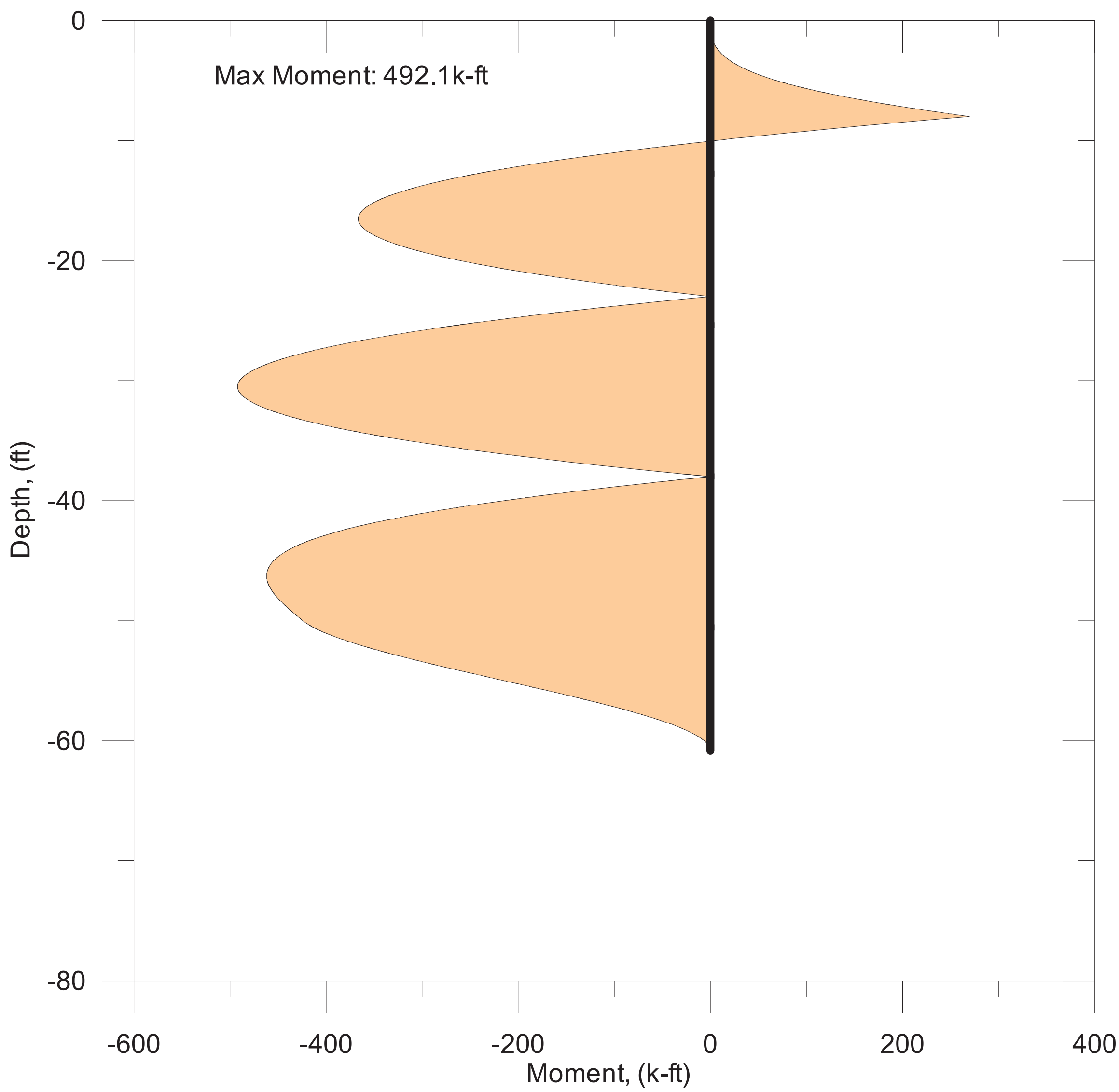

Figure 7. Soldier Pile Bending Moment Diagram for Static Condition (CT-Flex 2006)

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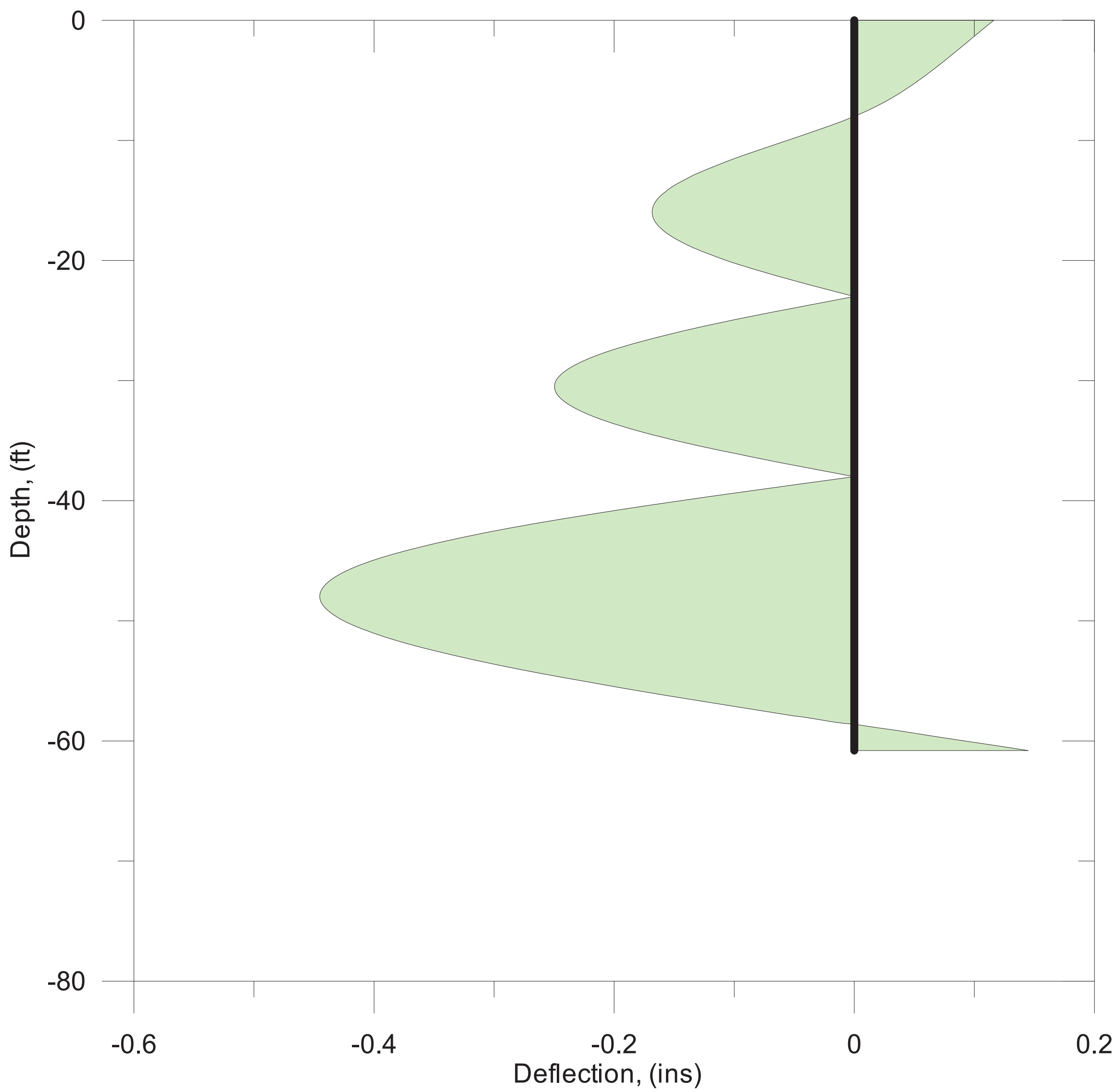

Figure 8. Soldier Pile Deflection Diagram for Static Condition (CT-Flex 2006)

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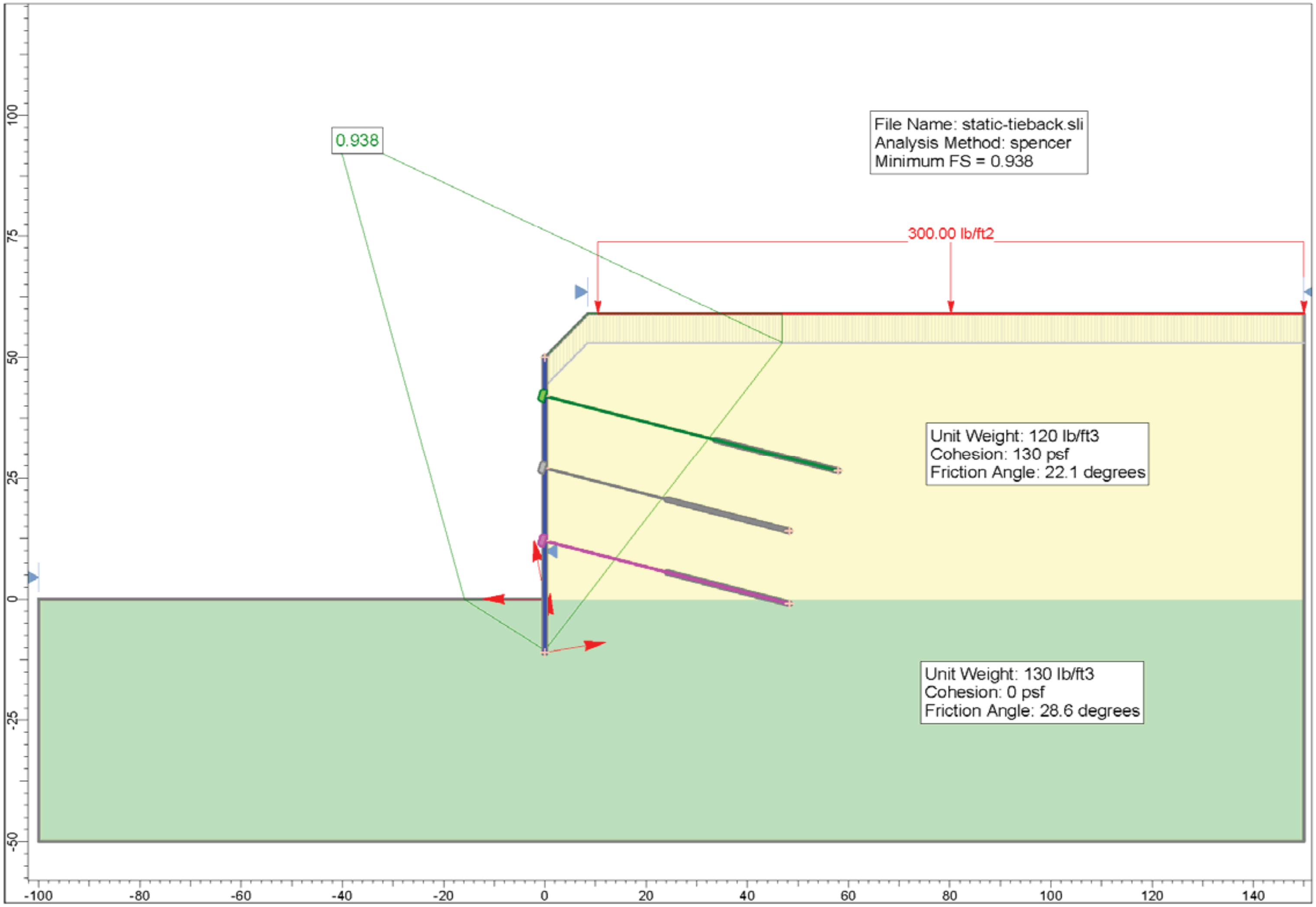

Figure 9. Global Stability for Static Condition, Planar Failure Surfaces

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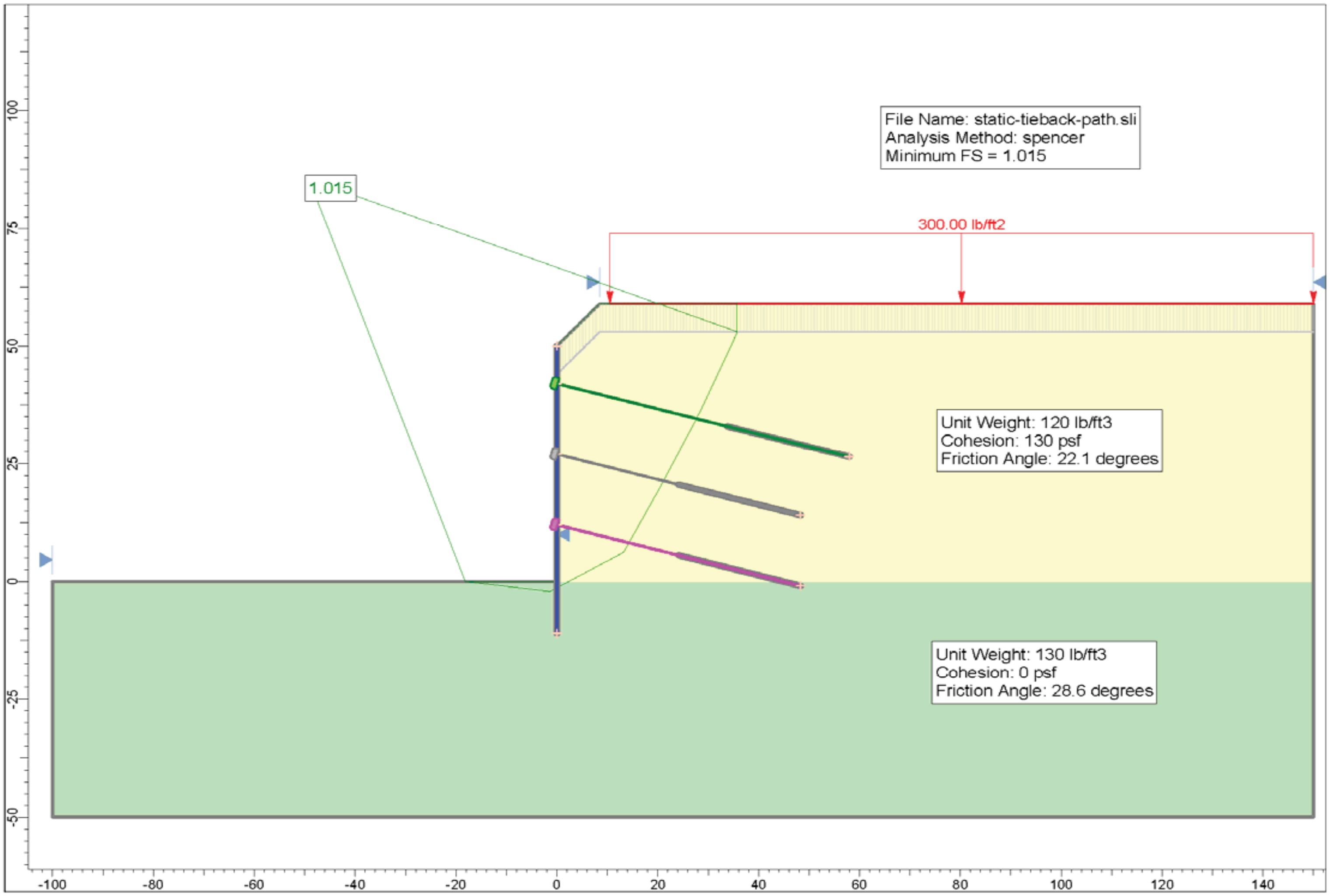

Figure 10. Global Stability for Static Condition, General Failure Surfaces

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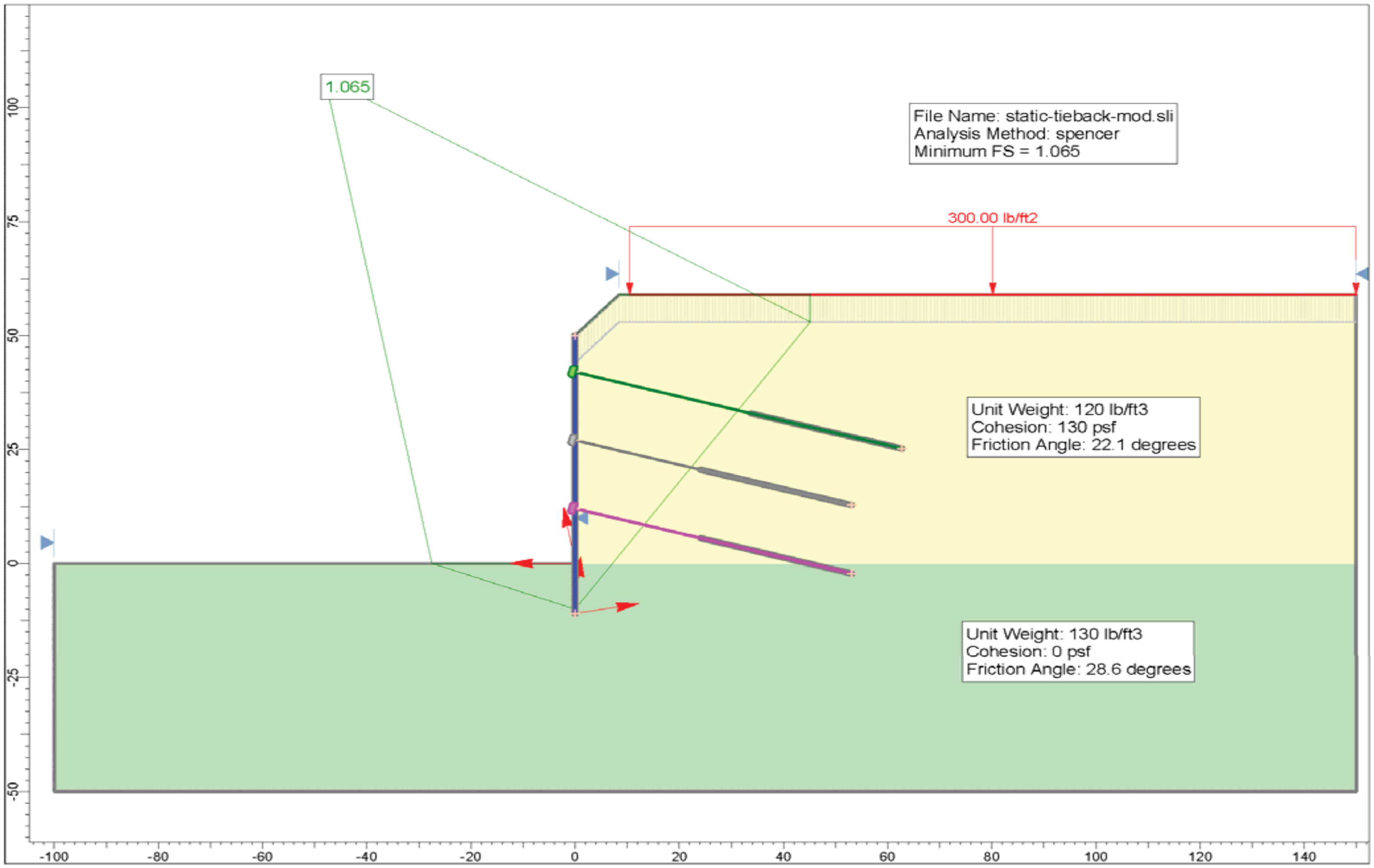

Figure 11. Global Stability for Static Condition, Planar Failure Surfaces, Revised Anchor Loads

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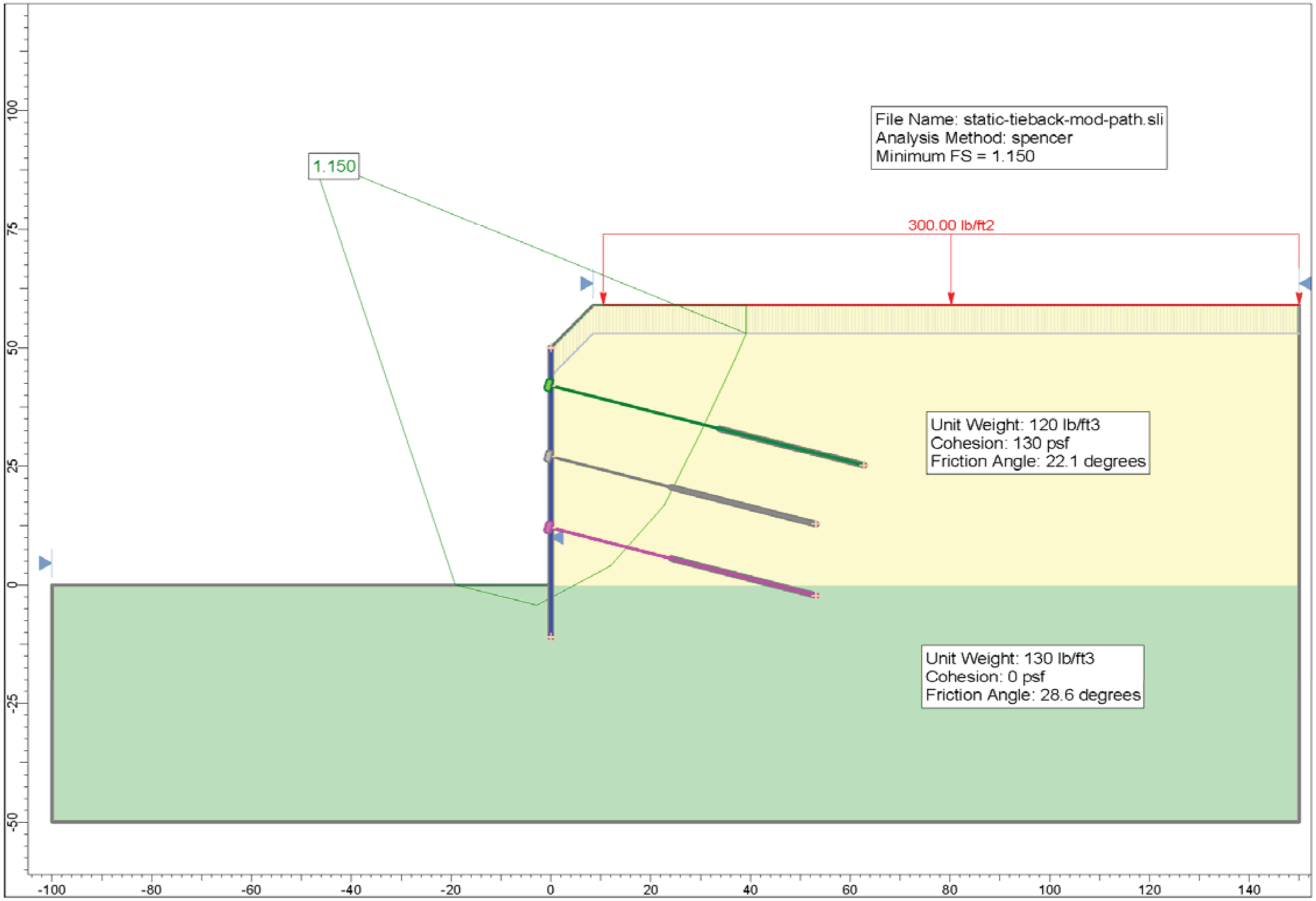

Figure 12. Global Stability for Static Condition, General Failure Surfaces, Revised Anchor Loads

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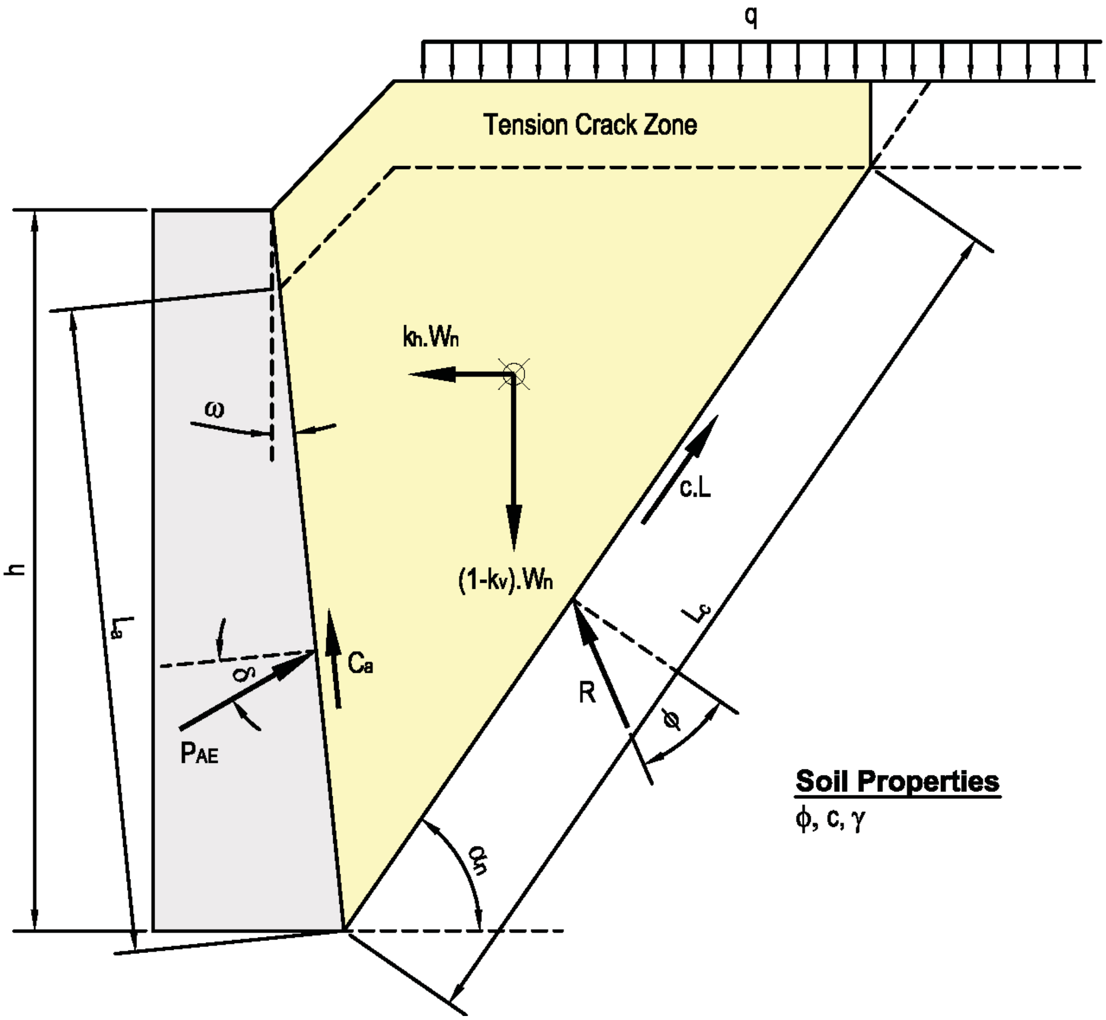

Figure 13. Trial Wedge Method under Seismic Loading for Retaining Walls

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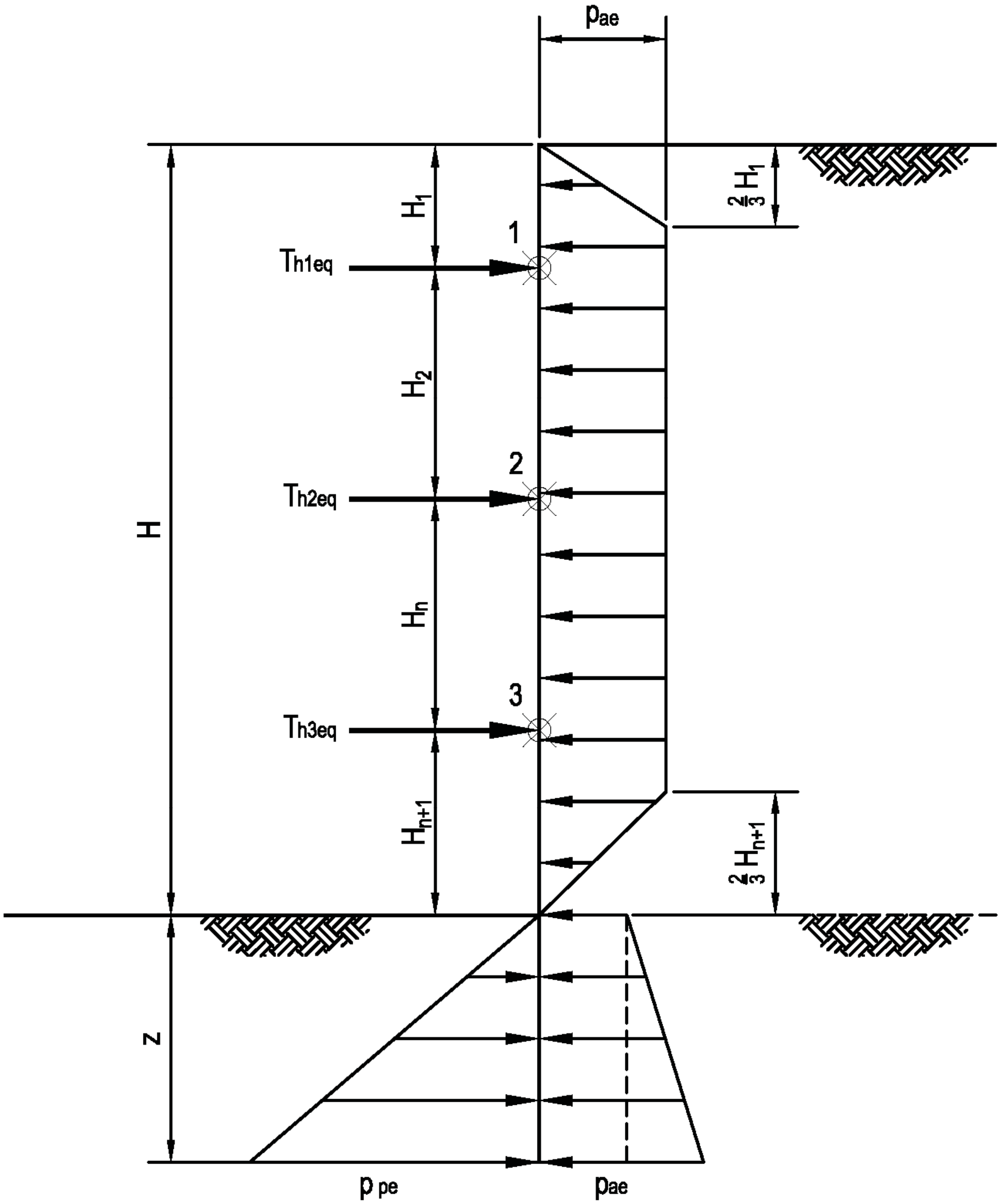

Figure 14. Determining Minimum Embedment Depth for Seismic Condition (Extreme Event I Load Case)

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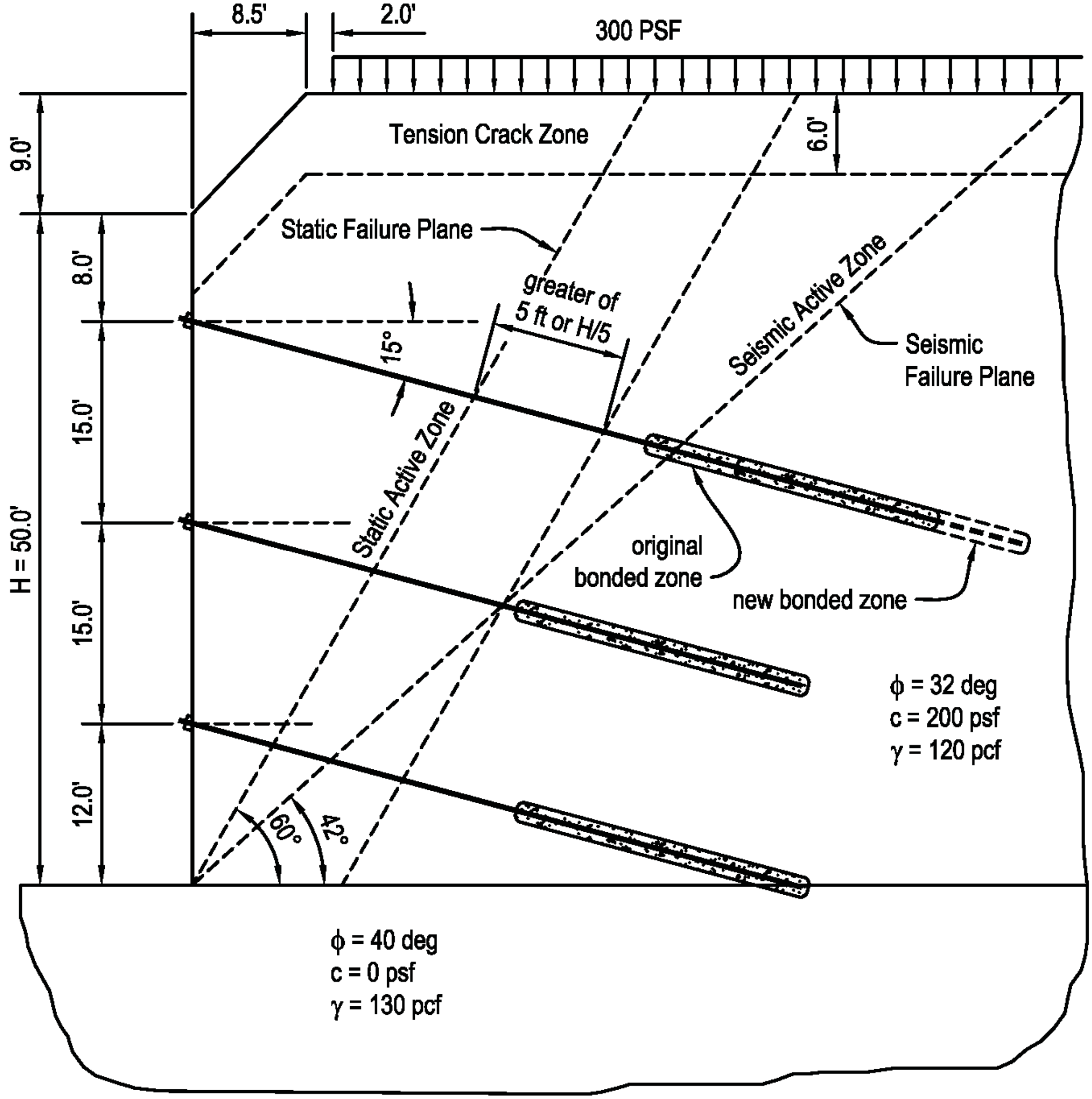

Figure 15. Comparison Between Active Zone for Static Conditions and Active Zone under Seismic Loading Conditions

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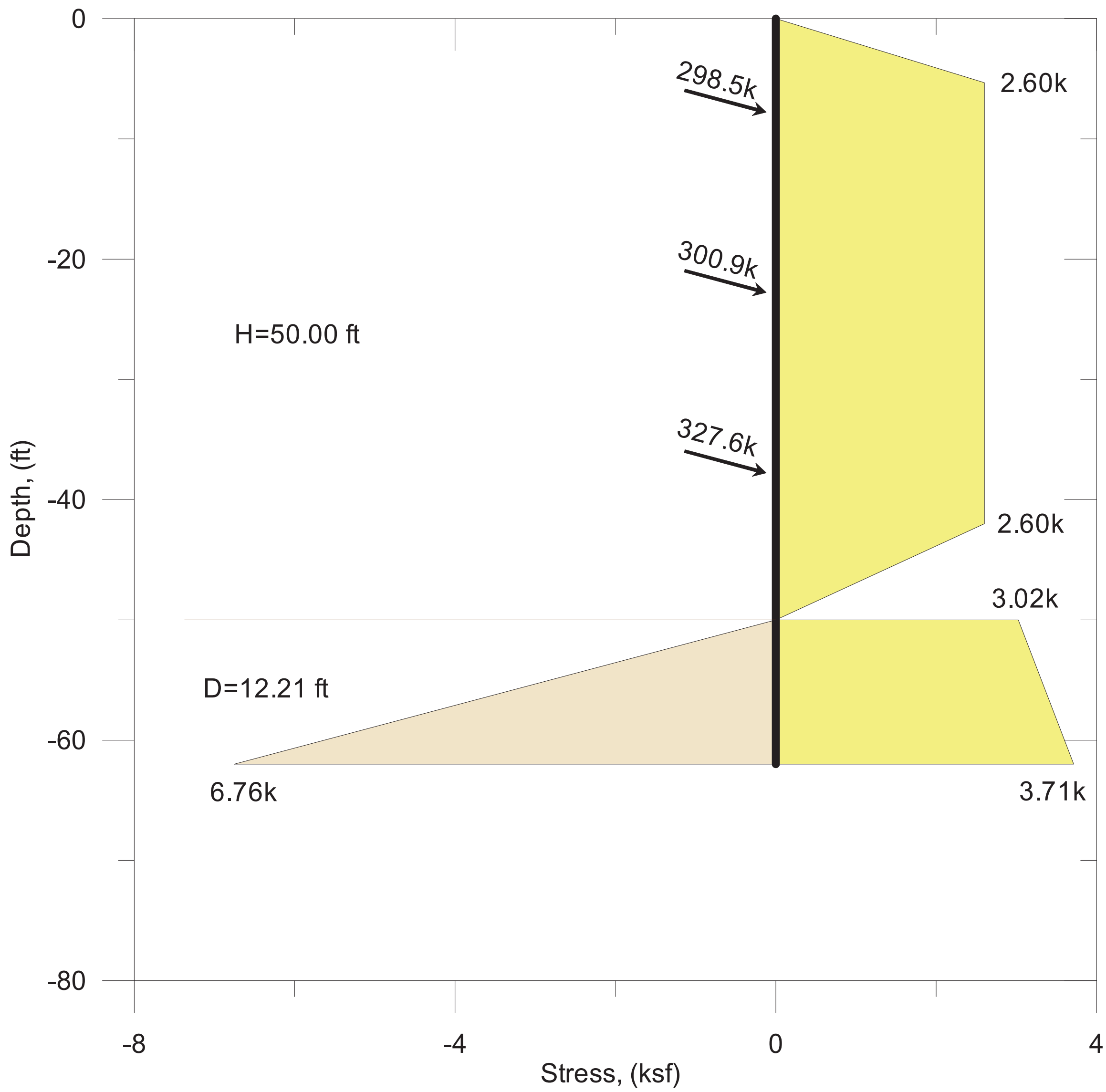

Figure 16. Soldier Pile Loading Diagram for Seismic Load Conditions (CT-Flex 2006)

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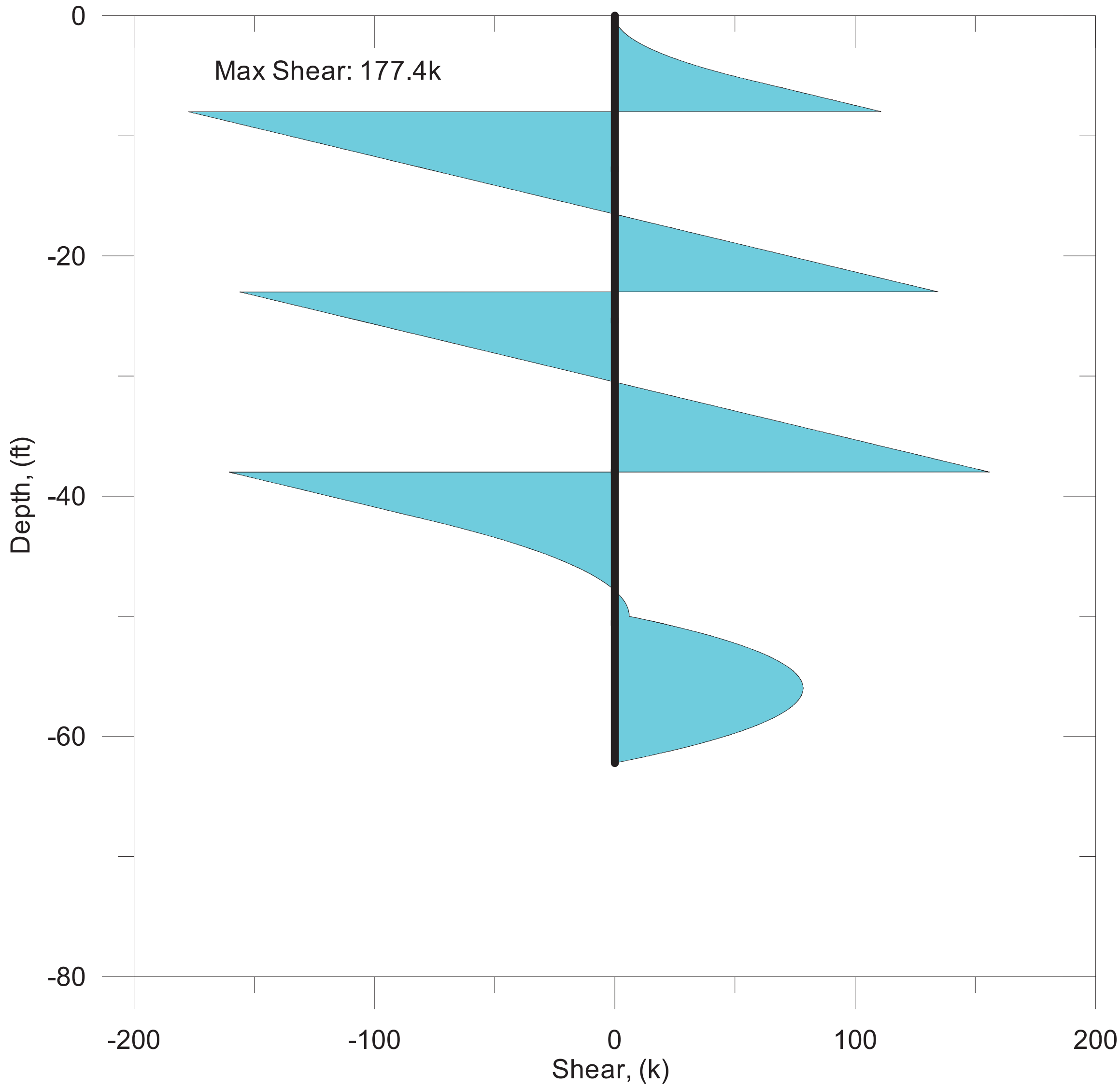

Figure 17. Soldier Pile Shear Force Diagram for Seismic Load Conditions (CT-Flex 2006)

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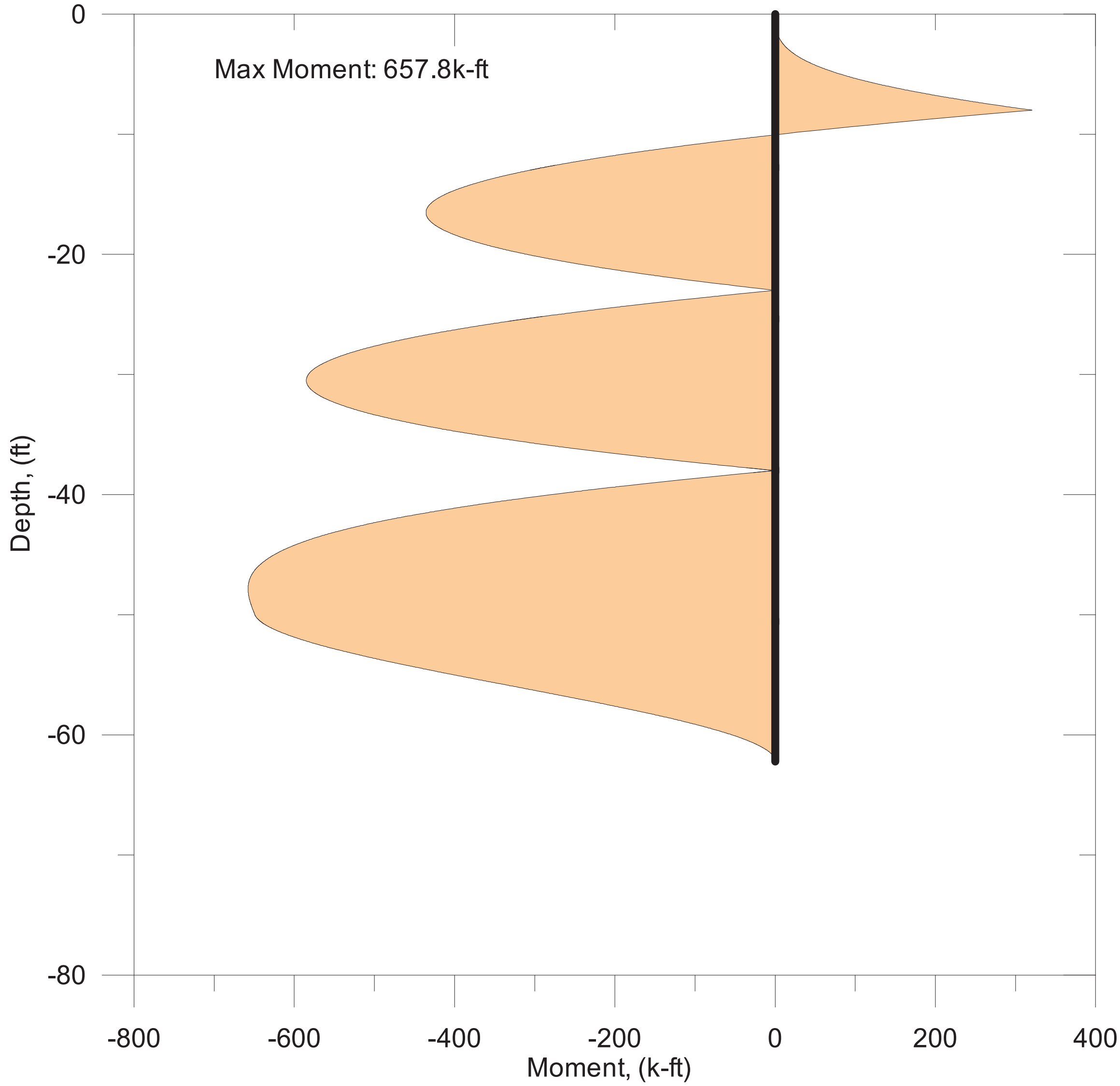

Figure 18. Soldier Pile Bending Moment Diagram for Seismic Load Conditions (CT-Flex 2006)

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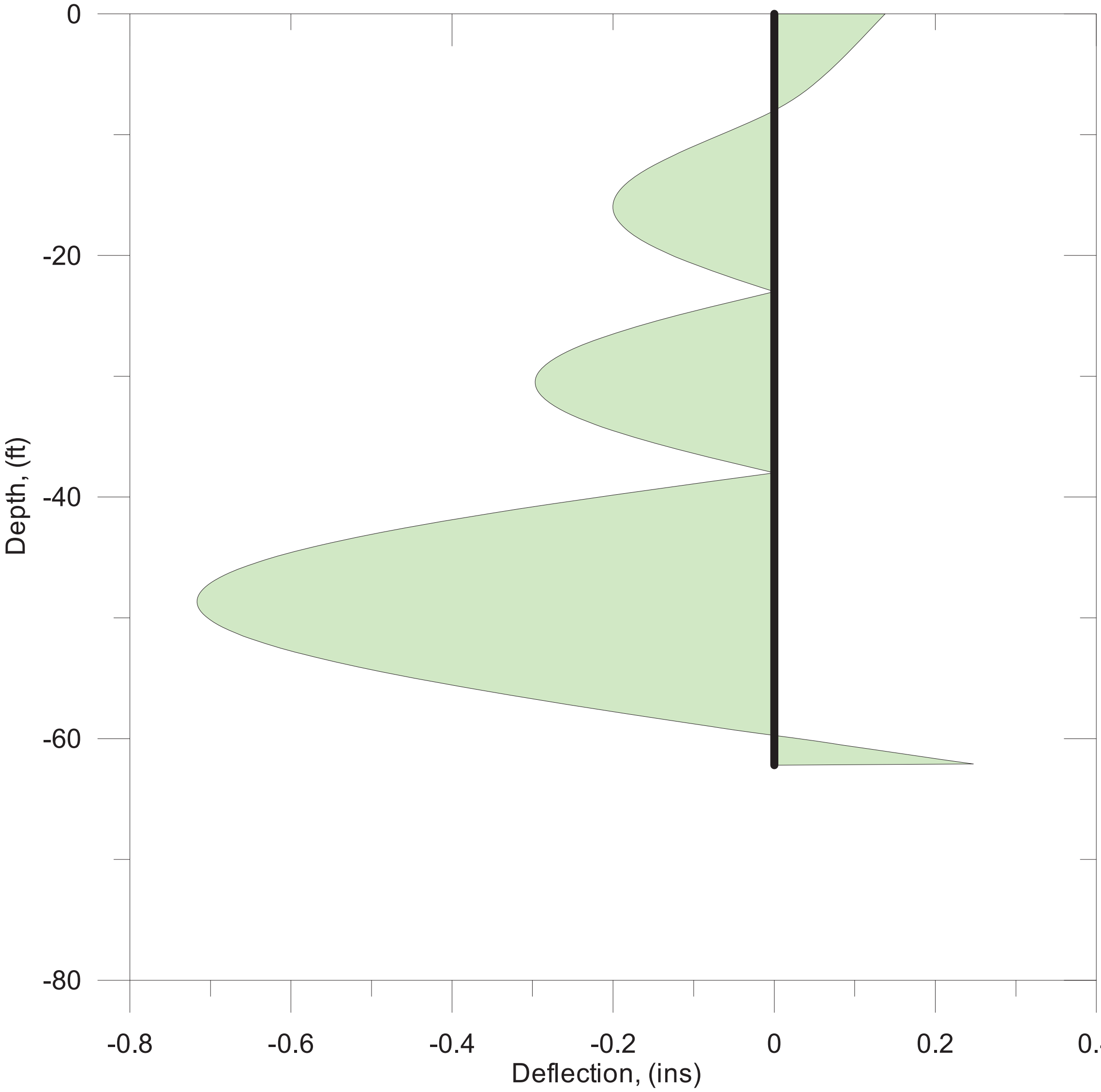

Figure 19. Soldier Pile Deflection Diagram for Seismic Load Conditions (CT-Flex 2006)

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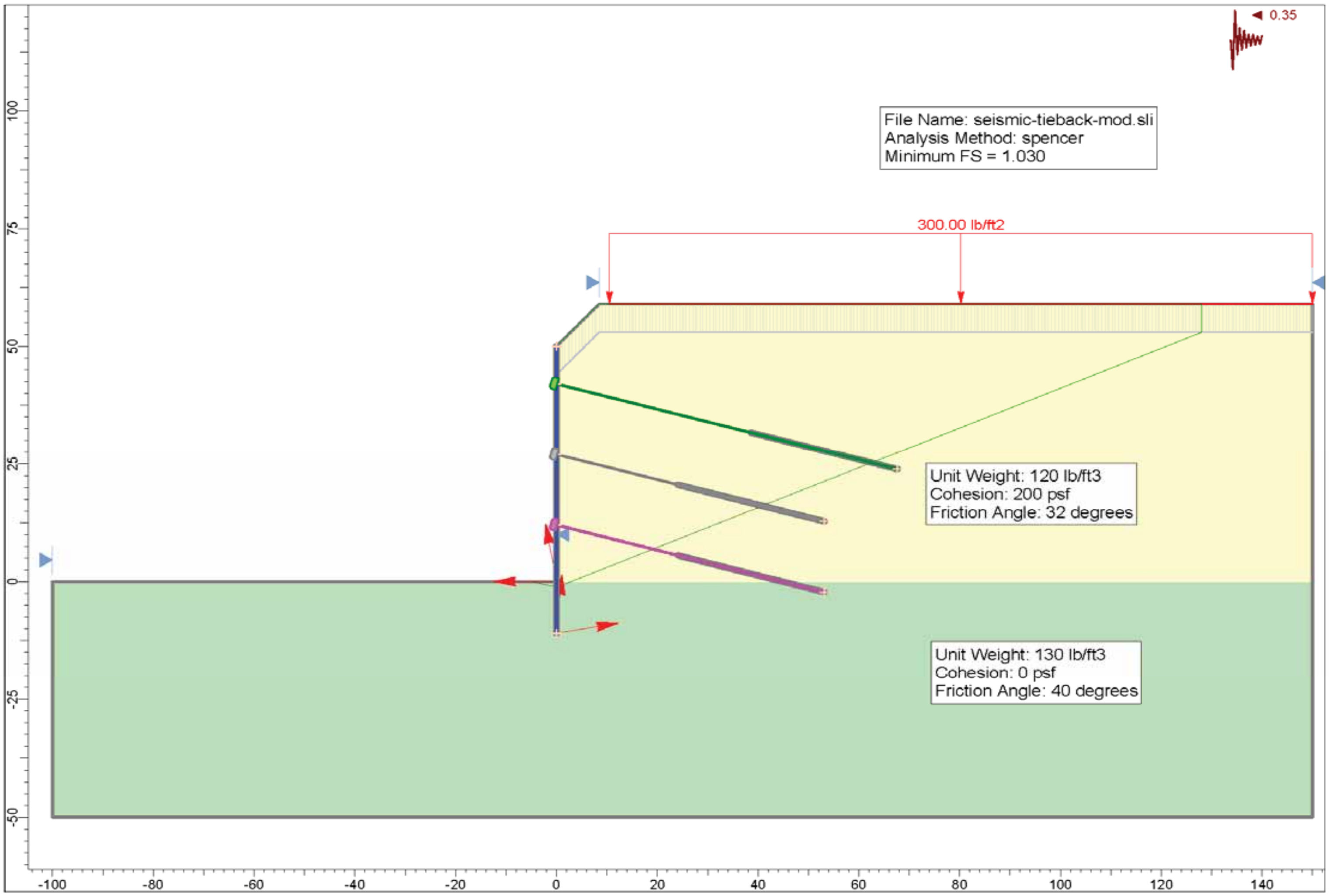

Figure 20. Global Stability for Seismic Condition, Planar Failure Surfaces, Revised Anchor Loads

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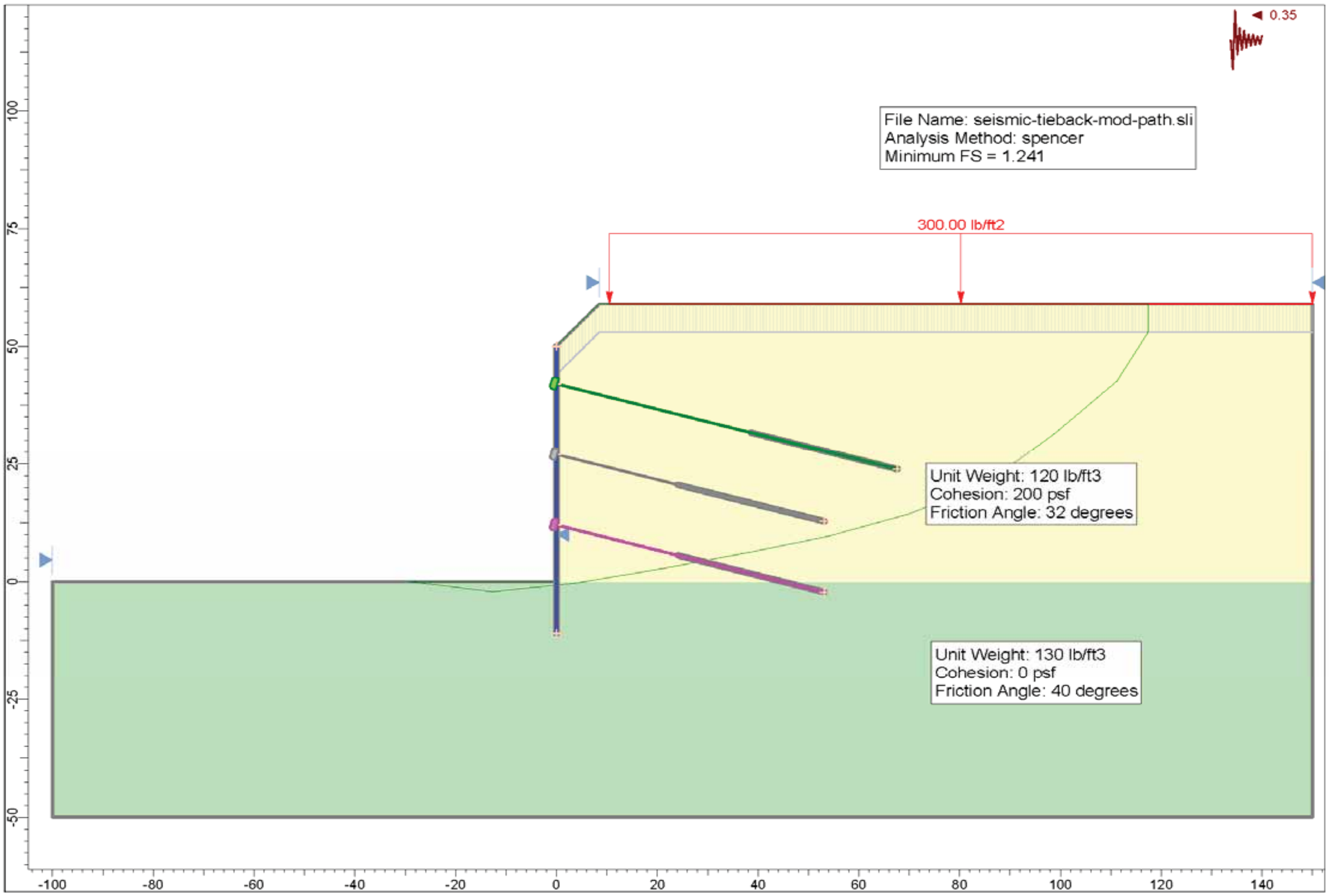

Figure 21. Global Stability for Seismic Condition, General Failure Surfaces, Revised Anchor Loads

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Example Anchored Wall Problem – P-Y Method

Introduction

This problem illustrates design of an anchored wall using the p-y method. This method differs from the limit equilibrium approach given in the current AASHTO *LRFD Bridge Design Specifications*. The p-y approach is a displacement-based method, and therefore, provides a better model of soil and structural properties during seismic loading. Various computer software are also now available, making this a preferred method of designing for both gravity and seismic loads.

The anchored wall is commonly used for wall heights greater than 15 to 20 feet. At these heights the active earth pressures on cantilever walls become very large, leading to significant amounts of wall deflection and often very large structural sections to meet load demands. For these taller, ground anchors (or tiebacks) are generally required to reduce the wall deflections, thereby reducing the size of structural members and the cost of the wall. Also, even for shorter walls, when they are adjacent to structures which would be adversely impacted for the ground movements, such as bridge structures, tiebacks are often used to protect the existing structure.

The wall for this problem is 25 feet in height and involves a single unit of homogeneous sandy soil behind the wall as well as the foundation soils below the excavation grade. It is recognized that this is an idealized condition. Most anchored alls for transportation are used for cuts into existing hillsides. The native soils in the hillside are rarely clean and cohesionless, More commonly the soil would include small to significant amounts of cohesive soils. As discussed in the proposed Specifications, the cohesive content changes the seismic active and passive earth pressures significantly – reducing active pressures and increasing passive resistance. For gravity loads it is common practice to neglect these contributions; however, during seismic loadings failure to consider this contribution to dynamic strength can lead to an overly conservative design.

The unit weight of the sand is assumed to be 120 pcf; the friction angle is 32 degrees. Similar to the p-y examples for nongravity cantilever walls, the loading condition is based on the active earth pressure theory. Using the Rankine earth pressure theory, the static active pressure coefficient (K_a) was determined to be 0.307. The soil support below the excavation level was estimated using the log-spiral failure surface, with an interface wall-soil friction angle equal to half of the internal friction angle of the sand (i.e., ϕ = 16 degrees) resulting in a passive earth pressure coefficient, K_p equal to 5.35.

Seismic loads were estimated using the Mononobe Okabe equation. As discussed in the proposed Specifications, this equation provides a reasonable method of estimating seismic active earth pressures as long as the soil is homogeneous and cohesionless. Charts in Appendix B can be used to estimate seismic earth pressures if there is a cohesive content. However, if the soils are layered as often occurs, the generalized limit equilibrium slope stability method will likely be the easiest

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method of estimating seismic earth pressures. In this case the total force necessary to achieve equilibrium (C/D ratio = 1.0) under pseudo static loading (see Article X.4) is determined. A very significant advantage of the slope stability approach is that the critical slope failure surface during seismic loading is also defined. Standard practice is to locate the anchor bond zone behind the anchor. Equations are also available for estimating the location of this failure surface if soils are cohesionless.

When determining the seismic load, two significant assumptions were made regarding the magnitude and distribution of the seismic load;

- The magnitude of the load was assumed to be 1.3 times the earth pressure estimated using the Mononobe-Okabe equation or from the generalized limit equilibrium slope stability method. This assumption was somewhat controversial. It was possible to argue that there is sufficient "stretch" in the anchors that the standard approach of increasing the static earth pressure by a factor of 1.3 is unnecessary during seismic loading. In this case the seismic active earth pressure is applied without modification. On the other hand if there is amplification of ground motion or if the anchors result in stiffening of the soil mass behind the wall, it is possible to argue that the 1.3 factor is applicable during seismic, as well as gravity loading. In the absence of specific information on the correct approach, a conservative approach was adopted.

- The second assumption involved the distribution of the seismic earth pressure. For semi-gravity and nongravity cantilever walls either a uniform or inverted triangular distribution is normally assumed for the seismic increment of earth pressures. In the case of anchored walls, the static earth pressure is assumed to be trapezoidal in shape. This shape is based on field measurements made during testing and gravity loading of walls. For the proposed Specifications and example problems, the same distribution is assumed for seismic loading.

The seismic coefficient used for determining the seismic earth pressure was determined by reducing the k_{max} by 50% to account for a combination of wall displacement, wave scattering effects, and soil cohesion. If a 50% reduction in k_{max} is used, several inches of permanent wall displacement could occur. The potential effect of these displacements on the soldier pile, as well as the anchor must be considered, when basing a design on use of the reduced PGA. Generally the elasticity of the strand of bar will allow some stretch of the anchor, but the permanent displacement may not be acceptable for the soldier piles. The p-y analyses described in this problem allows the effects of these movements to be explicitly evaluated.

Static Design Using Conventional and P-Y Methods

The wall was first designed for gravity loads following the methods given in the AASHTO *LRFD Bridge Design Specification*. The wall was then analyzed using a displacement-based approach. After completing the design for static loads using the two methods, earthquake loads were applied.

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As discussed in Section X.9 of the proposed Specifications, the tieback anchor increases the wall soil stiffness, which leads to an increase in the earth pressure. For gravity loads a scaling factor of 1.3 is generally applied to the active earth pressure for the earth pressure loading. The earth pressure shown on the right side of the wall above the excavation level corresponds to a 1.3 factor applied to the static active earth pressure values (based on the conventional equation $p_a = K_a \gamma z'$), which leads to an integrated static earth pressure load equal to 14,981 lb per foot of wall width acting over the 25 feet of wall height.

The trapezoidal earth pressure distribution shown in the figure was developed based on FHWA recommended procedures for tieback walls when a single tieback is used. The depth where the earth pressure increases to a constant plateau value of 898.73 psf (i.e., z = 5.33 feet) and the depth where it starts to decrease from this constant plateau value (i.e., z = 5.33 + 8.34 = 13.67 feet) was based on conventional rules from FHWA involving the 2/3 scaling factor applied to the anchor depth of 8 feet. As discussed earlier, the design load of 14,981 lb per foot of earth pressure load includes a 1.3 amplification factor applied to the conventional active pressure load value of 11,520 lb per foot determined by the Rankine active earth pressure coefficient K_a of 0.307.

In conjunction with the trapezoidal earth pressure load above the excavation level, an additional loading is needed for the portion of the anchored pile wall beneath the excavation level. This set of earth pressure loads increases linearly from 922 psf at the excavation grade to 1,160 psf at a 6.5-foot depth at the tip of the wall. The load was calculated using the static Rankine active earth pressure theory without the 1.3 amplification factor. It is an implicit assumption that the soil zone beneath the excavation grade would be sufficiently far away from the tieback anchors and hence, would not be affected by the anchor stiffness.

The Caltrans CT-FLEX program (Shamsabadi, 2006) was used to conduct the design calculations for the wall. This method is based on limit equilibrium methods as illustrated earlier for the static load case. A sheet pile embedment depth of 6.5 feet was required to meet global stability requirements (i.e., 1.5 times the static active pressure load based on moment equilibrium of the sheet pile wall).

In addition to the earth pressure loads, Figure 1 also shows the passive pressure soil resistance mechanism beneath the excavation level on the left side of the wall. For the p-y analysis, the soil resistance will be modeled by a set of p-y springs. The ultimate capacity of the p-y springs is selected to be compatible to the assumed passive earth capacity based on the appropriate earth pressure theory. In this case the ultimate earth pressure capacity was based on the passive earth pressure K_p coefficient of 5.35.

The right hand side of Figure 1 shows the sheet pile modeled used for the beam-column load deflection analysis. In addition to the conventional earth pressure load, and the Winkler springs to model the passive earth pressure reaction, an additional linear spring representing the tieback anchors was modeled shown at a depth of 8 feet below the top of the sheet pile wall.

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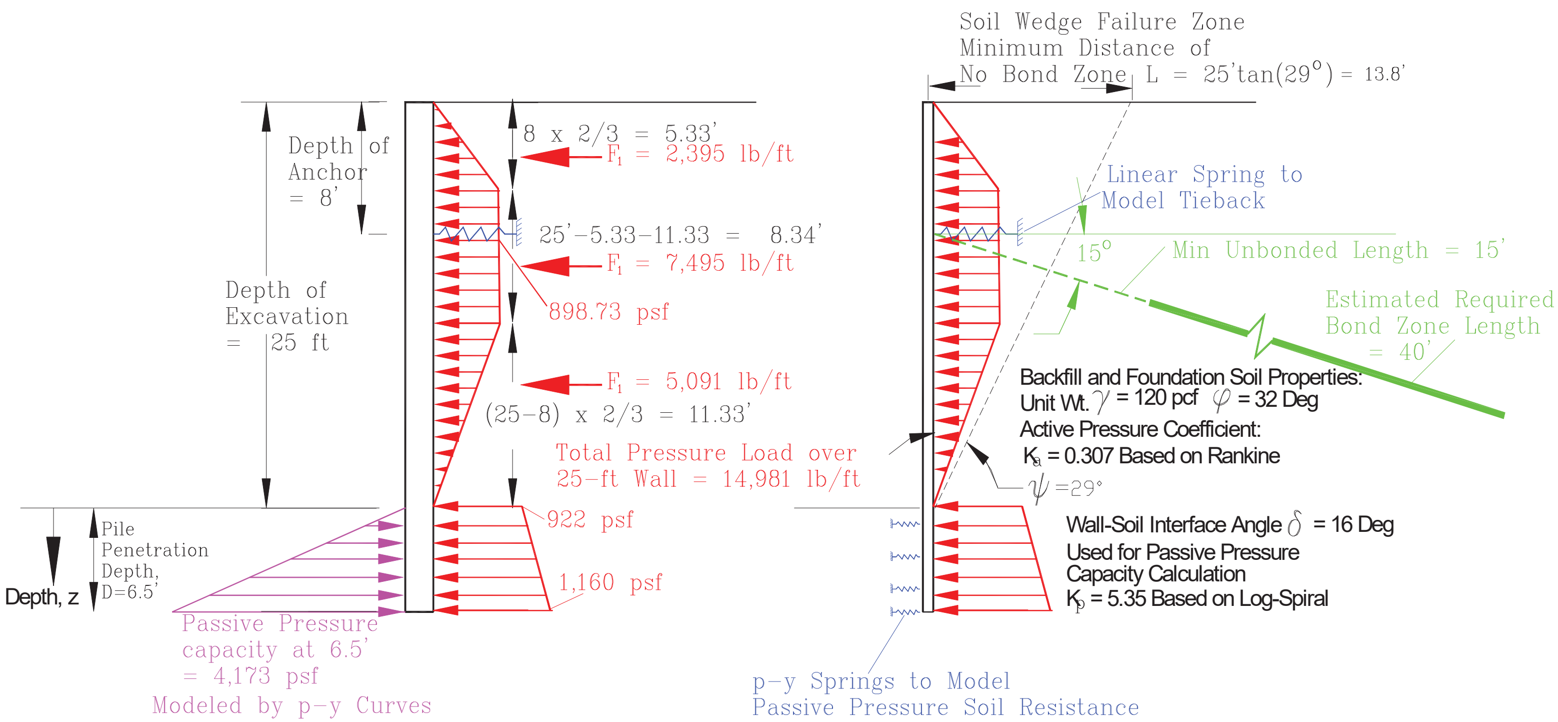

Figure 1. Tieback Wall Problem

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Results of Anchor Stiffness Analyses

From Figure 1, an earth pressure load of 14,981 lb per foot of wall width was estimated. Assuming a lateral spacing between anchors of 8 feet, the design load per anchor would be 14,981 x 8 = 120 kips per anchor. A design load of 120 kip per anchor is reasonable based on design loads for commonly constructed tieback anchors. Generally, anchors with capacity requirements higher than 200 kips should be avoided since these high-capacity anchors tend to be very expensive. The high cost results from not only the higher capacity strand and head assemblies, but also the higher capacity load test equipment required for proof testing each of the constructed tieback anchors.

Both strands and bars have been used for constructions of tiebacks. Therefore it may be necessary to allow for a significant variation in the eventual constructed anchors selected by the contractors in the design process. For this problem, the anchors are assumed to be constructed using steel strands with an ultimate tensile strength of 270 ksi. The cross sectional area of the strand is estimated below. This area is used as the basis for stiffness estimate of the tieback anchors.

Cross Sectional Area for the basic design anchor load:

$$A_{design} = 120 \text{ kip}/(0.6 \times 270 \text{ ksi}) = 0.74 \text{ in}^2.$$

In addition to the basic design load case, the design also needs to evaluate loads associated with proof testing. Typically, the design load is multiplied by a factor between 1.2 and 2.0. Theoretically, the choice of the appropriate factor should reflect potential loading conditions expected during the life of the anchor. Therefore, in regions of high seismicity, a higher factor is justified because of the higher earthquake load. From experience, a factor at about 1.5 is typically specified for the proof test condition. After proof testing to this higher load value, the anchors are relaxed to the theoretical lockoff design load value (equal to 120 kip). For this example, the proof test was assumed to be 1.5 times the design load (or anchor proof test load = 120 kip x 1.5 = 180 kip). For this non-sustained loading, condition, the cross sectional area can be estimated based on the equation: A_{proof} = 120 x 1.5 kips /(0.8 x 270 ksi) = 0.833 in^2. Following conventional load and material factors adopted by structural codes for temporary and sustained loading conditions, the cross sectional area, A, of 0.833 in^2 was used for estimating the anchor stiffness value.

The next step involved calculating the stiffness of the anchor spring in the beam-column load-deflection model based on the stiffness equation K = AE/L. The length parameter L needs to be estimated. Conventional practice in DOTs is to assume L based on the length of the minimum unbonded soil zone as shown in the right side of Figure 1. Based on the Rankine active wedge failure surface, the lateral distance on the ground surface needs to be at least 13.8 feet. Realizing the basic uncertainty with this, a minimum no-bond length exceeding the 13.8 foot distance should be used. In the example problem, an unbonded length L value of 15 feet was assumed.

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In addition to specifying the minimum unbonded zone of 15 feet, the tieback anchor design also needs to estimate the bonded length based on an estimated ultimate soil strength. Following recommendations by FHWA, a bond length on the order of about 40 feet was assumed to provide for the specified proof test anchor load of 180 kip per anchor. Following conventional practice, the upper bound stiffness value estimated for the tieback stiffness would be AE/L = 0.833 in^2 x $29x10^6$ psi/(15 ft) = 1,610,467 lb per foot per anchor. After distributing over the 8-foot wall spacing, the anchor stiffness equals 201,308 lb per foot of wall width.

The bending stiffness of the sheet pile was also determined. Based on experience in prior example problems, a maximum static moment of 22,000 ft-lb per foot of wall width resulted in a sheet pile bending stiffness of 1.144 x 10^7 ft^2-lb per foot of wall width.

Seismic Loading

In addition to the static earth pressure diagram shown in Figure 1, Table 1 tabulates the incremental earth pressure for various horizontal seismic coefficients (k_{max}) following the procedure illustrated in Example No. 1 for the non-gravity cantilever wall (as tabulated in Table 1). Table 2 also shows the corresponding earthquake-induced earth pressure values for the 25 foot tieback wall problem. Similar to the static loading condition, the conventional dynamic active pressure load is increased by 30 percent to account for the stiffer tieback system for the earth pressure distribution above the excavation grade.

Table 1. Tabulation of Dynamic EQ Load Case for 25-foot Tieback Wall

Seismic Coefficient, k_{max}	**0.1**	**0.2**	**0.4**
Net EQ Earth Pressure Coefficient, $K_{ae\ (Net\ EQ)}$ from Figure 6	0.04	0.1	0.3
Total EQ Load on Wall (lb/ft) = ½X $K_{ae\ (Net\ EQ)}$XgXH^2 X 1.3	1,950	4,875	14,630
Distributed Pressure (psf)	78	195	585

Results of Analyses

Beam-column analyses were conducted for the above described problem, with the static earth pressure loading condition as shown in Figure 1, and then incremental dynamic earth pressure loads tabulated in Table 1 were superimposed over the basic static loading condition for the earthquake load cases..

Figure 2 presents the results of the beam-column analyses for the tieback sheet pile wall problem. The beam-column analyses indicate that the anchor forces would be 104, 116, 135, and 197 kips per anchor for the (1) static load case, (2) static plus k_{max} = 0.1, (3) static plus k_{max} = 0.2, and (4) static plus k_{max} = 0.4 seismic coefficient load cases, respectively.

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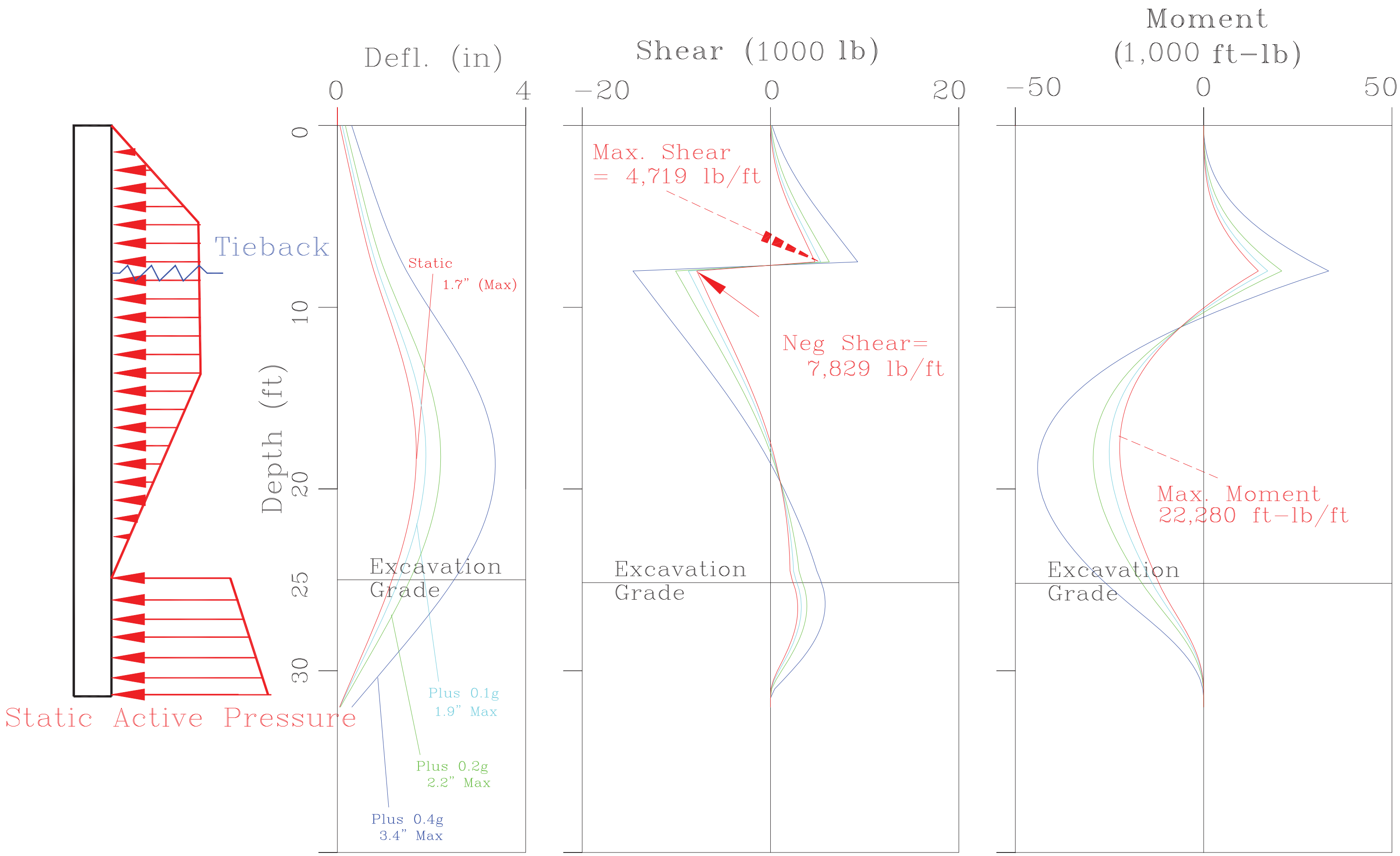

Figure 2. Beam-Column Solutions of the Tieback Sheet Pile Wall Problem.

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Discussion of Results

The basic model is a useful tool for conducting additional sensitivity studies, especially to account for basic uncertainty associated with the tieback anchor stiffness. The previous calculations correspond to an upper bound anchor stiffness associated with a minimum length L. Additional compliance of the bonded anchor zones would add to the compliance in the 15-foot unbonded anchor zone from elastic stretching. However, experience shows that the upper bound tieback stiffness value generally leads to a conservative anchor load for anchor design. A softer anchor stiffness, however, may control other elements of the system (e.g., the moment of the sheet pile wall), and hence it is good practice to conduct additional sensitivity analyses for a softer anchor stiffness to evaluate the potential for higher sheet pile moment.

Figure 3 presents results of an additional sensitivity evaluation for the tieback anchor wall with a lower tieback anchor stiffness. A much softer tieback anchor stiffness was developed using a length L = (15 + 40) = 55 feet. The resulting anchor stiffness would then be 0.273 times the upper bound stiffness value. The solutions shown in Figure 2 indicate that the effect of such a reduction in anchor stiffness would primarily be increases in the wall deflection to 3 inches maximum from the original peak deflection of approximately 2 inches. Otherwise, the anchor forces, and the sheet pile moment do not change significantly. It appears that the problem is primarily statically determinate and hence the resultant forces are not sensitive to the anchor stiffness. However, variations in the anchor stiffness might be a more significant design issue for tieback walls with multiple levels of anchors, especially at closer anchor spacing, when load redistribution can be more significant.

Based on results of the analyses for seismic coefficients k_{max} = 0.1, 0.2 and 0.4, it can be observed that the practice of proof loading the anchor to 1.5 times the static design load would inherently provide an acceptable design for high seismic loading condition, with k_{max} = approaching 0.4, which could correspond to a PGA close to 0.8g, assuming that the design k_{max} = 0.5 k_{max}. In many situations the amount of cohesive soil in the soil profile behind the wall will justify this reduction.

Note that the amount of deformation occurring in the wall under the highest seismic load is greater than several inches. This amount of displacement is consistent with the permanent displacement allowed for the design of a semi-gravity wall. However, as noted in the proposed Specifications, the design of an anchored wall would not be based on a 50% reduction in the seismic coefficient unless it could be shown that the permanent displacement associated with the 50% reduction would not damage the sheet pile wall. On the other hand a 50% reduction on the basis of wave scattering or cohesive soil effects would be considered appropriate.

References

Shamsabadi, A. (2006). *CT-FLEX – Computer Manual*, Office of Earthquake Engineering, California Department of Transportation.

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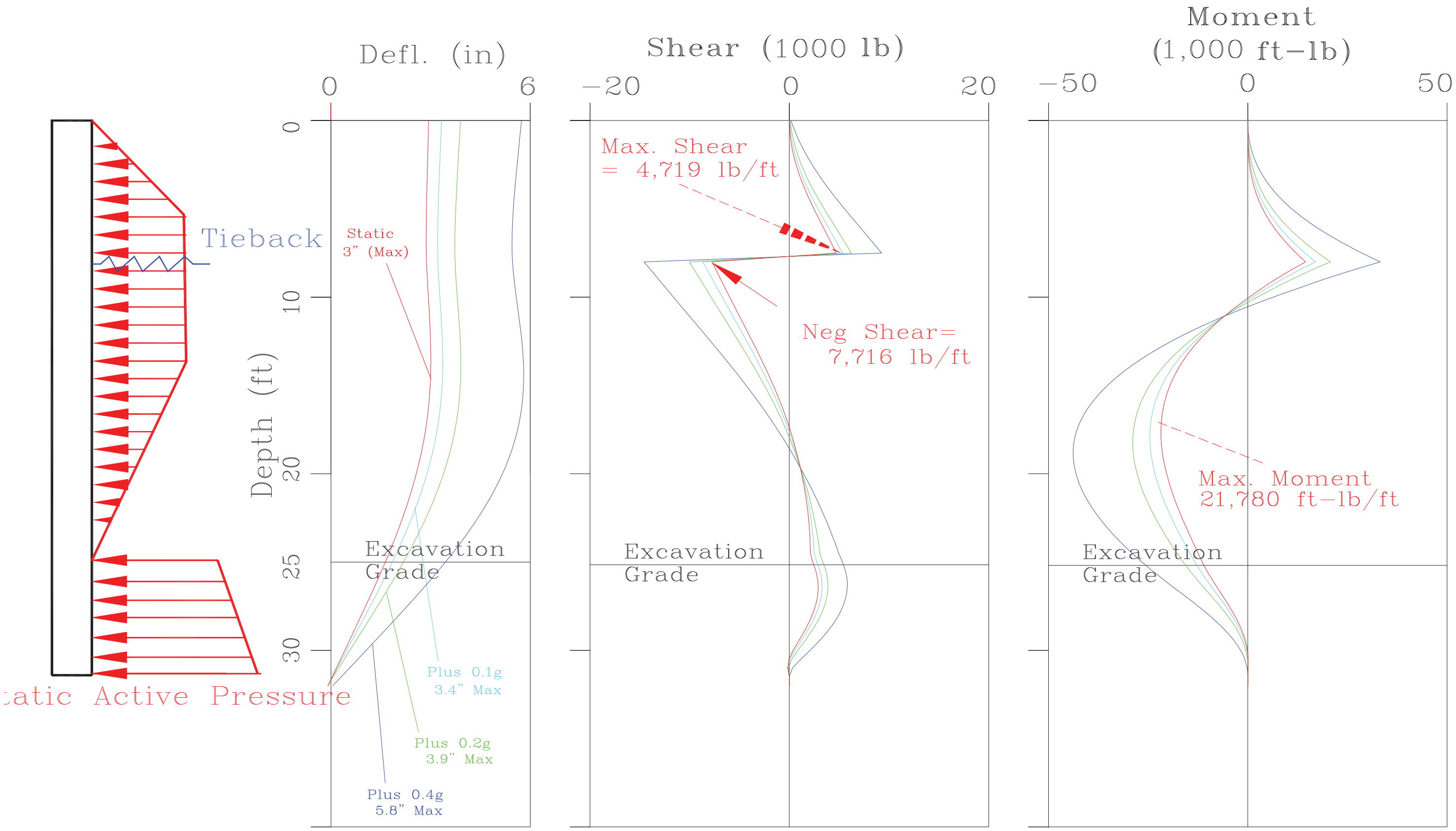

Figure 3. Beam-Column Solutions of the Tieback Sheet Pile Wall Problem for Softer Anchor Stiffness.

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Example MSE Wall Problem

Introduction

The following example demonstrates the application of the proposed procedure outlined in Section X.10 of the proposed Specifications for the seismic design of MSE Walls. The example focuses on the design procedure for external stability only. The procedure does not apply to internal stability evaluation.

The following subsections summarize (1) the slope geometry and soil properties used in the example, (2) the seismicity for the three sites considered, (3) the general methodology followed, (4) the results of the stability analyses, and (5) preliminary conclusions made from these analyses. Information from these analyses is used to develop a step-by-step presentation of the example for one of the cases (Appendix A).

MSE Wall Geometry and Soil Properties

The dimensions for the design example are based on example 4.6 from FHWA-NHI-00-043 (FHWA, 2001). The example wall is a 25.6 feet tall wall with steel strip reinforcement.

The foundation soil below the MSE wall is a cohesive soil with a friction angle of 10° and cohesion of 3,000 psf; therefore, deep failure planes through the foundation material were not a design consideration for this study. A firm-ground condition was assumed for the embankment base to avoid additional complexity from base failures that might be associated with liquefaction or soft ground conditions. It was assumed that some type of ground improvement would have to occur before walls of this height were constructed on either liquefiable soils or soft soils, resulting in conditions consistent with the example.

The retained soil is a granular soil with unit weight of 120 pcf, friction angle of 30° and zero cohesion. The reinforced soil has a unit weight of 120 pcf, friction angle of 34°, and zero cohesion. The MSE wall is drained and there is no hydrostatic pressure behind the wall.

The facing elements are 4-inch thick precast concrete panels; however, the facing design is not discussed here. Figure 1 shows the geometry of the MSE wall.

Seismicity

Three sites with different levels of seismic activity were included in this study. Two of the sites are located in the Western United States (WUS), one in Los Angeles area and the other one in Seattle. The third site is located in Central and Eastern United State region (CEUS), in Charleston, South Carolina.

Peak Ground Accelerations (PGA) for each site were determined from USGS/AASHTO Seismic Design Parameters for 2006 AASHTO Seismic Guidelines. Seismic accelerations were

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calculated for an average return period of 1,000 years. PGA values were initially determined for bedrock (Soil Type B) and modified for the foundation soil, assumed Soil Type D. A summary of site locations and seismicity data is given in Table 1.

Methodology

The methodology followed that outlined in Section X.10 of the proposed Specifications . The external stability was evaluated using a spreadsheet. For external stability evaluations, the reinforced soil is treated as a rigid block. The earth pressure is applied at one-third from the base for static condition, and mid-height for seismic condition.

Only failure planes through the fill were examined; the potential for deeper failure planes through the foundation material was not evaluated. The PGA was adjusted for slope-height effects when the maximum depth of the failure plane below the ground surface was greater than 20 feet, following the procedure recommended in the proposed Specifications. Since the MSE wall in this example is taller than 20 feet, adjusted accelerations were used in the external stability evaluation. For internal stability evaluations at different levels of reinforcement, different seismic acceleration values would be considered based on the height of the wall above that reinforcement level.

Based on these recommendations, the seismic acceleration was adjusted using the following equation:

$$k_{max} = \alpha F_{pga} PGA$$

For site category D, α is calculated from the following equation:

$$\alpha = 1 + 0.01 H [(0.5 \beta) - 1]$$

where H is the slope height in feet and β is calculated from the equation:

$$\beta = F_v S_1 / k_{max}$$

where $F_v S_1$ is the spectral acceleration coefficient at a period at one second adjusted for soil type.

The resulting seismic coefficient ($k_{max}/2$) was used in pseudo-static seismic stability analyses of the MSE wall as stated in the proposed Specifications. The k_{max} values used in design are shown in Table 2.

Newmark displacement correlations in Section X.4.5 of the proposed Specifications were used to estimate the wall movement during seismic loading for those cases where the capacity to demand (C/D) ratio for sliding was less than 1.0. Newmark deformation was estimated from the following equation:

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$$\log(d) = -1.51 - 0.74 \log(k_y/k_{max}) + 3.27 \log(1 - k_y/k_{max}) - 0.8 \log(k_{max}) + 1.59 \log(PGV)$$

where PGV was estimated from the following equation:

$$PGV = 55\ F_v\ S_1$$

The yield acceleration coefficient (k_y) for each case was calculated using the SLIDE computer program. The yield acceleration is the seismic acceleration that results in a C/D ratio of 1.0.

No dead or live load surcharge is considered in this study; therefore the MSE wall was designed for static and seismic conditions only. Load combinations Strength I and Extreme Event I (earthquake) were evaluated. Other load combinations are not controlling the design of the MSE wall.

Two sets of load factors for Strength I load combinations were used for initial static design of the MSE wall. One set induces the maximum eccentricity on the foundation, while the other set induces the maximum bearing pressure. These load combinations are differentiated by Strength I-a and Strength I-b designations. Load factors for these two combinations are summarized on Figure 2. Load factors of 1.0 were used for all loads in Extreme Event I load combination. These factors are shown on Figure 3. Load and resistance factors used in this study are summarized in Table 2.

A satisfactory design requires the wall to satisfy the criteria for eccentricity, sliding, and bearing capacity. The following eccentricity criteria, which are identical to AASHTO criteria, were adopted in this study:

- $e / W \leq 1/6$ for Strength I Load Case on Soil
- $e / W \leq 1/4$ for Strength I Load Case on Rock
- $e / W \leq 1/3$ for Extreme Event I Load Case (no live load)

Bearing resistance was checked using the equations recommended in Section 10 of the AASHTO *LRFD Bridge Design Specifications*.

Active Earth Pressure Coefficients

Active earth pressure coefficients were calculated using limit equilibrium methods with a seismic coefficient equal to 50% of k_{max} as defined in the proposed AASHTO Specifications. The active earth pressure was calculated based on the Coulomb method. For granular material closed-form Mononobe-Okabe solutions are available.

The active pressure was assumed to have a triangular distribution for static loading. The resultant active force for the static case was applied at the one-third point above the base of the wall. For the seismic case, a uniform distribution was assumed, and the resultant active load was applied at the mid-height of the wall. Active pressure for both static and seismic cases was applied to the

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back of the reinforced soil block, as shown in Figures 2 to 4. In order to make the results comparable with FHWA-NHI-00-043, the soil-on-soil friction angle at this imaginary line was assumed to be zero; however, larger friction angles (e.g., up to two-thirds of the internal friction angle of the soil) or the slope of the backfill may be used based upon designer discretion.

The passive earth pressure was neglected in the calculations because of the very shallow embedment depth of the MSE wall system. Additionally, if sliding failure were to occur, it would most likely develop at the base of the wall at the ground surface, and not below the ground surface. With this failure mechanism passive pressures would not be developed.

Due to the small coverage ratio of the steel strip reinforcement, it was assumed that the sliding along the lowest reinforcement level is not critical. For MSE walls reinforced with geo-synthetics and other reinforcements with large coverage ratios, the soil/reinforcement interface is likely to be critical in sliding evaluations. The low interface strength between soil and the geosynthetic makes this location particularly susceptible to base sliding.

Results of Analyses

Three analyses were performed, one for each seismic acceleration level. As discussed before and according to the proposed Specifications, 50% of k_{max} was used in seismic analysis. This approach assumes that some amount of deformation (1 to 2 inch) is acceptable. The reinforcement length was selected as 0.7H ≈ 18.5 feet. The calculated C/D ratios for each analysis are reported in Table 3.

As shown in Table 3, the C/D ratios are less than 1.0 for the seismic case (Extreme Event I load combination) for the two sites in WUS. The C/D ratios can be increased by using longer reinforcement; however, an acceptable design based on the owner's performance criteria may be achieved by adopting a displacement-based design, as noted in the proposed Specifications. As an example, the design of the retaining wall was further refined for the above cases using a displacement-based design approach as discussed below.

Displacement-based Design

In order to estimate the permanent displacement of the designed wall during the earthquake, the yield acceleration (k_y) for the MSE wall was calculated. Results of the yield acceleration calculation gave a k_y equal to 0.176, corresponding to C/D ratio of 1.0 for sliding (i.e., conventional FS = 1.0). At this acceleration level, the MSE wall satisfies all design criteria except sliding for the seismic case.

Newmark displacement was calculated using correlations in the proposed AASHTO Specifications:

$$\log(d) = -1.51 - 0.74 \log(k_y/k_{max}) + 3.27 \log(1 - k_y/k_{max}) - 0.8 \log(k_{max}) + 1.59 \log(PGV)$$

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where PGV can be estimated from the following equation:

$$PGV = 55\ F_v\ S_1 \qquad (2)$$

The C/D ratios for the yield acceleration are shown in Table 5. The Newmark displacements for all three cases are shown in Table 6. The displacement for Site 3 is negligible, and Site 2 displacement is judged to be acceptable for a majority of practical cases. Site 1 displacement is marginal and might be acceptable under some circumstances. Smaller displacements may be achieved by increasing the length of reinforcement.

The small displacements for two of the cases could allow the designer to use a seismic coefficient less than 50% of k_{max} for the designs in the two lower seismicity areas. For example, if 6-inch displacements during the seismic event are tolerable, the design seismic coefficient for Seattle could be reduced to about 0.35. This would further reduce the loading demands on the wall for seismic loading.

Concluding Comments

These results show that based on external stability criteria MSE walls designed for static conditions using criteria in the proposed Specifications perform very well for lower levels of seismicity typical in CEUS. For WUS sites, these walls are expected to undergo permanent displacements during large seismic events, unless they are specifically designed for the seismic condition and longer reinforcement is used. The magnitude of this displacement depends on several factors such as foundation soil type and backfill slope.

References

FHWA (2001). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines. Federal Highways Administration, Publication No. FHWA-NHI-00-043, March.

Table 1. Site Coordinates and Seismicity Data

Site Coordinates		Region	Soil Type B (Bedrock)		Soil Type D	
Longitude	**Latitude**		**PGA**	**S_1[1]**	**$F_{pga}PGA$**	**F_vS_1[1]**
-117.9750	34.0500	WUS (Los Angeles)	0.60	0.52	0.60	0.78
-122.2500	47.2700	WUS (Seattle)	0.40	0.30	0.46	0.54
-079.2370	33.1000	CEUS (Charleston)	0.20	0.10	0.30	0.24

1. Spectral acceleration coefficient at 1.0 second period.

Table 2. Height-Adjusted Seismic Accelerations for Site Class D

Longitude	Latitude	Region	$F_{pga}PGA$	F_vS_1[1]	β	H [ft]	α	k_{max}
-117.9750	34.0500	WUS (Los Angeles)	0.60	0.78	1.30	25.6	0.91	0.546
-122.2500	47.2700	WUS (Seattle)	0.46	0.54	1.16	25.6	0.89	0.411
-079.2370	33.1000	CEUS (Charleston)	0.30	0.24	0.80	25.6	0.85	0.252

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Table 3. Load Factors and Resistance Factors

Load	Load Factor		
	Strength I-a	**Strength I-b**	**Extreme Event I**
EAH Active earth pressure, horizontal component	1.50	0.90	1.00
EAV Active earth pressure, vertical component	1.00	1.35	1.00
EV Vertical soil pressure	1.00	1.35	1.00
Sliding	**Resistance Factor**[1]		
	Strength I-a	**Strength I-b**	**Extreme Event I**
Cohesion, c	0.80	0.80	1.00
Friction angle, ϕ	0.80	0.80	1.00
Soil on Soil	1.00	1.00	1.00
Bearing Capacity	**Resistance Factor**[1]		
	Strength I-a	**Strength I-b**	**Extreme Event I**
Cohesion, c	0.60	0.60	1.00
Friction angle, ϕ	0.55	0.55	1.00

1. Resistance factor for earth material in AASHTO depends on soil investigation method.

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Table 4. Summary of Analyses Results for Three Sites

		Active Earth Pressure Coefficient			Capacity/Demand: Strength I-a			Capacity/Demand: Strength I-b			Capacity/Demand: Extreme Event I		
PGA	50% k_{max}	Static k_a	Seismic k_{ae}	Foundation Width, W [ft]	Eccentricity e/W	Bearing	Sliding Horizontal	Eccentricity e/W	Bearing	Sliding Horizontal	Eccentricity e/W	Bearing	Sliding Horizontal
0.6	0.273	0.333	0.541	18.5	0.159	3.62	1.95	0.071	3.41	4.39	0.448[1]	0.90[1]	0.43[1]
0.46	0.205	0.333	0.478	18.5	0.159	3.62	1.95	0.071	3.41	4.39	0.371[1]	2.25	0.80[1]
0.30	0.126	0.333	0.415	18.5	0.159	3.62	1.95	0.071	3.41	4.39	0.286	3.76	1.44

1. Design criteria not satisfied.

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Table 5. Summary of Analyses Results for Yield Acceleration

	Active Earth Pressure Coefficient			Capacity/Demand: Strength I-a			Capacity/Demand: Strength I-b			Capacity/Demand: Extreme Event I		
k^y	Static k_a	Seismic k_{ae}	Foundation Width, W [ft]	Eccentricity e/W	Bearing	Sliding Horizontal	Eccentricity e/W	Bearing	Sliding Horizontal	Eccentricity e/W	Bearing	Sliding Horizontal
0.176	0.333	0.453	18.5	0.159	3.62	1.95	0.071	3.41	4.39	0.339	2.82	1.00

1. Yield acceleration (k_y) was used in seismic analysis case.

Table 6. Newmark Displacements for Three Sites

F_{pga}PGA	k_{max}	Yield Acceleration k_y	PGV [in/sec]	Newmark Displacement [in]
0.60	0.546	0.176	43.0	11.9
0.46	0.411	0.176	29.4	3.7
0.30	0.252	0.176	13.0	0.1

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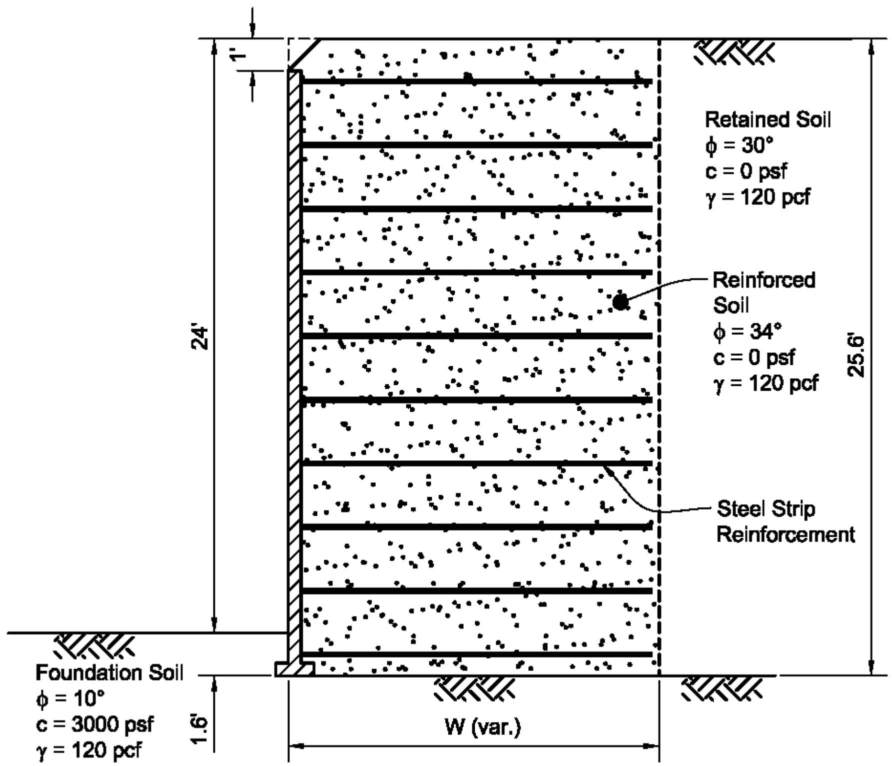

Figure 1. MSE Wall Geometry

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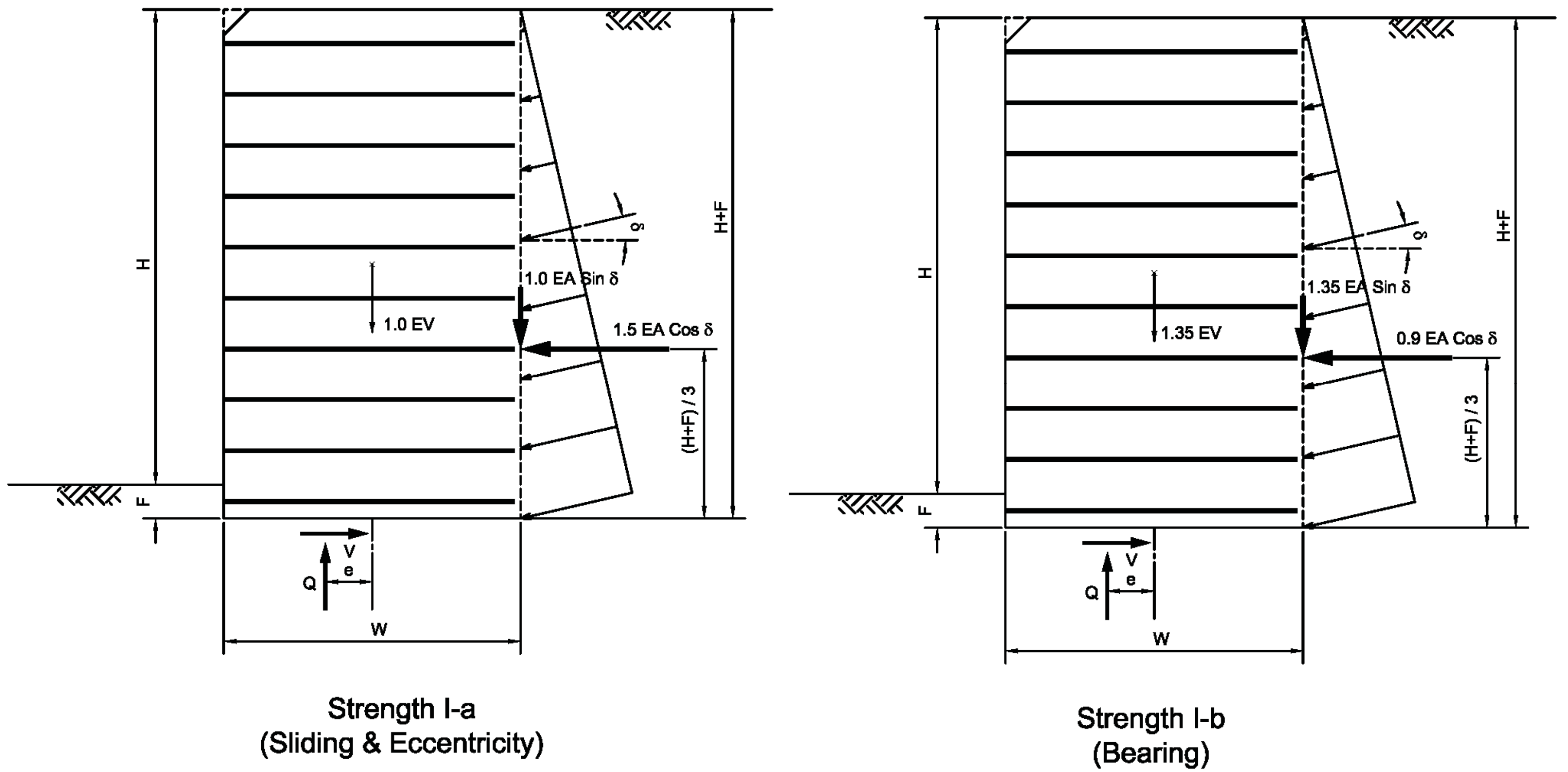

Figure 2. Strength I Load Combination for External Stability

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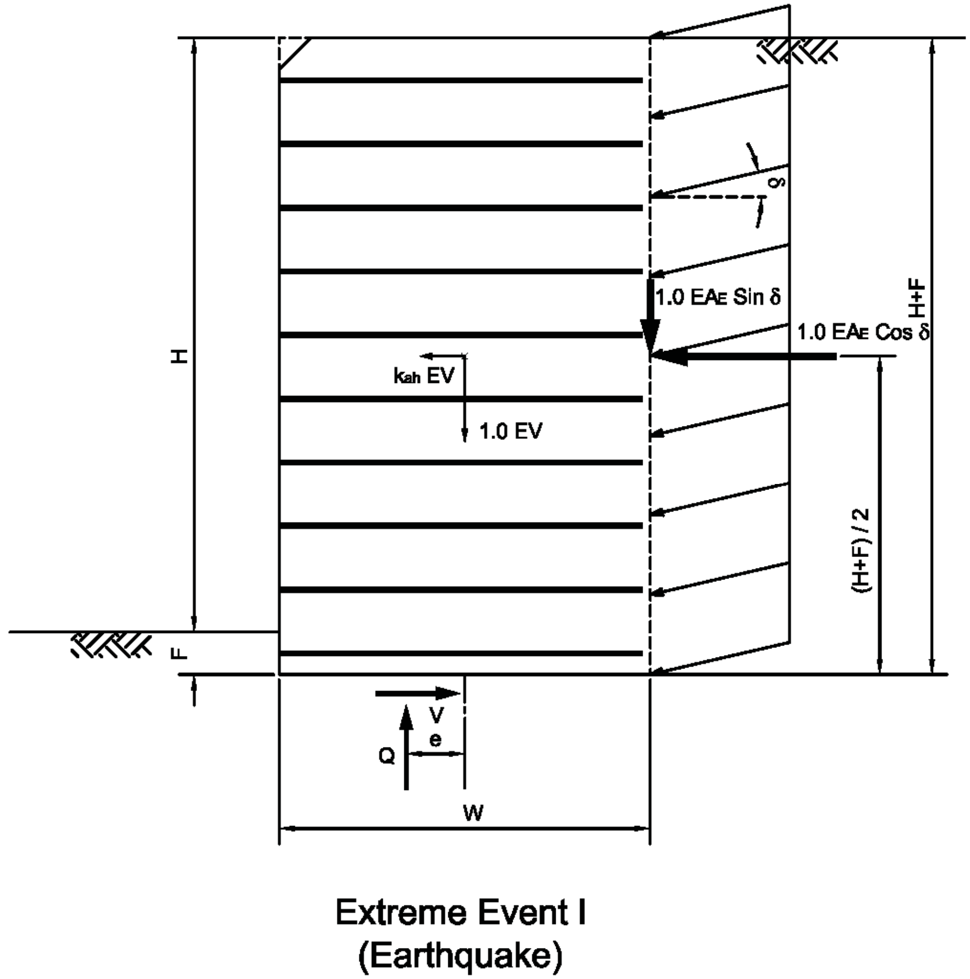

Figure 3. Extreme Event I Load Combination for External Stability

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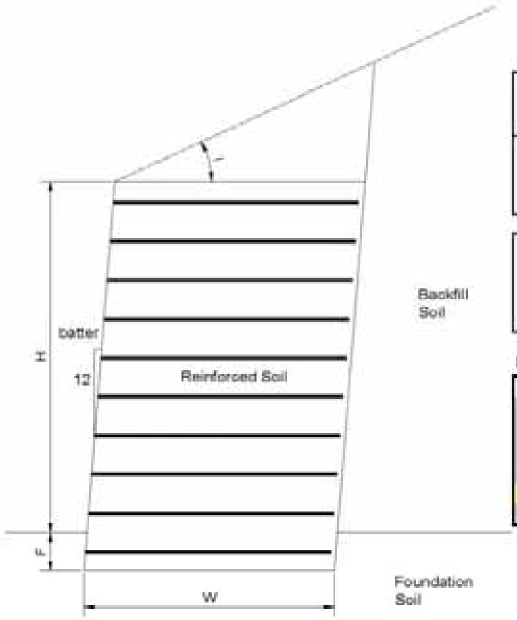
Appendix A: Screen Shot of Spreadsheet Used for Calculations



Calculation

	LRFD Factors Load min	max	Resistance
DC	0.9	1.25	
EH	0.9	1.5	
ES	1.5	0.75	
EV	1	1.35	

	c		ϕ	
Passive	0.6	0.6	0.55	0.55
Bearing	0.6	0.6	0.55	0.55
Sliding	0.8	0.8	0.8	0.8
Soil on Soil	1	1	1	1
Passive Sliding	0.5	0.5	0.5	0.5

load combo: *heel pressure*

MSE wall dimensions:

	length (ft)
Design H	24.00
W	18.50
Batter	0 : 12
i	0.00 deg
β=Atan(Batter)	0.00 deg
F	1.60 ft

(grey)	**Input Data**
(yellow)	**Calculation Result**

Material properties

$\gamma_{backfill}$	120.00 pcf
γ_{slope}	120.00 pcf
γ_{found}	120.00 pcf
γ_{reinf}	120.00 pcf
δ active*	0.00 deg
δ soil-footing	10.00 deg
$\phi_{backfill}$	30.00 deg
ϕ_{reinf}	34.00 deg
ϕ_{found}	10.00 deg
K_v **	0.00
K_h	0.273
% k on reinf. Mass	100%
θ	15.27 deg
i	0.00 deg
δ soil-reinf	30.00 deg
β soil-wall	0.00 deg
$c_{backfill}$	0.00
c_{found}	3000.00
K_a (Coulomb)	0.33
K_{ae} (Coulomb)	0.54
ΔK_{ae}	0.21

* angle from horizontal
** positive down

		Weight (lbs/ft)	Arm (ft)	Load Factor
	W_{wall} =	56832	9.25	1.35
	W_{soil} =	0	12.333	1.35
	Pa =	13094		
	Pax =	13094	8.53	0.9
	Pay =	0	18.50	1.35
	Pae =	21273		
	Paex =	21273	12.80	1
	Paey =	0	18.50	1
horizontal inertial load on soil	ΔFex =	0	25.60	1
horizontal inertial load on wall	ΔFwx =	15515	12.800	1
vertical inertial load on soil	ΔFsy =	0	12.33	1
vertical inertial load on wall	ΔFwy =	0	9.25	1
	h' =	25.60	ft	
	h'' =	0.000	ft	

	Static State	Seismic State	
Q =	76723	56832	lbs
e =	1.311	8.286	ft
q1 =	5910	11327	lb/ft2
q2 =	2384	-5183	lb/ft2
Lx =	23.82	2.89	ft
q1' =	5910	39287	lb/ft2
q2' =	2384	0	lb/ft2
qx =	5910	39287	lb/ft2
T =	11785	36788	lbs
e/w =	0.071	0.448	
B' =	15.88	1.93	ft
Pmax =	51750	15807	lbs

$\phi\, q_{u,static}$ (psf)	C/D
16,455	3.41

$q_{u,seismic}$ (psf)	C/D
26,523	0.90

Sliding stability (sliding on foundation soil, static state)

Fs,slide = $\frac{Pmax+\phi_s Pp}{Pa}$ **=4.39** **>=1.0** **safe**

Sliding stability (sliding on foundation soil, seismic state)

Fs,slide = $\frac{Pmax+\phi_s(Pp+\Delta Ppe)}{Pa+\Delta Pae}$ **=0.43** **<1.0** **unsafe**

Overturning

Foundation type: (soil/rock) soil

W/6 =	**3.08 >= e**	**static**	**safe**
W/3 =	**6.17 < e**	**seismic**	**unsafe**

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Geotechnical & Earthquake Engineering

Example Soil Nail Wall Problem

Introduction

This example problem is for a soil nail wall that is located in California. The wall is being designed to Caltrans' requirements for gravity and seismic loading. The computer program SNAIL was used to conduct the soil nail design. Independent checks on stability were performed using the computer program PCSTABL.

The wall is 60 feet in height at its maximum and will be approximately 170 feet in length. The wall will have a 6V:1H batter and an 8-foot wide bench roughly at mid-height. The intent of the bench is to reduce the driving force on the wall to reduce nail lengths for the upper portion of soil nail wall. Figure 1 shows a plan view of the site, and Figure 2 shows two sectional views of the wall.

The soil nail design will generally follow normal soil nail design practice in the United States:

- Initial design was based on No. 10 steel bar nails with a 75 ksi strength installed in 8-inch diameter drilled holes. Hole inclination was assumed to be approximately 15 degrees from the horizontal.

- Vertical spacing of the nails was assumed to be approximately 4 to 5 feet on center. Nail lengths will range from 0.8 to 1.5 times the slope height (H), based on experience from other walls constructed in similar soil and seismic conditions.

Soils at the site consist of dense, partially cemented sand and colluvium with 4 to 20% fines. The field investigation used to characterize the site included soil borings with Standard Penetration Test (SPT) blowcounts and cone penetrometer tests. Figure 1 shows the location of explorations that were performed. Uncorrected blowcounts ranged from 20 to 70 blows per foot. End resistance values from CPT soundings typically varied from 100 to 200 tsf. Results of unconsolidated undrained and direct shear tests were available to interpret strength parameters of the soil.

Groundwater is located below the base of the wall excavation

Soil Nail Design Parameters

After review of the site conditions, including local seismic environment, the following design parameters were selected for use in the SNAIL analysis:

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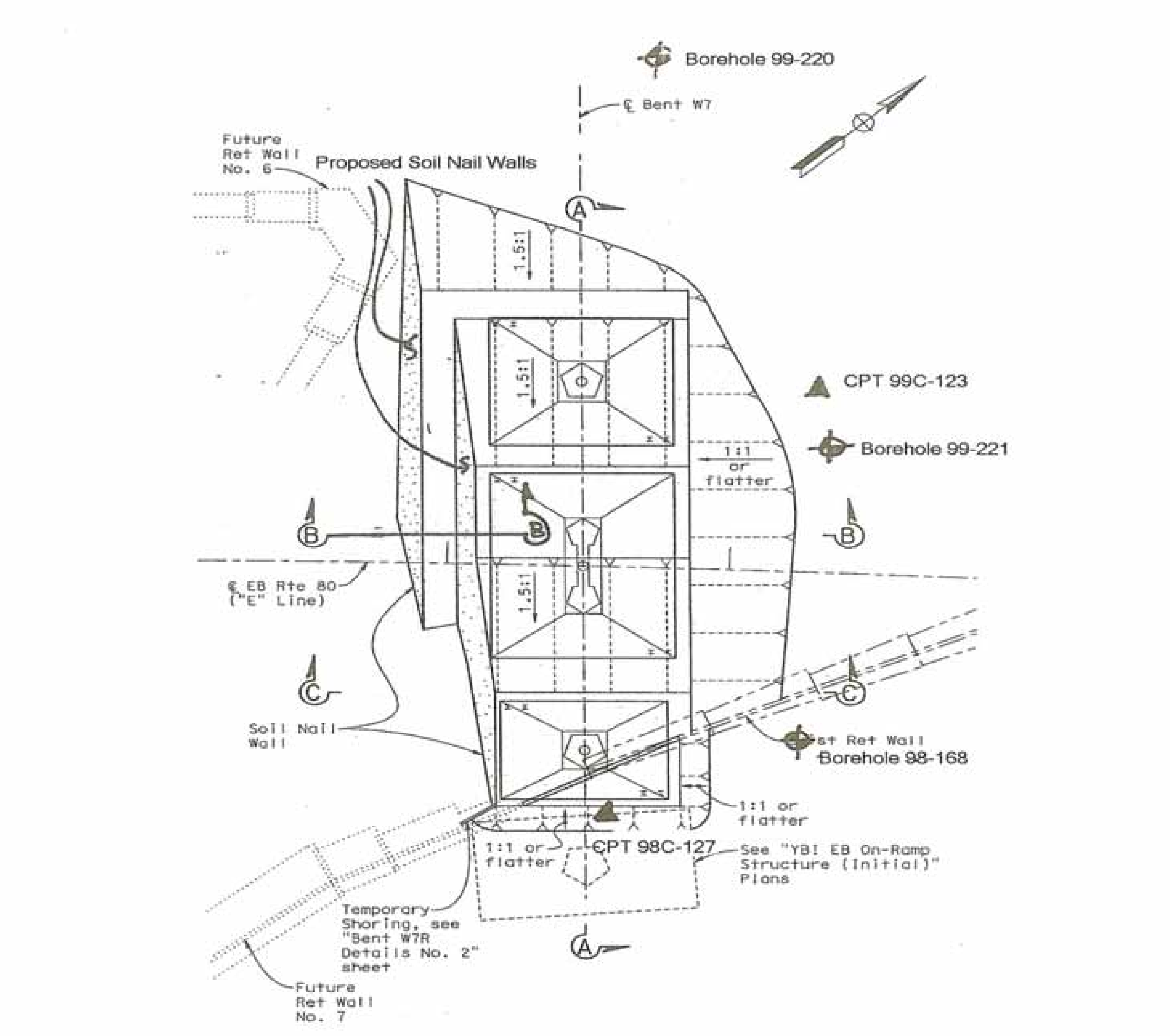

Figure 1. Site Plan for Soil Nail

CVO\081750026

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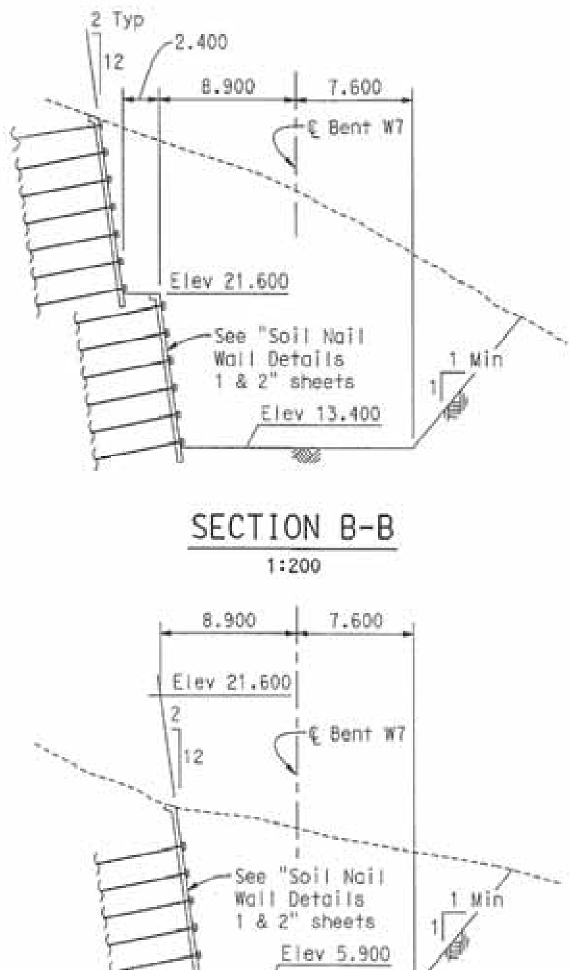

Figure 2. Cross-section Views for Soil Nail Wall Design

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- Soil strength

  - Upper soil: ϕ= 33 degrees, c = 150 psf
  - Lower Sand: ϕ= 35 degrees, c = 250 psf

  The cohesion intercept for static strength parameters was assumed to be the result of some cementation and the contributions from fine-grained soil. In the absence of specific laboratory testing and in view of the general consistency of the soil, the same strength parameters were used for seismic loading

- Ultimate bond strength of nail at soil-grout interface = 10 psi

  - Based on literature review of typical values (e.g., FHWA, 2003)
  - Results of nearby nail pullout tests for other projects

- Design seismic coefficient = 0.2. Vertical coefficient = 0.0 based on normal practice. The value of 0.2 is the default value used by Caltrans. A separate check on this value was made on the basis of the AASHTO hazards maps for a 1,000-year return period. This check is summarized below.

  - Soil Type D with k_{max} = F_{pga} PGA = 0.6 and site-adjusted spectral acceleration coefficient at 1 second (F_v S_1) = 0.78

  $\beta = F_v S_1 / k_{max} = 1.3$

  $\alpha = 1 + 0.01H\ (0.5\ \beta - 1) = 0.79$

  $k_{av} = \alpha\ k_{max} = 0.47$

  The Owner will allow several inches of movement during the design seismic event.

  $K_{av\text{-}adj} = 0.5 * \alpha\ k_{max} = 0.23g$

  The value of 0.2 in the analyses implies that the deformations will be slightly greater than the 1 to 2 inches implied by use of 0.5 k_{max}. Following the methods of Article X.4 in Section X of the proposed Specifications, the displacement was estimated to be as follows:

  $\log(d) = -1.51 - 0.74 \log(k_y/k_{max}) + 3.27 \log(1 - k_y/k_{max}) - 0.80 \log(k_{max}) + 1.59 \log(PGV)$

  0.6 to 0.8

  $d =$ 4 to 6 inches

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These displacements are within the range that would normally be accepted for the design of a soil nail wall – suggesting that the Caltrans' use of 0.2 is acceptable.

Soil-Nail Design

The soil nail wall was designed using the computer program SNAIL following Caltrans' requirements and the FHWA guidelines.

- Three nail failure mechanisms were considered during design:
  - Nail pullout – C/D ratio (factor of safety) of 2.0 for static and 1.5 for seismic – applied to ultimate bond strength per FHWA manual.
  - Nail yielding – C/D ratio of 1.8 was applied to the steel yield stress to obtain an "allowable" yield stress. The maximum nail capacity is 75 ksi x 1.27 in^2 = 95 kips/nail. The allowable capacity is 95 kips/1.8 = 53 kip/nail
  - Punching shear – capacity at nail head assembly/shotcrete wall was set to a high value. The C/D ratio was 1.5 was applied to the ultimate shear of 100 kips giving an allowable capacity of 67 kips. Initial SNAIL runs assumed that the nail head assembly and facing are structurally designed to exceed the maximum capacity of the nail loads.
- SNAIL analyses were conducted for the static and seismic load cases.
  - A 240 psf surcharge was applied to the backslopes and bench
  - Seismic coefficient of 0.2 used

  Nail lengths were varied until minimum C/D ratio for global stability of both 1.35 (temporary static condition) and 1.1 for the seismic condition were achieved. The theoretical model of failure was expected to be internal; e.g., failure planes begin near the toe and traverses through the system of nails. The C/D ratio is based on mobilized nail forces determined from the nail lengths beyond the theoretical critical failure planes.
- For the critical static and seismic cases, the maximum nail load and the average nail load were determined from the SNAIL output. The maximum nail design force was determined per Section 5.5.4 of FHWA (2003). This guide indicates that a balanced design is achieved for all three failure mechanisms when the soil and nail strengths are fully mobilized simultaneously. The maximum nail force is calculated for the case

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when the global stability C/D ratio of the wall and soil system would be 1.0. This value was determined to be 53 kips (the allowable capacity).

- The design was verified using the slope stability program PCSTABL for the predefined nail forces. The resulting minimum C/D ratio for static loading was approximately 1.8, and the C/D ratio for seismic loading was 1.2.

- The structural designers will be responsible for designing the wall thickness and nail head assembly detail. If the capacity of this connection is less than the nail tensile capacity, the capacity is then returned to the geotechnical engineer to rerun the SNAIL analyses – which would result in a different distribution of nail forces. For this project punching shear did not control the SNAIL analyses.

- The geotechnical engineer conducted global stability analyses to show a C/D ratio of 1.3 for temporary conditions and 1.1 for the seismic case, and bearing capacity evaluations to show C/D of 2.0 for static temporary and 1.5 for the seismic case.

Results of Analyses

The results of the nail design are as follows:

- Nail No. 10 bar, 75 ksi grade, tremie grouted

- Hole: 8-inch diameter, dry auger

- Hole inclination: 15 degrees from horizontal

- Vertical Spacing: 4 to 5 feet

- Nail lengths: 43 to 53 feet

- Corrosion protection: minimum per Appendix C.3.2.1 of FHWA (2003)

- Lock off: wrench tight

- Facing embedment: 2-foot was used to limit excavation height and avoid interference with nails (typical 0.1H or 2 feet per Caltrans BDS 5-9.1)

Figure 3 shows the layout of the soil nails based on the SNAIL analyses, and Figure 4 shows details for the soil nail design. Appendix A to this example shows the output of the SNAIL analyses for gravity and seismic loads.

A testing program was developed for the nails. This program required performance testing of nails before production installation and approximately 5% of the production nails. The

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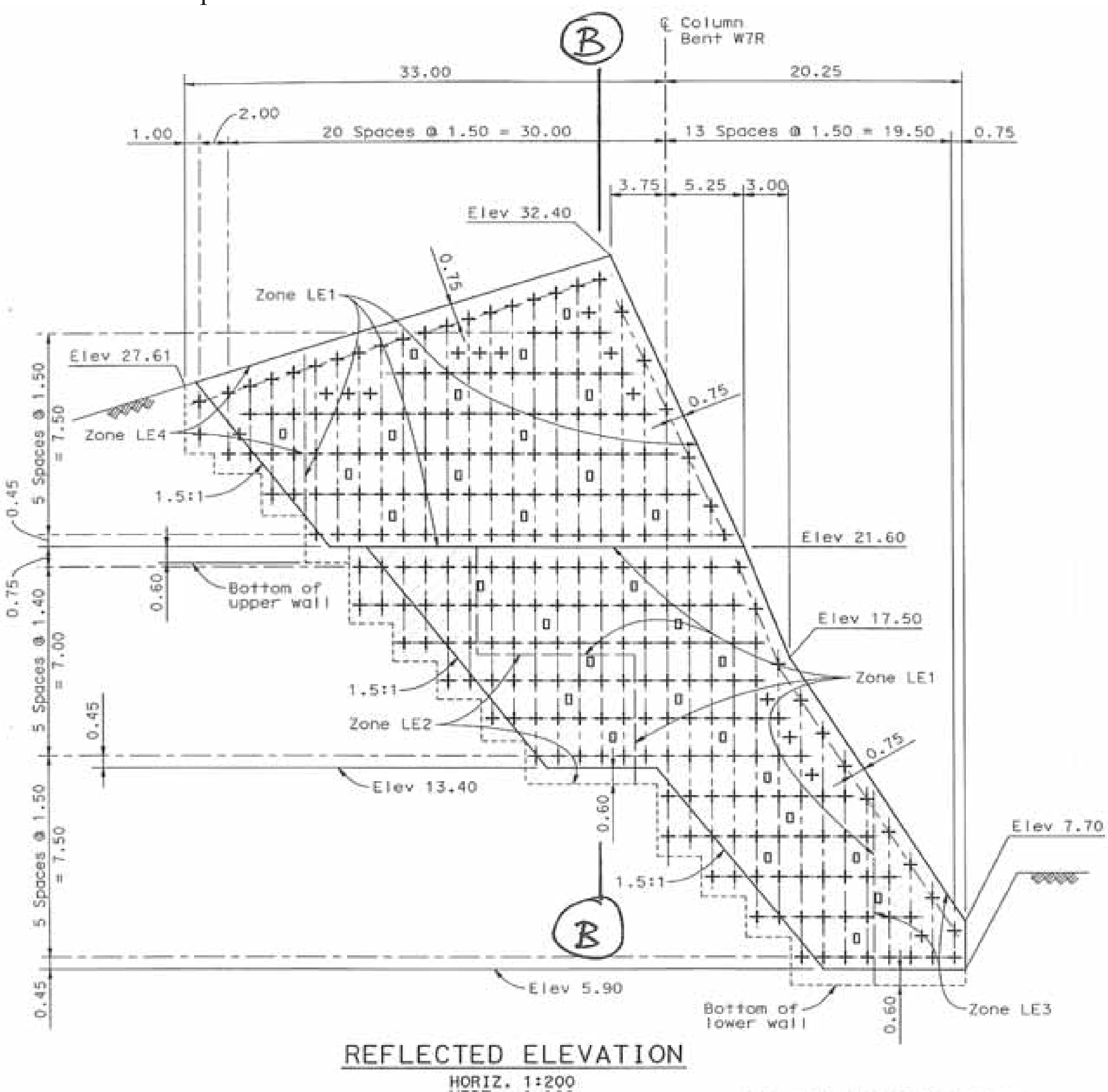
maximum load was 1.5 times the maximum calculated nail load following FHWA guidelines and Caltrans' Specifications.



Notes:

1. Exact location of test soil nails to be determined by Engineer in the field.
2. For weepholes at Soil Nail Wall, see "Soil Nail Wall Details No. 2" sheet.

+ Indicates location of soil nail assembly

▯ Indicates location of test soil nail

SOIL NAIL EMBEDMENT LENGTHS

LOCATION	Le METERS
Zone LE1	16.2
Zone LE2	13.2
Zone LE3	11.2
Zone LE4	10.2

Figure 3. Layout of Soil Nails

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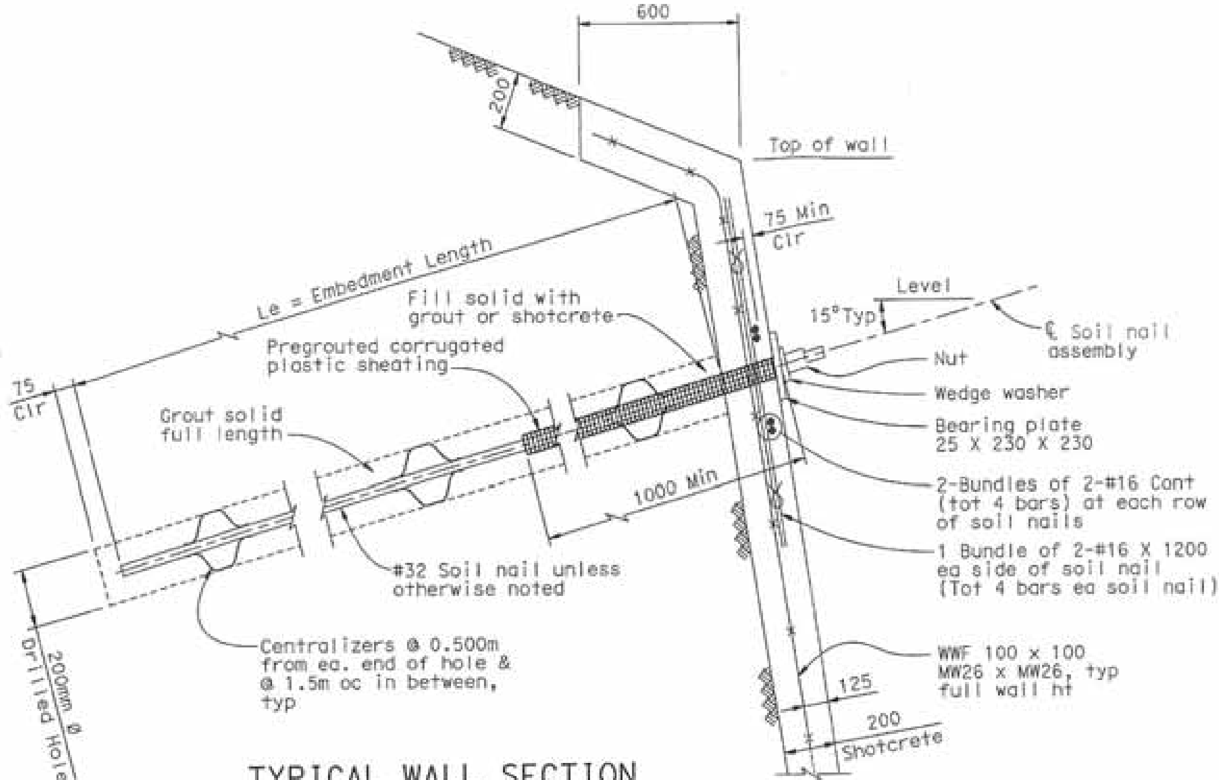

TYPICAL WALL SECTION

NO SCALE

SOIL NAIL WALL DESIGN DATA

Shotcrete: f'c = 22MPa

Soil Nails: ASTM Designation: A615/A615M, Grade 520
fy = 520MPa

Soil Parameters: Within 10m below top of wall

Unit Weight = 19.3 kN/m3
Soil Friction Angle: 33 degrees
Cohesion: 7.2 kPa
Ultimate Bond Stress = 70 kPa (between soil nail grout & drilled hole)

Other locations

Unit Weight = 19.6 kN/m3
Soil Friction Angle: 35 degrees
Cohesion: 12.0 kPa
Ultimate Bond Stress = 70 kPa (between soil nail grout & drilled hole)

Design Loads:
D = Dead Load
LL = Live Load
E = Lateral Earth Pressure
EQE = Seismic Earth Load = 0.20g

Safety Factors: Minimum Factor Of Safety, DL + LL + E = 1.35
DL + E + EQE = 1.10

Figure 4. Anchored Wall Details

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Conclusions

The use of the computer program SNAIL made application of the proposed Specifications relatively simple. The proposed requirements in Article X.4 were used to estimate the seismic coefficient to use in the design. Both wave scattering and permanent displacement corrections were considered in defining the design seismic coefficient – which was consistent with the Caltrans' default seismic coefficient. With this approach small permanent displacements were considered acceptable.

References

FHWA (2003). Geotechnical Engineering Circular No. 7: Soil Nail Walls, Federal Highway Administration, Report Nol. FHWA0-1F-03-017, March.

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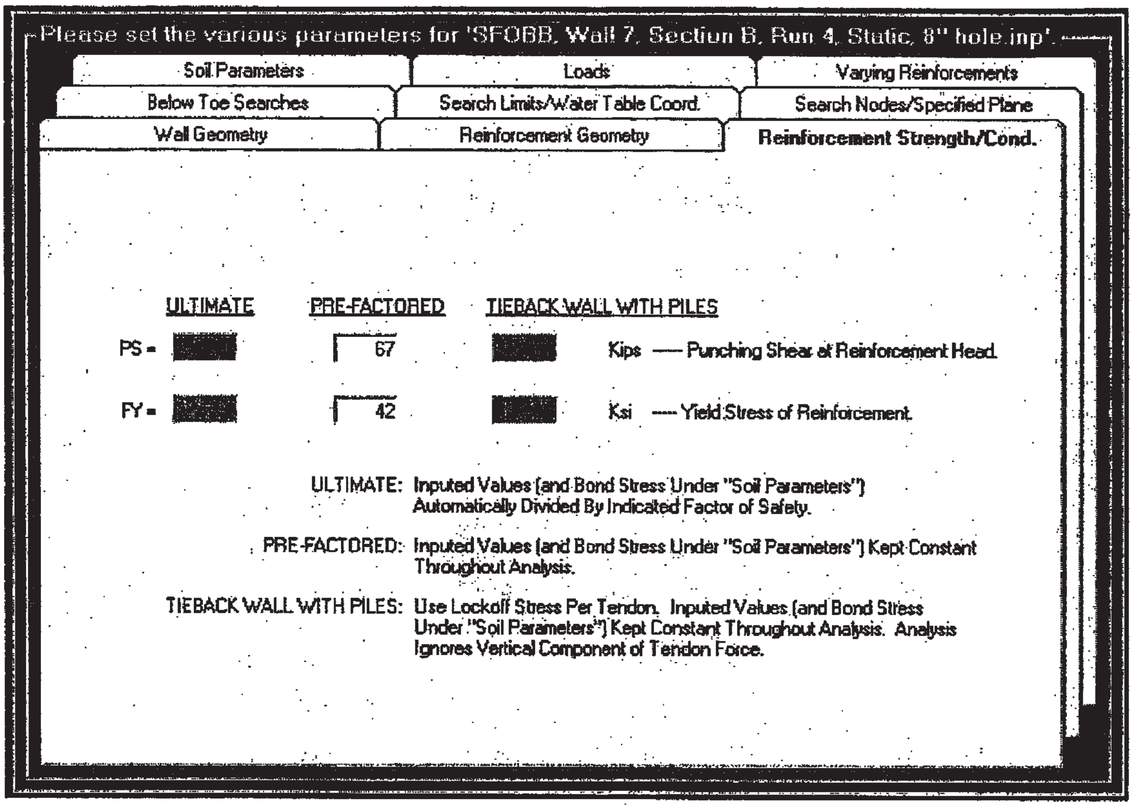
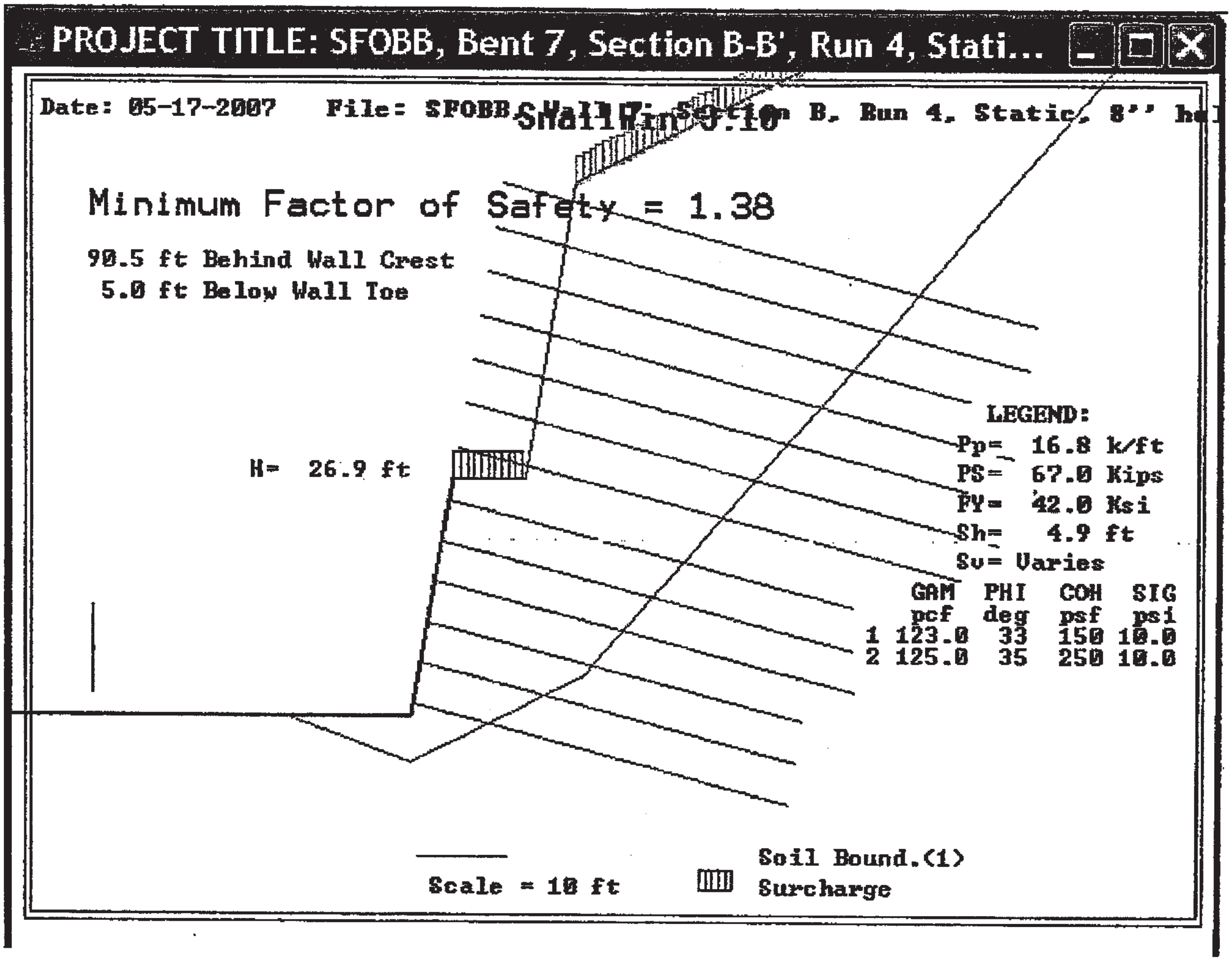
Appendix A – Computer Printouts from SNAIL Analyses



Nail structural design parameters in SNAIL analysis


PROJECT TITLE: SFOBB, Bent 7, Section B-B', Run 4, Stati...
Date: 05-17-2007
Minimum Factor of Safety = 1.38
90.5 ft Behind Wall Crest
5.0 ft Below Wall Toe
H= 26.9 ft
LEGEND:
Pp= 16.8 k/ft
PS= 67.0 Kips
FY= 42.0 Ksi
Sh= 4.9 ft
Sv= Varies
GAM pcf PHI deg COH psf SIG psi
1 123.0 33 150 10.0
2 125.0 35 250 10.0
Scale = 10 ft
Soil Bound.(1)
Surcharge

```
File: SFOBB, Wall 7, Section B, Run 4, Static, 8'' hole
Page -  1

              **************************************************
              *    CALIFORNIA DEPARTMENT OF TRANSPORTATION     *
              *          ENGINEERING SERVICE CENTER            *
              *     DIVISION OF MATERIALS AND FOUNDATIONS      *
              *   Office of Roadway Geotechnical Engineering   *
              *    Date: 05-17-2007         Time: 12:58:30     *
              **************************************************

Project Identification - SFOBB, Bent 7, Section B-B', Run 4, Static, 8'' hole

             --------- WALL GEOMETRY ---------

 Vertical Wall Height          =  26.9 ft
 Wall Batter                   =   9.5 degree
                                 Angle   Length
                                 (Deg)   (Feet)
 First Slope from Wallcrest.  =   0.0      7.9
 Second Slope from 1st slope. =  80.5     33.5
 Third Slope from 2nd slope.  =  26.5     50.0
 Fourth Slope from 3rd slope. =  18.4    100.0
 Fifth Slope from 3rd slope.  =   0.0    100.0
 Sixth Slope from 3rd slope.  =   0.0      0.0
 Seventh Slope Angle.         =   0.0

          --------- SLOPE BELOW THE WALL ---------

 First Slope Angle below Toe.        =   0.0 degrees
 First Slope Distance from Toe.      =   0.0 ft
 Second Slope Angle.                 =   0.0 degrees
 Second Slope Distance from Toe.     =   0.0 ft
 Vertical Depth of Search.           =   5.0 ft
 Number of Searches below wall Toe.  =   3

               --------- SURCHARGE ---------

 THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

 Begin Surcharge - Distance from toe =     18.0 ft
 End Surcharge - Distance from toe   =     60.0 ft
 Loading Intensity - Begin           =    240.0 psf/ft
 Loading Intensity - End             =    240.0 psf/ft

 Begin Second Surcharge - Distance from toe =      4.5 ft
 End Second Surcharge - Distance from toe   =     12.5 ft
 Loading Intensity - Begin           =    240.0 psf/ft
 Loading Intensity - End             =    240.0 psf/ft

                --------- OPTION #1 ---------

 Factored Punching shear, Bond & Yield Stress are used.
```

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	123.0	33.0	150.0	10.0	0.0	0.0	0.0	0.0
2	125.0	35.0	250.0	10.0	0.0	20.0	100.0	20.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 100.0 to 50.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	=	13
Horizontal Spacing	=	4.9 ft
Yield Stress of Reinforcement	=	42.0 ksi
Diameter of Grouted Hole	=	8.0 in
Punching Shear	=	67.0 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	61.0	15.0	33.1	1.27	0.50
2	61.0	15.0	4.9	1.27	0.50
3	61.0	15.0	4.9	1.27	0.50
4	61.0	15.0	4.9	1.27	0.50
5	61.0	15.0	4.9	1.27	0.50
6	61.0	15.0	4.9	1.27	0.50
7	61.0	15.0	4.9	1.27	0.50
8	48.0	15.0	6.0	1.27	0.50
9	48.0	15.0	4.6	1.27	0.50
10	48.0	15.0	4.6	1.27	0.50
11	43.0	15.0	4.6	1.27	0.50
12	43.0	15.0	4.6	1.27	0.50
13	43.0	15.0	4.6	1.27	0.50

File: SFOBB, Wall 7, Section B, Run 4, Static, 8'' hole
Page - 3

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.50	95.0	26.1	21.2	47.8	113.1

Reinf. Stress at Level 1 = 18.072 Ksi (Pullout controls...)
2 = 21.687 Ksi (Pullout controls...)
3 = 25.301 Ksi (Pullout controls...)
4 = 28.916 Ksi (Pullout controls...)
5 = 32.530 Ksi (Pullout controls...)
6 = 36.145 Ksi (Pullout controls...)
7 = 39.759 Ksi (Pullout controls...)
8 = 28.696 Ksi (Pullout controls...)
9 = 32.070 Ksi (Pullout controls...)
10 = 35.443 Ksi (Pullout controls...)
11 = 35.204 Ksi (Pullout controls...)
12 = 42.000 Ksi (Yield Stress controls.)
13 = 42.000 Ksi (Yield Stress controls.)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
1.67	1.46	95.0	26.5	21.2	48.3	114.2

Reinf. Stress at Level 1 = 17.360 Ksi (Pullout controls...)
2 = 20.908 Ksi (Pullout controls...)
3 = 24.457 Ksi (Pullout controls...)
4 = 28.006 Ksi (Pullout controls...)
5 = 31.555 Ksi (Pullout controls...)
6 = 35.103 Ksi (Pullout controls...)
7 = 38.652 Ksi (Pullout controls...)
8 = 27.509 Ksi (Pullout controls...)
9 = 30.821 Ksi (Pullout controls...)
10 = 34.133 Ksi (Pullout controls...)
11 = 32.732 Ksi (Pullout controls...)
12 = 39.503 Ksi (Pullout controls...)
13 = 42.000 Ksi (Yield Stress controls.)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
3.33	1.42	95.0	26.9	21.3	48.8	115.3

Reinf. Stress at Level 1 = 16.667 Ksi (Pullout controls...)
2 = 20.152 Ksi (Pullout controls...)
3 = 23.637 Ksi (Pullout controls...)
4 = 27.121 Ksi (Pullout controls...)
5 = 30.606 Ksi (Pullout controls...)
6 = 34.091 Ksi (Pullout controls...)
7 = 37.576 Ksi (Pullout controls...)
8 = 26.355 Ksi (Pullout controls...)
9 = 29.607 Ksi (Pullout controls...)
10 = 32.860 Ksi (Pullout controls...)
11 = 30.316 Ksi (Pullout controls...)
12 = 37.000 Ksi (Pullout controls...)
13 = 42.000 Ksi (Yield Stress controls.)

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	1.38	95.0	27.3	21.4	49.3	116.4

> 1.35 OK

Reinf. Stress at Level 1 = 15.994 Ksi (Pullout controls...)
2 = 19.416 Ksi (Pullout controls...)

```
 3 =  22.839 Ksi (Pullout controls...)
 4 =  26.262 Ksi (Pullout controls...)
 5 =  29.684 Ksi (Pullout controls...)
 6 =  33.107 Ksi (Pullout controls...)
 7 =  36.530 Ksi (Pullout controls...)
 8 =  25.233 Ksi (Pullout controls...)
 9 =  28.427 Ksi (Pullout controls...)
10 =  31.622 Ksi (Pullout controls...)
11 =  28.864 Ksi (Pullout controls...)
12 =  34.553 Ksi (Pullout controls...)
13 =  41.151 Ksi (Pullout controls...)

***********************************************************************
*                 For Factor of Safety = 1.0                          *
*         Maximum Average Reinforcement Working Force:                *
*                      17.049 Kips/level                              *
***********************************************************************
```

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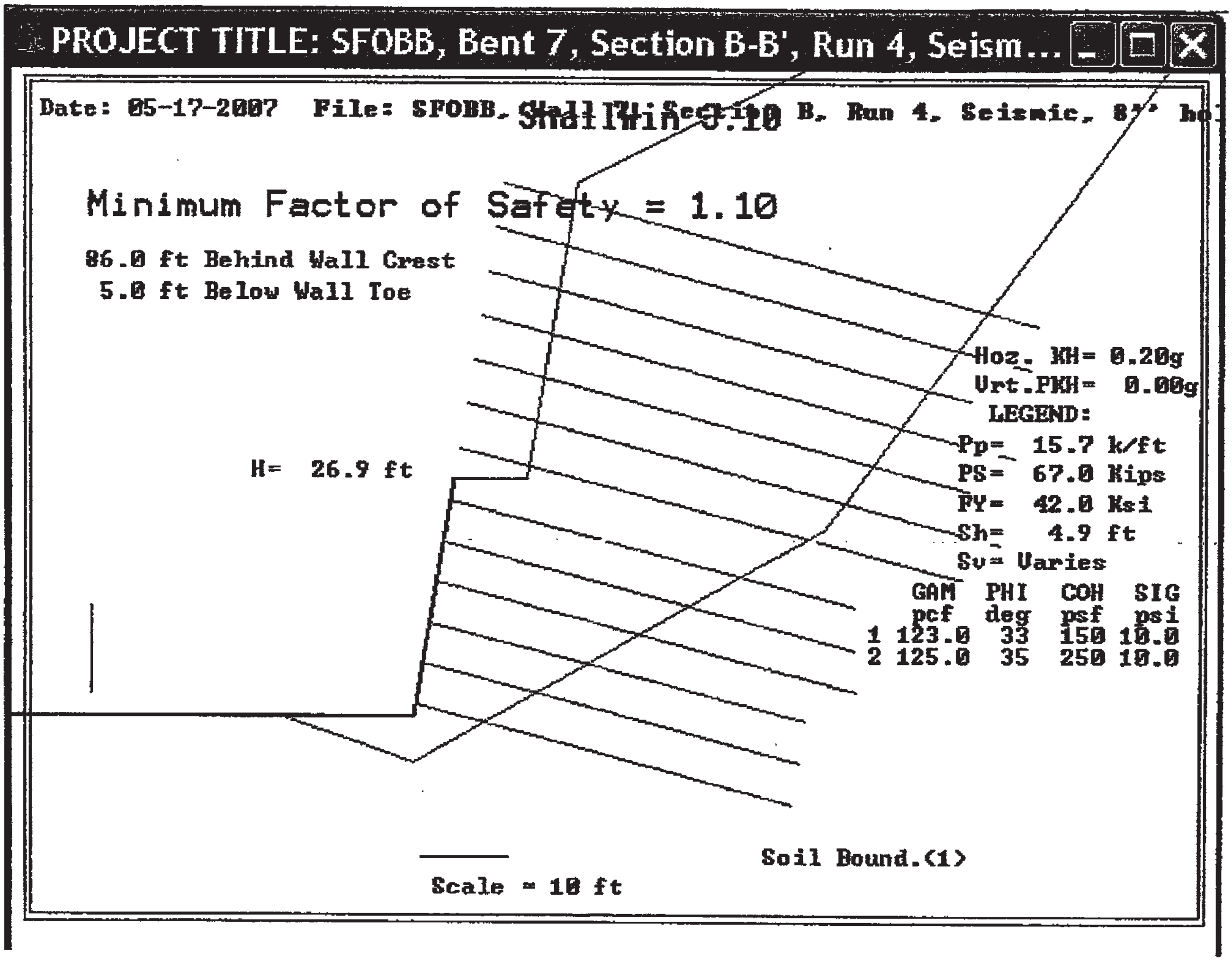
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Date: 05-17-2007 File: SFOBB
Snailwin 3.10
B. Run 4. Seismic
Minimum Factor of Safety = 1.10
86.0 ft Behind Wall Crest
5.0 ft Below Wall Toe
H= 26.9 ft
Hoz. KH= 0.20g
Vrt.PKH= 0.00g
LEGEND:
Pp= 15.7 k/ft
PS= 67.0 Kips
FY= 42.0 Ksi
Sh= 4.9 ft
Sv= Varies
GAM pcf PHI deg COH psf SIG psi
1 123.0 33 150 10.0
2 125.0 35 250 10.0
Soil Bound.(1)
Scale = 10 ft

File: SFOBB, Wall 7, Section B, Run 4, Seismic, 8'' hole
Page - 1

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***************************************************
*    CALIFORNIA DEPARTMENT OF TRANSPORTATION      *
*          ENGINEERING SERVICE CENTER             *
*     DIVISION OF MATERIALS AND FOUNDATIONS       *
*   Office of Roadway Geotechnical Engineering    *
*    Date: 05-17-2007          Time: 12:59:06     *
***************************************************
```

Project Identification - SFOBB, Bent 7, Section B-B', Run 4, Seismic, 8'' hole

--------- WALL GEOMETRY ---------

```
 Vertical Wall Height         =  26.9 ft
 Wall Batter                  =   9.5 degree
                                Angle   Length
                                (Deg)   (Feet)
 First Slope from Wallcrest.  =   0.0      7.9
 Second Slope from 1st slope. =  80.5     33.5
 Third Slope from 2nd slope.  =  26.5     50.0
 Fourth Slope from 3rd slope. =  18.4      0.0
 Fifth Slope from 3rd slope.  =   0.0      0.0
 Sixth Slope from 3rd slope.  =   0.0      0.0
 Seventh Slope Angle.         =   0.0
```

--------- SLOPE BELOW THE WALL ---------

```
 First Slope Angle below Toe.        =   0.0 degrees
 First Slope Distance from Toe.      =   0.0 ft
 Second Slope Angle.                 =   0.0 degrees
 Second Slope Distance from Toe.     =   0.0 ft
 Vertical Depth of Search.           =   5.0 ft
 Number of Searches below wall Toe. =   3
```

--------- SURCHARGE ---------

There is NO SURCHARGE imposed on the system.

--------- OPTION #1 ---------

Factored Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

Soil Layer	Unit Weight (Pcf)	Friction Angle (Degree)	Cohesion Intercept (Psf)	Bond* Stress (Psi)	Coordinates of Boundary XS1 (ft)	YS1 (ft)	XS2 (ft)	YS2 (ft)
1	123.0	33.0	150.0	10.0	0.0	0.0	0.0	0.0
2	125.0	35.0	250.0	10.0	0.0	20.0	100.0	20.0

* Bond Stress also depends on BSF Factor in Option #5 when enabled.

--------- EARTHQUAKE ACCELERATION ---------

Horizontal Earthquake Coefficient = 0.20 (a/g)
Vertical Earthquake Coefficient = 0.00

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 95.0 to 50.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels	= 13
Horizontal Spacing	= 4.9 ft
Yield Stress of Reinforcement	= 42.0 ksi
Diameter of Grouted Hole	= 8.0 in
Punching Shear	= 67.0 kips

---------- (Varying Reinforcement Parameters) ---------

Level	Length (ft)	Inclination (degrees)	Vertical Spacing (ft)	Bar Diameter (in)	Bond Stress Factor
1	61.0	15.0	33.1	1.27	0.67
2	61.0	15.0	4.9	1.27	0.67
3	61.0	15.0	4.9	1.27	0.67
4	61.0	15.0	4.9	1.27	0.67
5	61.0	15.0	4.9	1.27	0.67
6	61.0	15.0	4.9	1.27	0.67
7	61.0	15.0	4.9	1.27	0.67
8	48.0	15.0	6.0	1.27	0.67
9	48.0	15.0	4.6	1.27	0.67
10	48.0	15.0	4.6	1.27	0.67
11	43.0	15.0	4.6	1.27	0.67
12	43.0	15.0	4.6	1.27	0.67
13	43.0	15.0	4.6	1.27	0.67

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
Toe	1.17	90.5	28.6	51.5	51.8	73.2

```
Reinf. Stress at Level  1 =  11.278 Ksi (Pullout controls...)
                        2 =  15.409 Ksi (Pullout controls...)
                        3 =  19.540 Ksi (Pullout controls...)
                        4 =  23.672 Ksi (Pullout controls...)
                        5 =  27.803 Ksi (Pullout controls...)
                        6 =  31.935 Ksi (Pullout controls...)
                        7 =  40.791 Ksi (Pullout controls...)
                        8 =  31.199 Ksi (Pullout controls...)
                        9 =  39.626 Ksi (Pullout controls...)
                       10 =  42.000 Ksi (Yield Stress controls.)
                       11 =  42.000 Ksi (Yield Stress controls.)
                       12 =  42.000 Ksi (Yield Stress controls.)
                       13 =  42.000 Ksi (Yield Stress controls.)
```

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
1.67	1.14	90.5	29.1	51.8	52.4	74.2

```
Reinf. Stress at Level  1 =  10.510 Ksi (Pullout controls...)
                        2 =  14.551 Ksi (Pullout controls...)
                        3 =  18.593 Ksi (Pullout controls...)
                        4 =  22.634 Ksi (Pullout controls...)
                        5 =  26.675 Ksi (Pullout controls...)
                        6 =  30.716 Ksi (Pullout controls...)
                        7 =  38.335 Ksi (Pullout controls...)
                        8 =  28.573 Ksi (Pullout controls...)
                        9 =  36.869 Ksi (Pullout controls...)
                       10 =  42.000 Ksi (Yield Stress controls.)
                       11 =  42.000 Ksi (Yield Stress controls.)
                       12 =  42.000 Ksi (Yield Stress controls.)
                       13 =  42.000 Ksi (Yield Stress controls.)
```

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
3.33	1.11	90.5	32.2	64.2	54.8	62.8

```
Reinf. Stress at Level  1 =   7.234 Ksi (Pullout controls...)
                        2 =  10.891 Ksi (Pullout controls...)
                        3 =  14.548 Ksi (Pullout controls...)
                        4 =  18.204 Ksi (Pullout controls...)
                        5 =  24.704 Ksi (Pullout controls...)
                        6 =  32.749 Ksi (Pullout controls...)
                        7 =  40.794 Ksi (Pullout controls...)
                        8 =  30.001 Ksi (Pullout controls...)
                        9 =  37.510 Ksi (Pullout controls...)
                       10 =  42.000 Ksi (Yield Stress controls.)
                       11 =  42.000 Ksi (Yield Stress controls.)
                       12 =  42.000 Ksi (Yield Stress controls.)
                       13 =  42.000 Ksi (Yield Stress controls.)
```

O.K.

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	1.10	90.5	30.1	52.3	53.5	76.0

```
Reinf. Stress at Level  1 =   9.049 Ksi (Pullout controls...)
                        2 =  12.919 Ksi (Pullout controls...)
```

```
     3 =  16.788 Ksi (Pullout controls...)
     4 =  20.657 Ksi (Pullout controls...)
     5 =  24.527 Ksi (Pullout controls...)
     6 =  28.396 Ksi (Pullout controls...)
     7 =  33.614 Ksi (Pullout controls...)
     8 =  23.526 Ksi (Pullout controls...)
     9 =  31.573 Ksi (Pullout controls...)
    10 =  39.620 Ksi (Pullout controls...)
    11 =  39.738 Ksi (Pullout controls...)
    12 =  42.000 Ksi (Yield Stress controls.)
    13 =  42.000 Ksi (Yield Stress controls.)

*******************************************************************
*               For Factor of Safety = 1.0                        *
*        Maximum Average Reinforcement Working Force:             *
*                   35.777 Kips/level                             *
*******************************************************************
```

SNAIL OUTPUT INTERPRETATION
SFOBB, WALL AT TRANSITION BENT W7
5-22-07 MK/FUGRO-EMI

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)

SNAIL NAIL STRESS OUTPUT TABLE FOR STATIC:

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	1.38	95.0	27.3	21.4	49.3	116.4

```
Reinf. Stress at Level  1 = 15.994 Ksi (Pullout controls...)
                        2 = 19.416 Ksi (Pullout controls...)
                        3 = 22.839 Ksi (Pullout controls...)
                        4 = 26.262 Ksi (Pullout controls...)
                        5 = 29.684 Ksi (Pullout controls...)
                        6 = 33.107 Ksi (Pullout controls...)
                        7 = 36.530 Ksi (Pullout controls...)
                        8 = 25.233 Ksi (Pullout controls...)
                        9 = 28.427 Ksi (Pullout controls...)
                       10 = 31.622 Ksi (Pullout controls...)
                       11 = 28.864 Ksi (Pullout controls...)
                       12 = 34.553 Ksi (Pullout controls...)
                       13 = 41.151 Ksi (Pullout controls...)
```

SNAIL NAIL STRESS OUTPUT TABLE FOR SEISMIC:

DEPTH BELOW WALL TOE (ft)	MINIMUM SAFETY FACTOR	DISTANCE BEHIND WALL TOE (ft)	LOWER FAILURE PLANE ANGLE (deg)	LOWER FAILURE PLANE LENGTH (ft)	UPPER FAILURE PLANE ANGLE (deg)	UPPER FAILURE PLANE LENGTH (ft)
5.00	1.10	90.5	30.1	52.3	53.5	76.0

```
Reinf. Stress at Level  1 =  9.049 Ksi (Pullout controls...)
                        2 = 12.919 Ksi (Pullout controls...)
                        3 = 16.788 Ksi (Pullout controls...)
                        4 = 20.657 Ksi (Pullout controls...)
                        5 = 24.527 Ksi (Pullout controls...)
                        6 = 28.396 Ksi (Pullout controls...)
                        7 = 33.614 Ksi (Pullout controls...)
                        8 = 23.526 Ksi (Pullout controls...)
                        9 = 31.573 Ksi (Pullout controls...)
                       10 = 39.620 Ksi (Pullout controls...)
                       11 = 39.738 Ksi (Pullout controls...)
                       12 = 42.000 Ksi (Yield Stress controls.)
                       13 = 42.000 Ksi (Yield Stress controls.)
```

SNAIL FORCE OUTPUT TABLE FOR STATIC:

```
*****************************************************************
*               For Factor of Safety = 1.0
*        Maximum Average Reinforcement Working Force:
*                    17.049 Kips/level
*****************************************************************
```

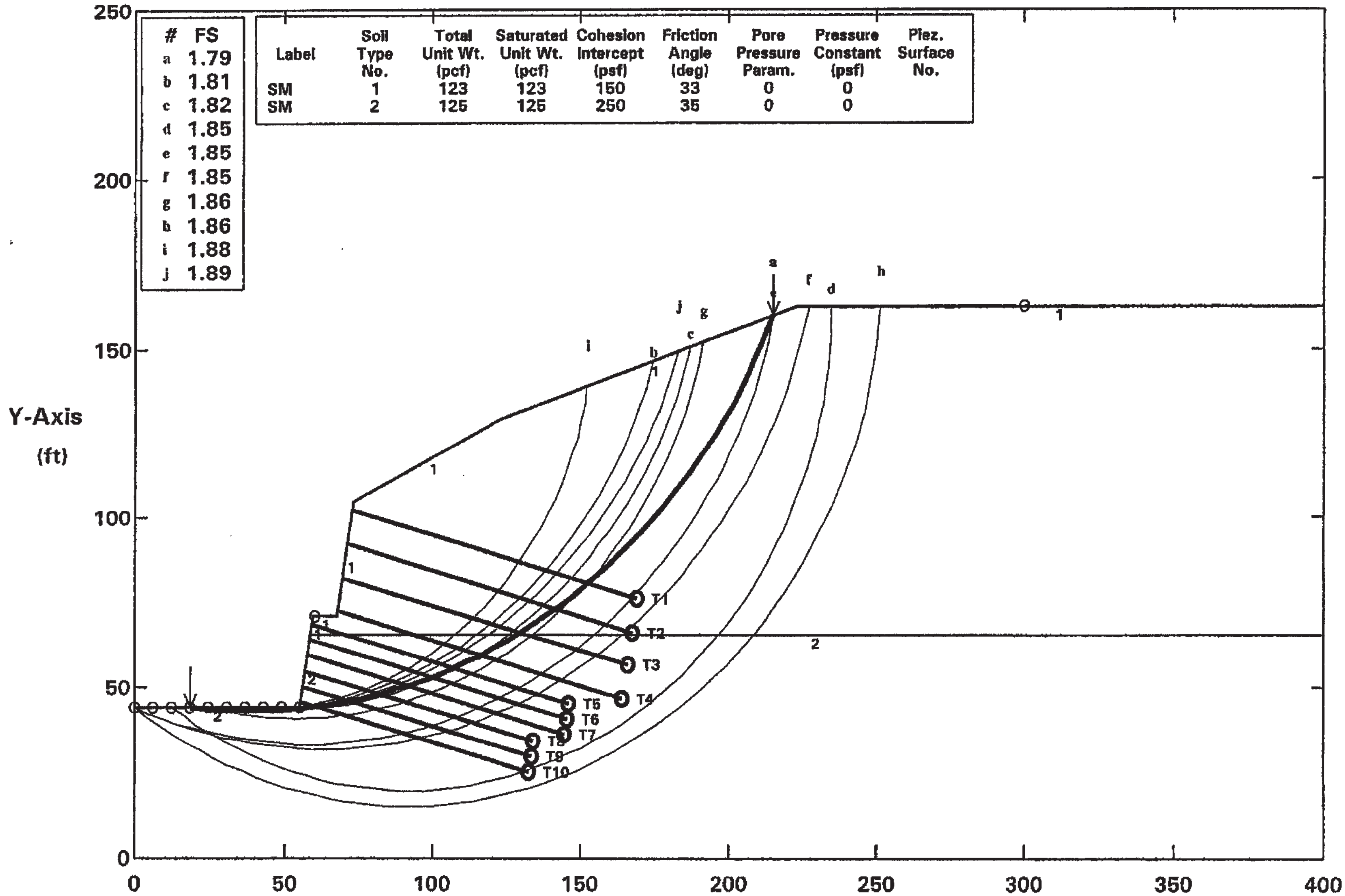
SNAIL FORCE OUTPUT TABLE FOR SEISMIC:

```
*****************************************************************
*               For Factor of Safety = 1.0
*        Maximum Average Reinforcement Working Force:
*                    35.777 Kips/level
*****************************************************************
```

Design Wall Cross Section	B1		
Nail cross sectional area =	1.272 in2		
Nail yield stress, fy =	75 ksi		
Applied/Required FOS for yield stress, FST =	1.8		
Nail yield stress, allowable =	42 ksi		
	Static	**Seismic**	**Max**
Applied/Required FOS for Pull-Out, FSP =	2.00	1.50	NA
Required FOS for Global Stability =	1.35	1.10	NA
Critical FOS for Global Stability, FSG =	1.38	1.10	NA
Nail stress LE1 (ksi)	15.90	9.00	15.90
Nail stress LE2	19.42	12.92	19.42
Nail stress LE3	22.84	16.79	22.84
Nail stress LE4	26.26	20.66	26.26
Nail stress LE5	29.68	24.53	29.68
Nail stress LE6	33.11	28.40	33.11
Nail stress LE7	36.53	33.61	36.53
Nail stress LE8	25.23	23.53	25.23
Nail stress LE9	28.43	31.57	31.57
Nail stress LE10	31.62	39.62	39.62
Nail stress LE11	28.86	39.74	39.74
Nail stress LE12	34.55	42.00	42.00
Nail stress LE13 (ksi)	41.15	42.00	42.00
Average nail stress (ksi) at crit. FSG	28.74	28.03	31.07
Maximum nail stress (ksi) at crit. FSG	41.15	42.00	42.00
Average nail force Tave (K) at crit. FSG	36.6	35.7	NA
Average nail force Tave,s (K) at FSG=1	17.0	35.8	NA
Maximum nail force Tmax (K) at crit. FSG	52.3	53.4	NA
Maximum nail force Tmax,s (K) at FSG=FST=1 per FHWA Circ. 7, Eq. 6.13	24.4	53.6	53.6
Required steel area per FHWA Circ 7, Eq. 6.14 (in2)	NA	NA	1.276

SFOBB, WALL 7, SECTION B WITH SOIL NAILS, STATIC

Ten Most Critical. D:SNAIL-S.PLT By: LY 05-16-07 10:10am



#	FS
a	1.79
b	1.81
c	1.82
d	1.85
e	1.85
f	1.85
g	1.86
h	1.86
i	1.88
j	1.89

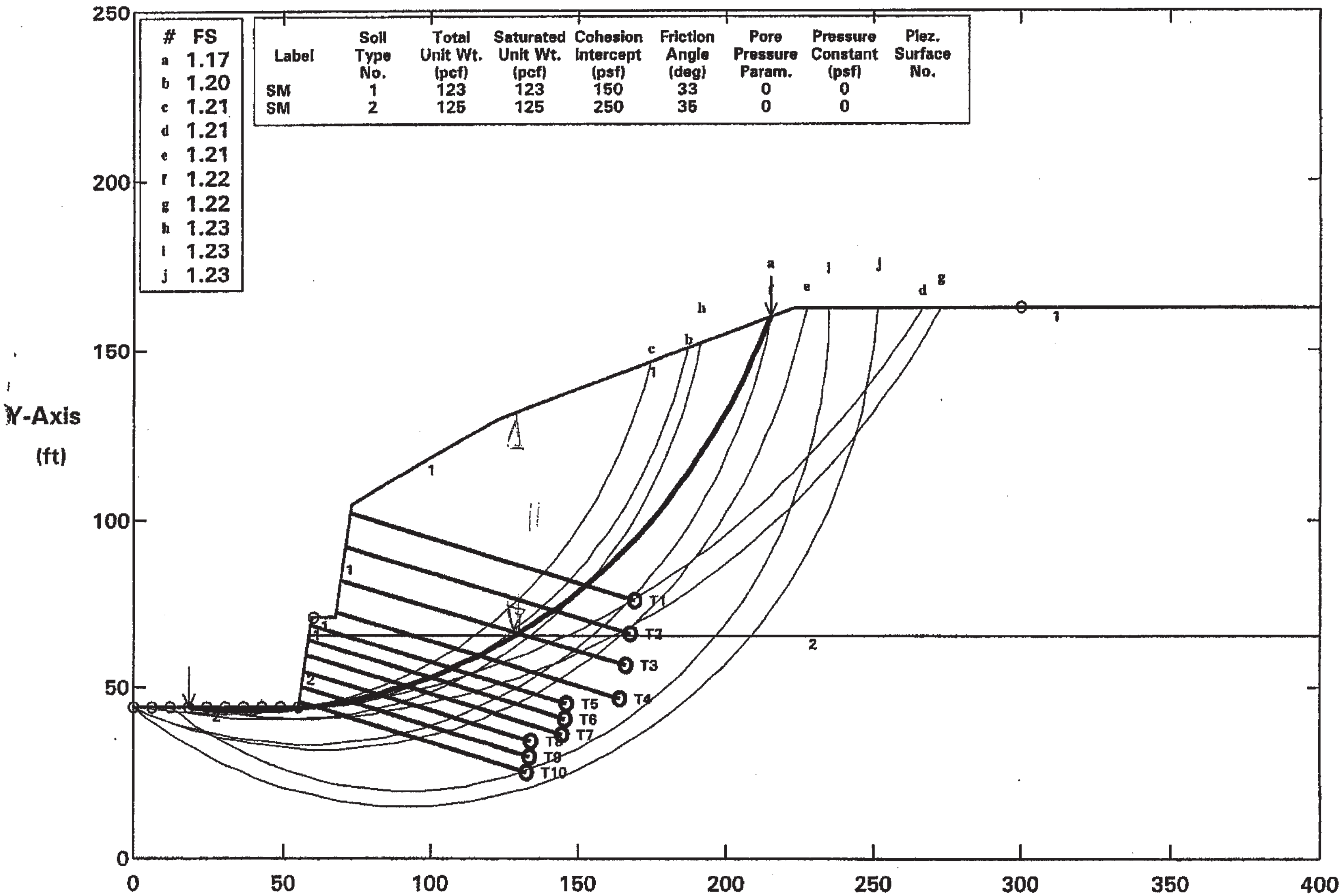
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SM	1	123	123	150	33	0	0	
SM	2	125	125	250	35	0	0	

PCSTABL5M FSmin = 1.79 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

SFOBB, WALL 7, SECTION B WITH SOIL NAILS, PSEUDO-STATIC, Kh = 0.2

Ten Most Critical. D:SNAIL-E.PLT By: LY 05-16-07 10:09am



PCSTABL5M FSmin = 1.17 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

Example Embankment Slope Problem

Introduction

The following example demonstrates the application of the proposed procedure outlined in Section Y of the proposed Specifications for the seismic design of embankment slopes. The example considers embankments constructed at two slope angles: 2:1 and 1.5:1. These slopes are very typical of constructed embankment slopes. The maximum steepness of 1.5:1 is about as steep as can be maintained or that can be placed without special slope erosion protection.

The following subsections summarize (1) the slope geometry and soil properties used in the example, (2) the seismicity for the three sites considered, (3) the general methodology followed, (4) the results of the stability analyses, and (5) preliminary conclusions made from these analyses. Information from these analyses is used to develop a step-by-step presentation of the example for one of the cases (Appendix A). Results from these analyses are also used to confirm screening levels below which seismic analysis will not be required.

Slope Geometry and Soil Properties

Two slope angles (2:1 and 1.5:1) and three slope heights (15, 30, and 45 feet) were examined in this study. The geometry of the slope is shown on Figure 1.

The foundation soil below the embankment fill comprises cohesive soil with a friction angle of 10° and cohesion of 4,000 psf; therefore, deep failure planes through the foundation material were not a design consideration for this study. A firm-ground condition was assumed for the embankment base to avoid additional complexity from base failures that might be associated with liquefaction or soft ground conditions. It was assumed that some type of ground improvement would have to occur before slopes of this height were constructed on either liquefiable soils or soft soils, resulting in conditions consistent with the example.

Three different fill materials were considered in this study. The properties for these fills are as follows:

- Fill 1: ϕ = 33° and c = 500 psf (silty sand borrow)
- Fill 2: ϕ = 23° and c = 2,000 psf (clayey sand borrow)
- Fill 3: ϕ = 40° and c = 50 psf (gravel borrow)

The above fill properties represent a range of conditions that can be expected in different areas of the country. In areas with wet climates, the fill would be primarily cohesionless and exhibit low cohesion values. Areas with drier climates often will use soils with higher fines content and a higher cohesion value. A specific effort has been made to include representative values of cohesion even for Fill 3. The effects of the cohesion component are significant during seismic

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loading, and are part of the reason that retaining walls and slopes have performed so well during previous seismic events.

Seismicity

Three sites with different levels of seismic activity were included in this study. Two of the sites are located in the Western United States (WUS), one in Los Angeles area and the other one in Seattle. The third site is located in Central and Eastern United State region (CEUS), in Charleston, South Carolina.

Peak Ground Accelerations (PGA) for each site were determined from USGS/AASHTO Seismic Design Parameters for 2006 AASHTO Seismic Guidelines. Seismic accelerations were calculated for an average return period of 1,000 years. PGA values were initially determined for bedrock (Soil Type B) and modified for the foundation soil, assumed Soil Type D. A summary of site locations and seismicity data is given in Table 1.

Methodology

The methodology followed that outlined in Section Y of the proposed Specifications. The computer program SLIDE (Rocscience, 2007) was utilized for this study. Only circular failure planes were examined. Spencer's slope stability analysis method was used to calculate factors of safety. A detailed discussion of the assumptions in Spencer's method can be found in Abramson et al. (2001).

Only failure planes through the fill were examined; the potential for deeper failure planes through the foundation material was not evaluated. The PGA was adjusted for slope-height effects when the maximum depth of the failure plane below the ground surface was greater than 20 feet, following the procedure recommended in the proposed Specifications. As critical failure surfaces were tangential to the foundation, the height factor was taken as the slope height H.

Based on these recommendations, the seismic acceleration was adjusted using the following equation:

$$k_{av} = \alpha\, k_{max}$$

For site category D, α is calculated from the following equation:

$$\alpha = 1 + 0.01H\,[(0.5\beta) - 1]$$

where H is the slope height in feet and β is calculated from the equation:

$$\beta = F_v S_1 / k_{max}$$

where $F_v S_1$ spectral acceleration coefficient at a period at one second adjusted for site conditions.

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The resulting seismic coefficient ($k_{av}/2$) was used in pseudo-static seismic slope stability analyses as stated in the proposed Specifications. The values of k_{av} used for the analyses are shown in Table 2.

Newmark displacement correlations in Section X.4.5 of the proposed Specifications were used to estimate the slope movement during seismic loading for those cases where the Capacity to Demand (C/D) ratio (i.e., factor of safety) was less than 1.0. Newmark deformation was estimated from the following equation:

$$\log(d) = -1.51 - 0.74 \log(k_y/k_{max}) + 3.27 \log(1 - k_y/k_{max}) - 0.8 \log(k_{max}) + 1.50 \log PGV) \quad (1)$$

where PGV was estimated from the following equation:

$$PGV = 55\ F_v\ S_1$$

The yield acceleration (k_y) for each case was calculated using the SLIDE program. The yield acceleration is the seismic acceleration that results in a C/D ratio of 1.0 (factor of safety of 1.0). For these analyses k_{av} was used in place of k_{max} in the displacement calculation.

Results of Analyses

The C/D ratios for each analysis are reported in Table 3. Yield acceleration values are shown in Table 4. Permanent displacements were only expected for cases where k_{av} is larger than yield acceleration (k_y).

For these analyses the full k_{av} was applied. As discussed in the proposed Specifications, the design approach involves using half the k_{av} value and confirming that the FS > 1.0. This approach assumes that some amount of deformation (1 to 2 inch) is acceptable.

As shown in Table 3, the C/D ratios are less than 1.0 only for the steepest slopes and for the higher values of PGA. Newmark displacement estimates were made for this case (Fill Type 3). Results of these analyses are shown in Table 5.

Concluding Comments

These results show that embankments constructed at slopes varying from 2:1 to 1.5:1 using common types of embankment fill will perform very well during ground motions that might be encountered in seismically active areas of southern California, the Pacific Northwest, and southeastern United States. A conclusion that can be reached from these results is that the screening level at which seismic analyses are required will be relatively high for engineered slopes – say as high as 0.6g as long as some permanent slope displacement is acceptable.

From the above analyses, it also appears that the more critical types of slopes for seismic loading will be in locations where liquefiable soils exist or in natural slopes where very low strength

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bedding planes occur. Evaluation of these two conditions is very dependent on site-specific conditions.

References

Abramson, L.W., Lee, T.S., Sharma, S. and Boyce, G.M. (2001). *Slope Stability and Stabilization Methods*, 2nd Edition, John Wiley & Sons.

RocScience (2007). "SLIDE: Stability Analysis for Soil and Rock Slopes." www.rocscience.com.

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Table 1. Site Coordinates and Seismicity Data

Site Coordinates		Region	Soil Type B (Bedrock)		Soil Type D	
Longitude	Latitude		PGA	S_1[1]	$F_{pga}PGA$	F_vS_1[1]
-117.9750	34.0500	WUS (Los Angeles)	0.60	0.52	0.60	0.78
-122.2500	47.2700	WUS (Seattle)	0.40	0.30	0.46	0.54
-079.2370	33.1000	CEUS (Charleston)	0.20	0.10	0.30	0.24

1. Spectral acceleration coefficient at 1.0 second period.

Table 2. . Height-Adjusted Seismic Accelerations for Site Class D

Longitude	Latitude	Region	$F_{pga}PGA$	F_vS_1	β	H [ft]	α	k_{av}
-117.9750	34.0500	WUS (Los Angeles)	0.60	0.78	1.30	15	0.95	0.569
						30	0.90	0.537
						45	0.84	0.506
-122.2500	47.2700	WUS (Seattle)	0.46	0.54	1.16	15	0.94	0.431
						30	0.87	0.402
						45	0.81	0.373
-079.2370	33.1000	CEUS (Charleston)	0.30	0.24	0.80	15	0.91	0.271
						30	0.82	0.244
						45	0.73	0.217

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Table 3. Capacity to Demand Ratio (C/D or Factor of Safety) for Slope Stability Analyses[1]

Slope Angles [H:V]	Slope Height [ft]	C/D Ratio (Spencer)		
Site 1: Soil Type D (k_{max} = 0.6)				
		c = 500 psf / ϕ = 33°	c = 2000 psf / ϕ = 23°	c = 50 psf / ϕ = 40°
2:1	15	2.40	5.59	1.28
	30	1.73	3.31	1.17
	45	1.53	2.55	1.16
1.5:1	15	2.19	5.28	1.09
	30	1.58	3.09	0.98
	45	1.36	2.44	0.94
Site 2: Soil Type D (k_{max} = 0.46)				
		c = 500 psf / ϕ = 33°	c = 2000 psf / ϕ = 23°	c = 50 psf / ϕ = 40°
2:1	15	2.68	6.39	1.43
	30	1.94	3.76	1.32
	45	1.72	2.89	1.31
1.5:1	15	2.43	5.96	1.21
	30	1.75	3.48	1.10
	45	1.50	2.72	1.05
Site 3: Soil Type D (k_{max} = 0.298)				
		c = 500 psf / ϕ = 33°	c = 2000 psf / ϕ = 23°	c = 50 psf / ϕ = 40°
2:1	15	3.10	7.55	1.66
	30	2.26	4.43	1.54
	45	1.99	3.39	1.52
1.5:1	15	2.78	6.91	1.38
	30	1.97	4.01	1.26
	45	1.71	3.14	1.21

1. Factors of Safety calculated using 0.5 k_{max} from Table 2.

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Table 4. Yield Acceleration (k_y)

Slope Angles [H:V]	Slope Height [ft]	Yield Acceleration		
		c = 500 psf / ϕ = 33°	c = 2000 psf / ϕ = 23°	c = 50 psf / ϕ = 40°
2:1	15	> 0.6	> 0.6	0.44
	30	> 0.6	> 0.6	0.37
	45	0.55	> 0.6	0.34
1.5:1	15	> 0.6	> 0.6	0.34
	30	> 0.6	> 0.6	0.26
	45	0.48	> 0.6	0.22

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Table 5. Newmark Displacements

Slope Angles [H:V]	Slope Height [ft]	Newmark Displacement [in]		
Site 1: Soil Type D (k_{max} = 0.60)				
		c = 500 psf / ϕ = 33°	c = 2000 psf / ϕ = 23°	c = 50 psf / ϕ = 40°
2:1	15	0	0	<1
	30	0	0	<1
	45	0	0	<1
1.5:1	15	0	0	1-2
	30	0	0	4
	45	0.0	0	6
Site 2: Soil Type D (k_{max} = 0.46)				
Slope Angles [H:V]	Slope Height [ft]	Newmark Displacement [in]		
		c = 500 psf / ϕ = 33°	c = 2000 psf / ϕ = 23°	c = 50 psf / ϕ = 40°
2:1	15	0	0	0
	30	0	0	0.0
	45	0	0	0.0
1.5:1	15	0	0	<1
	30	0	0	1
	45	0	0	1
Site 3: Soil Type D (k_{max} = 0.298)				
Slope Angles [H:V]	Slope Height [ft]	Newmark Displacement [in]		
		c = 500 psf / ϕ = 33°	c = 2000 psf / ϕ = 23°	c = 50 psf / ϕ = 40°
2:1	15	0	0	0
	30	0	0	0
	45	0	0	0
1.5:1	15	0	0	0
	30	0	0	0
	45	0	0	0

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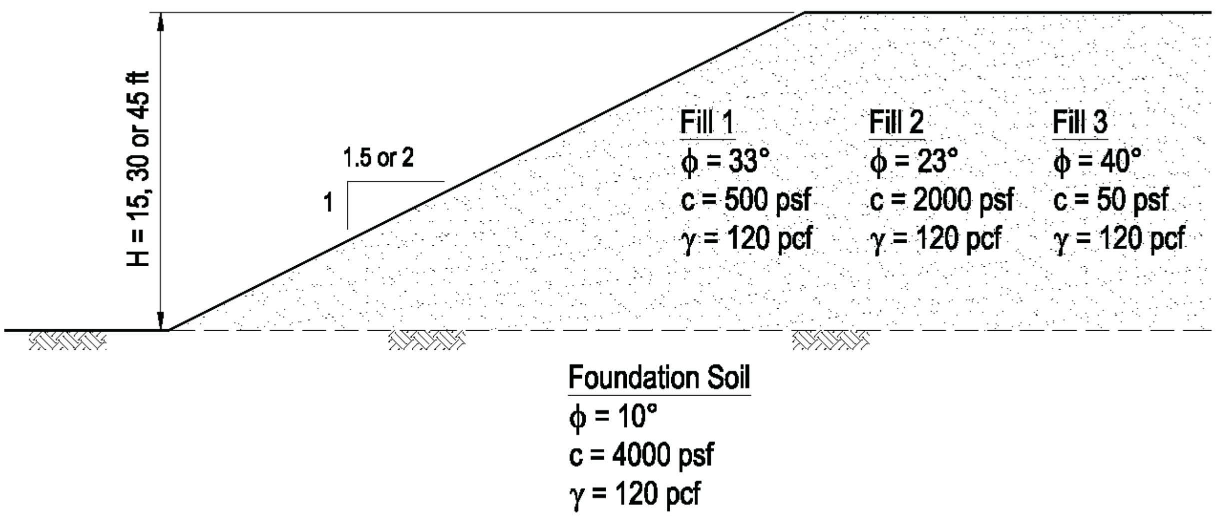

Figure 1. Slope Geometry

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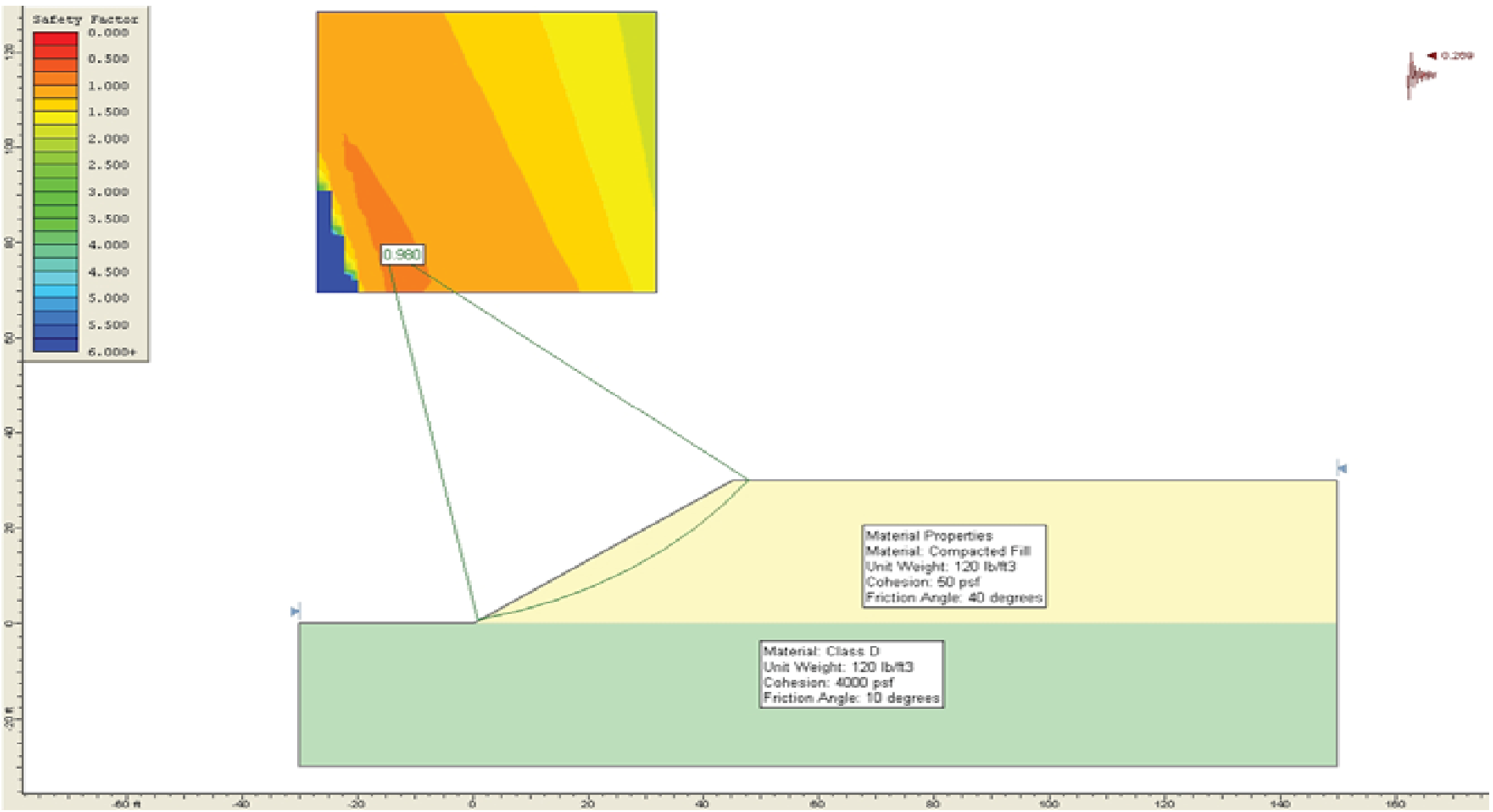
Appendix A: Screen Shots of Slope Stability Analyses for 30-foot High 1.5:1 Embankment on Site 1 (k_{max} =0.6) with c=50 psf and ϕ = 40° Backfill



Calculating C/D Ratio (Factor of Safety) using Slide Program and Spencer's Slope Stability Method

CVO\081750022

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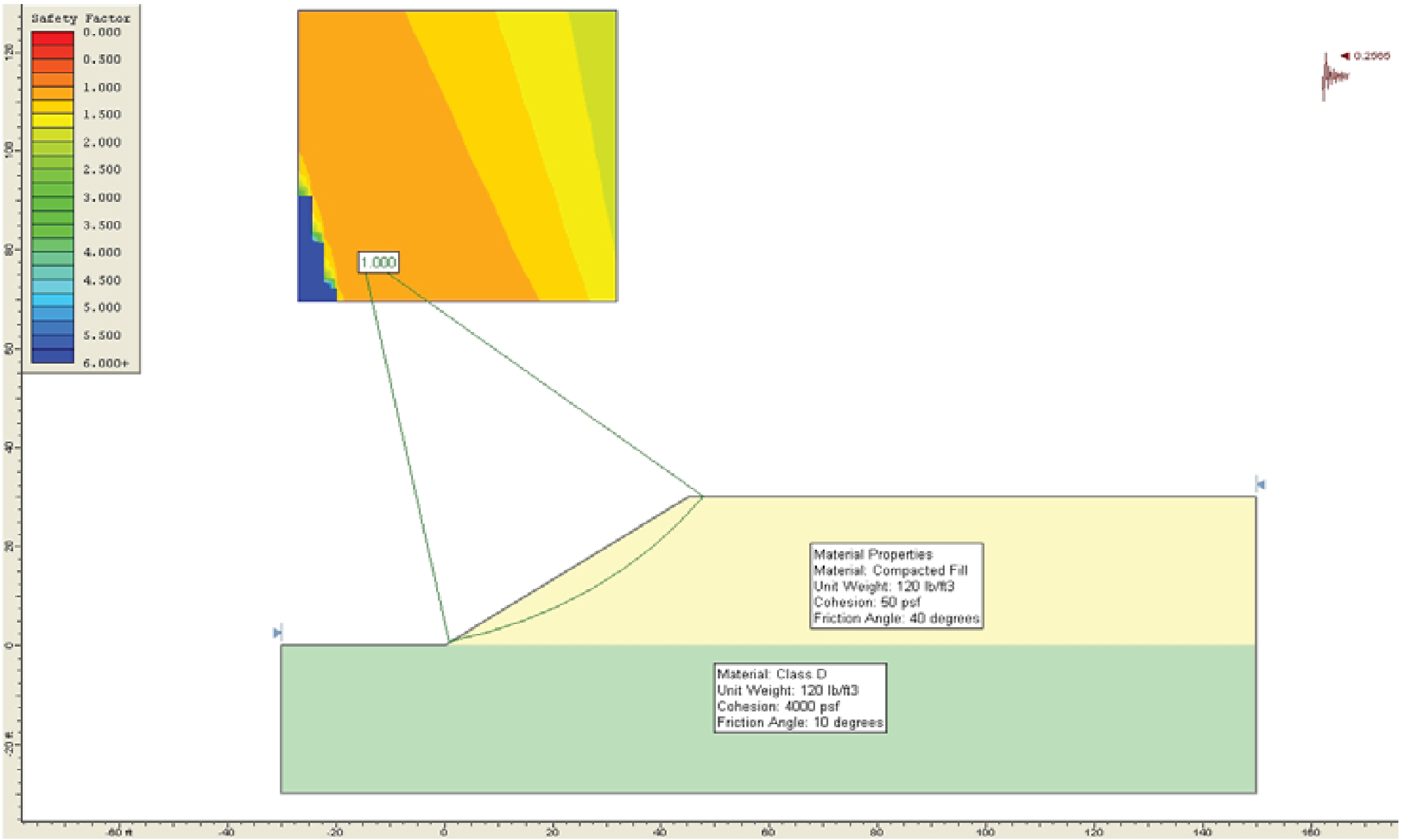

Calculating Yield Acceleration (k_y) using Slide Program and Spencer's Slope Stability Method

CVO\081750022

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Example Cut Slope Problem

Introduction

This example describes the evaluation of seismic slope stability of a natural (cut) slope following procedures given in the proposed Specifications. These procedures include the calculation of (1) a height-dependent wave-scattering factor, referred to as the α factor, and (2) the permanent displacement of a typical slope using the updated Newmark displacement relationship.

Problem Description

The slope stability problem was taken from an on-going project located in a suburb of Seattle, Washington. The project involves widening of an existing 2-lane roadway to accommodate projected traffic increases. The widening will be accomplished by cutting into an existing slope.

A steep slope exists on the downhill side of the existing roadway. The slope ranges in steepness from 2H to 1V (horizontal to vertical) to as steep as 1H to 1V in some locations. The "over-steepened" slope conditions are the result of past glacial loading in the Puget Sound region. Geologists believe that up to 4,000 feet of ice once occurred in this area. This led to a heavily overconsolidated granular till along the roadway alignment. Approximately 15 feet of granular fill were placed over the fill during the original construction of the roadway.

The owner of the roadway has expressed concern about the seismic performance of the "oversteepened" slope on the downside of the roadway during a design earthquake. Normally, an existing slope such as this would not be evaluated for seismic performance since the widening of the road will occur on the up-hill side of the roadway, primarily because of the environmental consequences of working on a steep hillside above a stream. However, the roadway will be heavily traveled, and therefore the owner would like to know the potential risk to the roadway users. If necessary, the slope could be flattened or some other ground improvement method could be used to improve stability.

Analyses were conducted to evaluate the performance of the slope during seismic loading. In order to evaluate risk, a range of earthquake sizes were considered. Performance was evaluated by estimating permanent ground displacements at the edge of the travel lane for the new roadway, which was located approximately 10 feet from the crest of the slope.

This stability problem was made somewhat more complicated by the uncertain strength properties of the till. The till was primarily silty sands with gravels. Blowcounts from Standard Penetration Tests (SPTs) were generally very high indicating a dense condition. With some slopes standing as steep as 1H to 1V, the friction angle of the granular soil could be as high as 45 degrees. However, common practice in the area is to assign this till either of two strengths: (1) ϕ = 42 degrees and c = 0 psf, or (2) ϕ = 38 degrees and c = 200 psf. A lower bound till strength of ϕ = 36 degrees and c = 0 psf has also been used on some occasions.

Another complicating factor is that a perched water table was found on top of the till in the roadway fill during the field explorations. Piezometer measurements suggest that this perched ground water level does not change much during the drier seasons. A second ground water table exists at the bottom of the slope.

Geometry, Soil Properties, and Earthquake Parameters

Seismic stability of the natural slopes was evaluated for the following conditions:

- Slope angles ranging from 2H to 1V up to 1H to 1V
- Soils comprised of glacial till and fill. Till is a dense silty sand with gravel. Standard Penetration Test (SPT) blowcounts range from 30 blows per foot to refusal. Soil strength values were interpreted from SPT blowcounts. A 15-foot layer of cohesionless fill is located above till.
- Groundwater perched on top of till and at depth within till
- PGA and S_1 for site are estimated from the USGS website to range from 0.37g to 0.41g and from 0.46g to 0.56g, respectively, for the 1,000-year earthquake. The soil conditions are representative of Site Class C to D.[1]

Since the owner is also interested in the risk to the roadway facility, stability was also evaluated for a 10 percent probability of exceedance in 50 years (475-year event) and for a 2 percent probability of exceedance (2,475-year event).

Analysis Method

The computer program SLIDE was used to perform the stability analyses. Factors of safety were first obtained for gravity loading to the slope. This step was then followed by pseudo static analyses where the yield accelerations for the various slope conditions (i.e., different slope angles and different till properties) were obtained. The yield acceleration is defined as the seismic coefficient that results in a factor of safety of 1.0. The Spencer method was used during the SLIDE analyses.

No adjustment in strength properties were made for cyclic loading effects. It was felt that the granular nature of these soils in combination with the overall density would not lead to degradation of soil strengths during seismic loading. Consideration was also given to including additional cohesion for the two cases where the cohesion of the soil was assumed to be zero, as suggested in the proposed Specifications. However, for this set of examples, the cohesion was defined as zero to be conservative.

Results of Evaluations

The following two subsections summarize the results of the earthquake and Newmark displacement analyses.

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[1] This example problem was developed before the AASHTO seismic hazard maps were available. However, it was expected that AASHTO would adopt a 1,000-year return period. In the absence of the AASHTO hazards maps, the USGS interactive hazards website was used to determine ground motion parameters for this example.

Earthquake Parameters

Figures 1 through 9 show deaggregation plots for PGA, S_s, and S_1, at the three earthquake return periods. Table 1 summarizes various parameters developed from these plots. Values for F_a and F_v were obtained from table given in Article X.4 of Section X to the proposed Specifications. PGV and the α factor were calculated from the equations in Article X.4.

Table 1. Ground Motions for Example Problem

Parameter	Units	Site Class	Ground Motion Parameter 7% in 75 Years	10% in 50 Years	2% in 50 Years
PGA		B	0.41	0.31	0.58
S_s		B	0.92	0.68	1.30
S_1		B	0.30	0.22	0.44
$S_s/2.5$			0.37	0.27	0.52
Magnitude			6.8	6.8	6.8
F_{pga}		C	1.00	1.10	1.00
		D	1.10	1.20	1.00
F_v		C	1.50	1.58	1.36
		D	1.80	1.96	1.56
PGV	In/sec	C	25	19	33
	In/sec	D	30	24	38
$\beta = F_v S_1 / k_{max}$		C	1.10	1.02	1.03
		D	1.20	1.16	1.18
Failure Slope Height	ft		15	15	15
α Factor per Equation 7-2		C	0.93	0.93	0.93
		D	0.94	0.94	0.94
$k_{av} = \alpha k_{max}$		C	0.38	0.32	0.54
		D	0.42	0.35	0.54

Note: 5% in 50 years is approximately equivalent to the AASHTO hazard level of 7% in 75 years

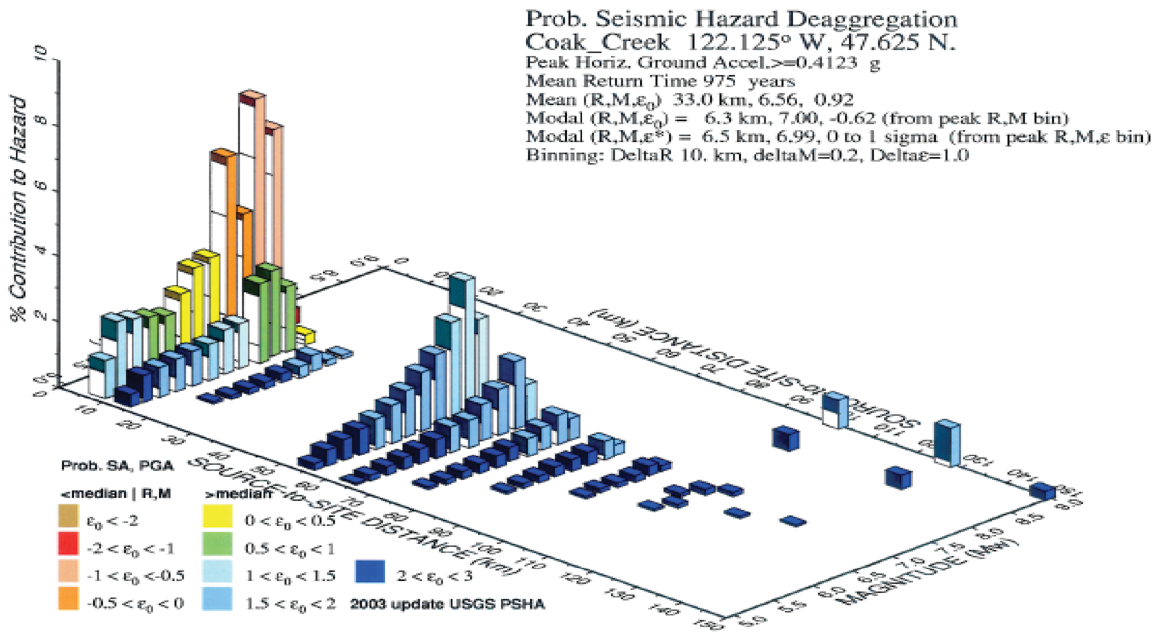

Figure 1. Deaggregation Results for PGA at 975-Year Return Period

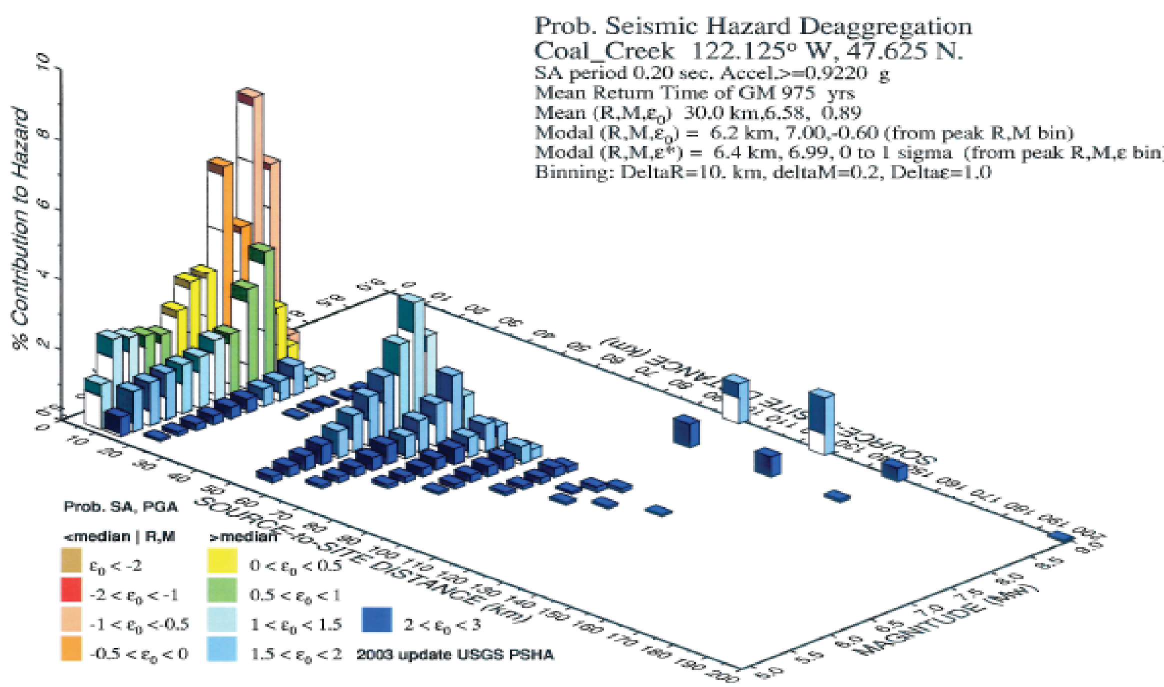

Figure 2. Deaggregation Results for S_s at 975-Year Return Period

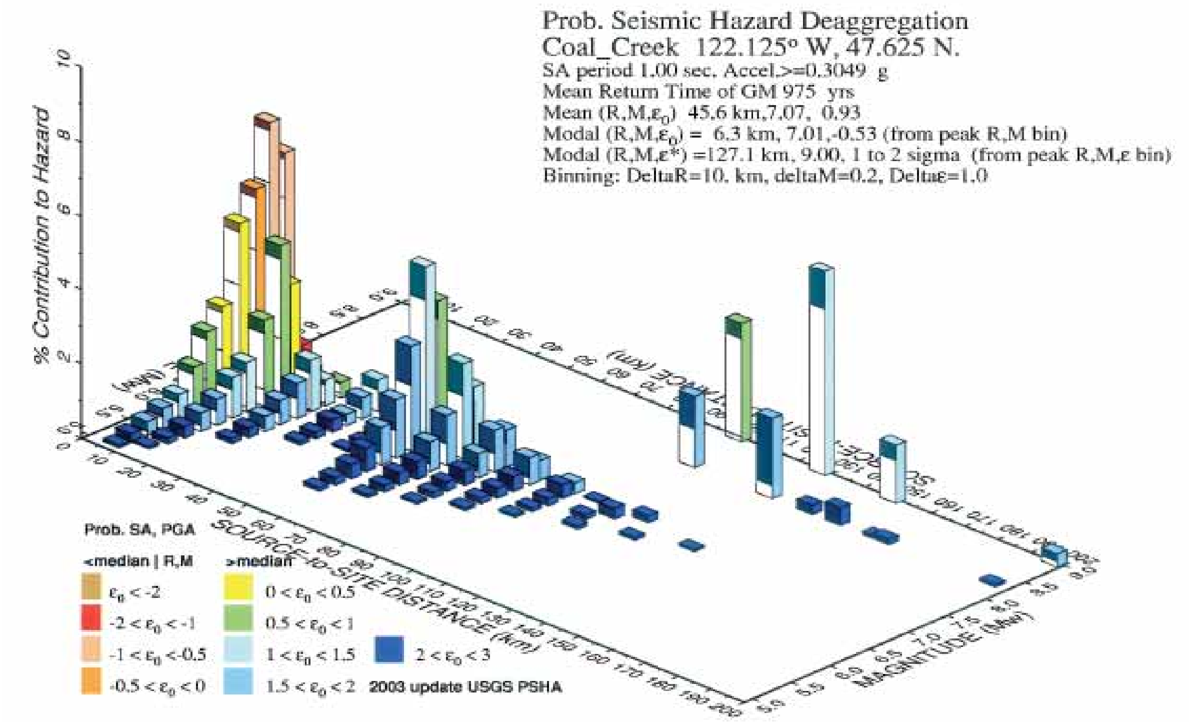

Figure 3. Deaggregation Results for S_1 at 975-Year Return Period

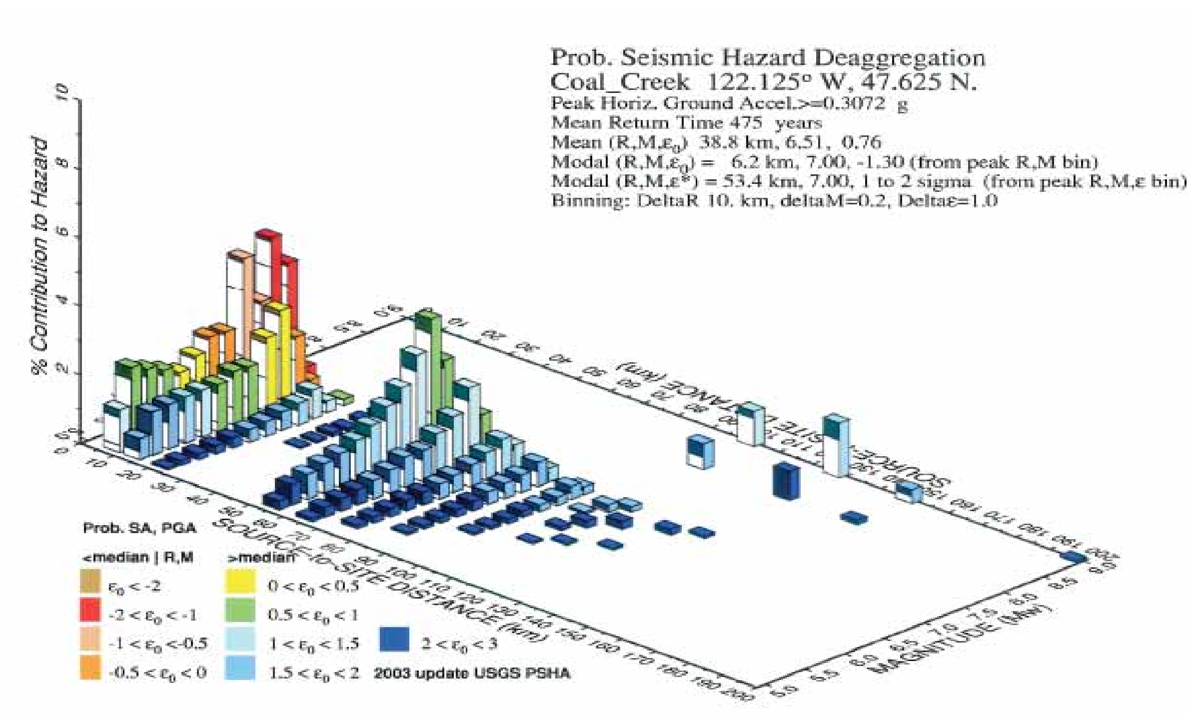

Figure 4. Deaggregation Results for PGA at 475-Year Return Period

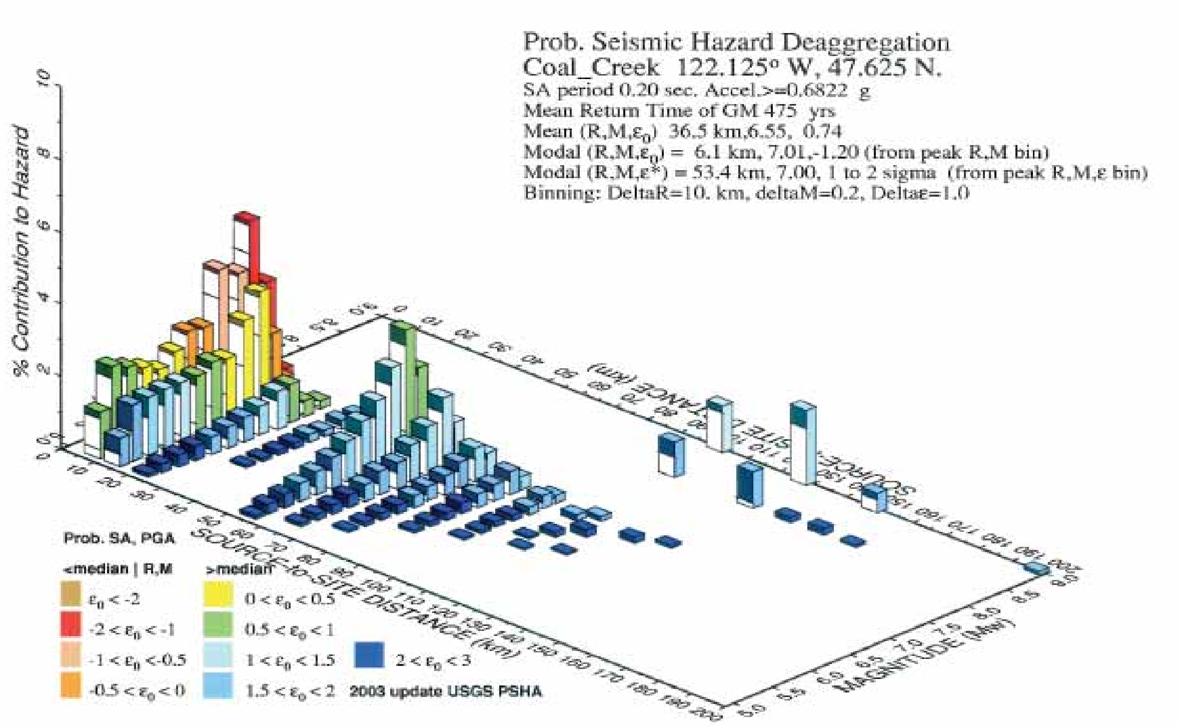

Figure 5. Deaggregation Results for S_s at 475-Year Return Period

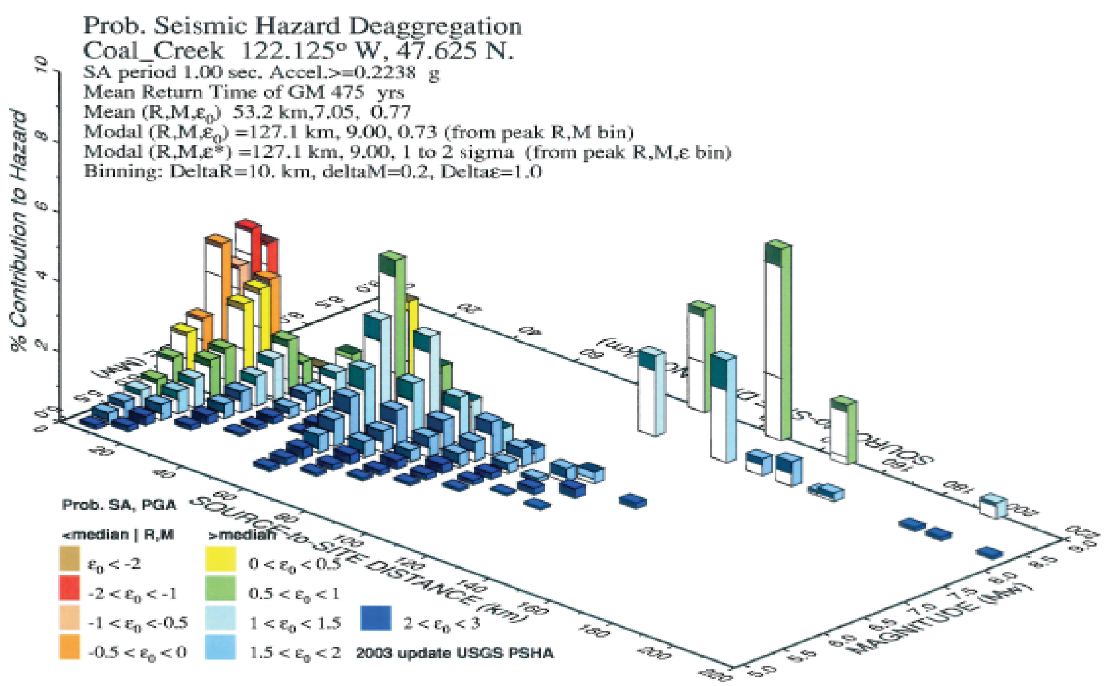

Figure 6. Deaggregation Results for S_1 at 475-Year Return Period

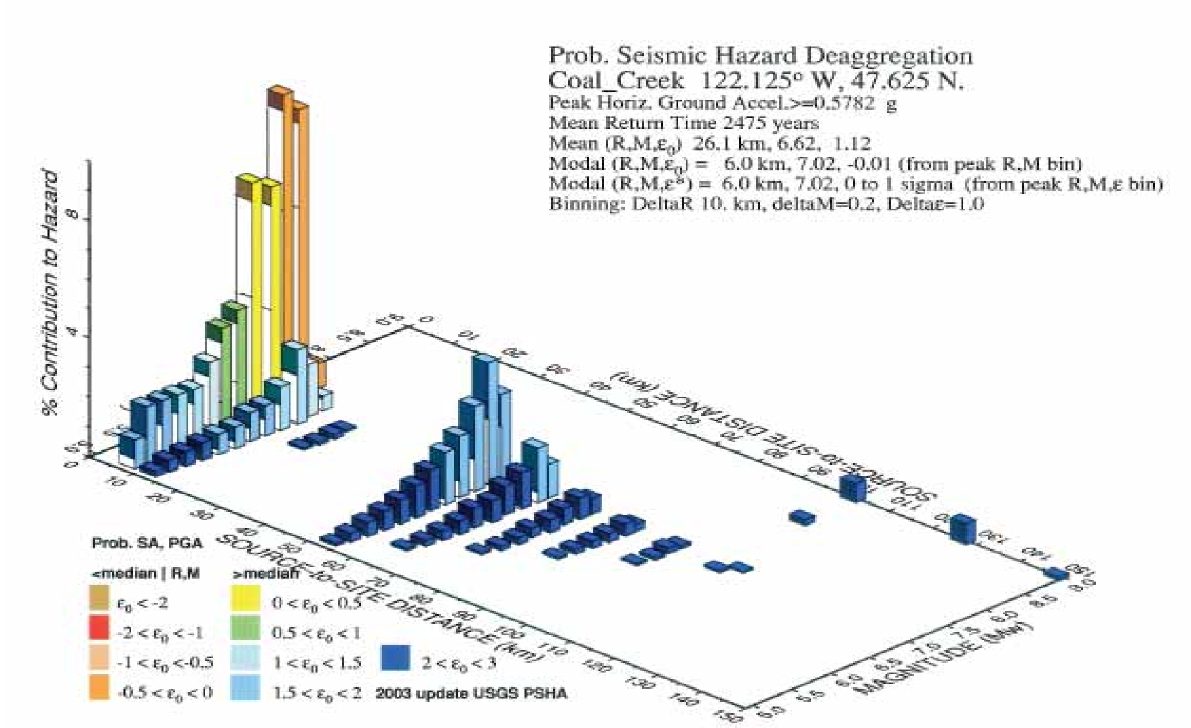

Figure 7. Deaggregation Results for PGA at 2,475-Year Return Period

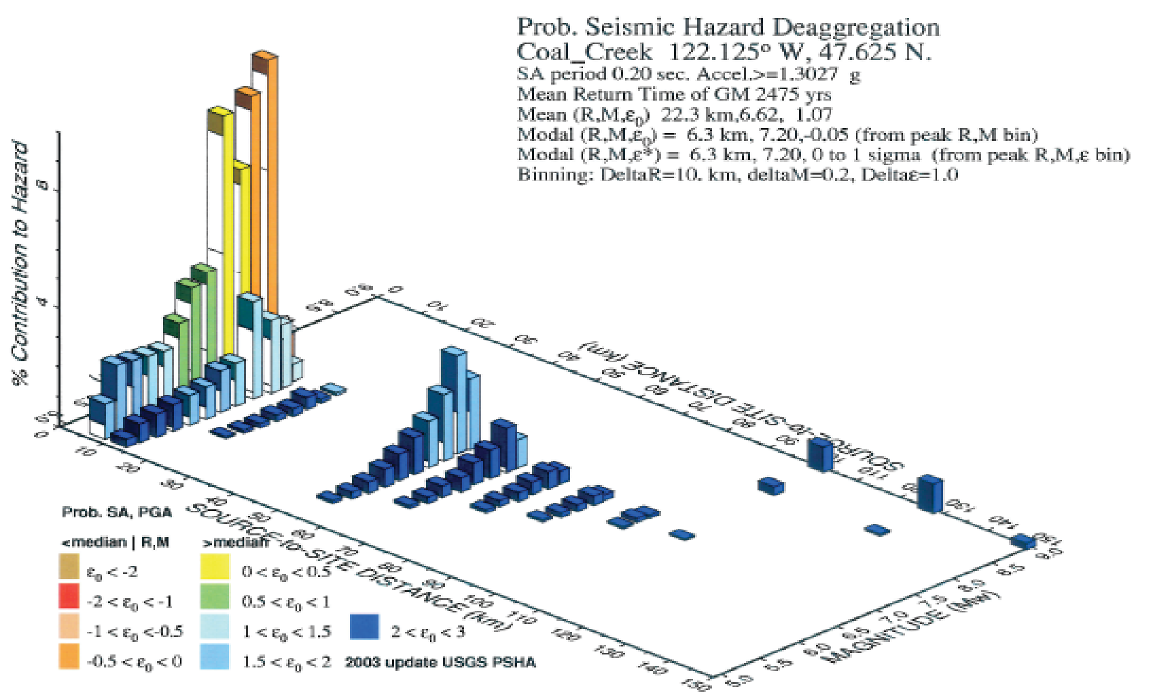

Figure 8. Deaggregation Results for S_s at 2,475-Year Return Period

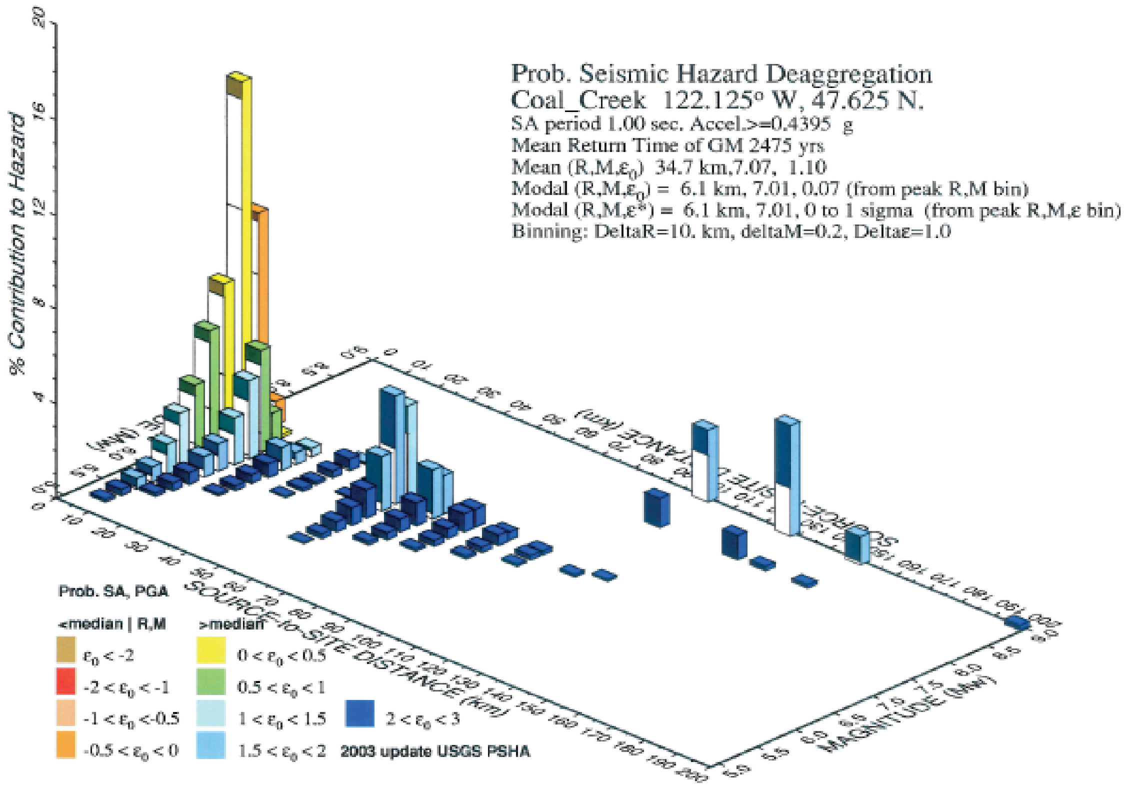

Figure 9. Deaggregation Results for S_1 at 2,475-Year Return Period

SLIDE Results

The computer program SLIDE was used to determine the static factor of safety and then the yield accelerations for the various cases involved. With the yield acceleration (k_y), PGV, and k_{max-d}, the displacements were estimated from the analyses, and results are summarized in Table 2. The SLIDE analyses that support these calculations are shown Figures 10 to 25. Note that the SLIDE results are shown in terms of factor of safety. As noted elsewhere, the factor of safety is equivalent to the capacity to demand (C/D) ratio that is being used in the Section Y Specifications.

Table 2. Results of Ground Displacement Estimates for Example Stability Evaluation

Parameter	Slope Angle	Static C/D Ratio	k_{yield}	Ground Motion Displacement (inches) 7% in 75 Years	10% in 50 Years	2% in 50 Years
Upper Bound Till (ϕ = 42 degrees)						
Case 1	1H to 1V	0.9	NA	NA	NA	NA
Case 2	1.5H to 1V	1.3	0.13	6-9	3-5	14-18
Case 3	2H to 1V	1.7	0.25	<1	<1	3-4
Upper Bound Till (ϕ = 38 degrees, c = 200 psf)						
Case 1	1H to 1V	1.2	0.09	12-19	7-11	26-32
Case 2	1.5H to 1V	1.6	0.26	<1	0	3
Case 3	2H to 1V	2.0	0.32	0	0	<1
Lower Bound Till (ϕ = 36 degrees)						
Case 1	1H to 1V	0.8	NA	NA	NA	NA
Case 2	1.5H to 1V	1.2	0.07	18-27	11-17	36-44
Case 3	2H to 1V	1.5	0.17	3-5	1-2	8-11

Note: (1) NA indicates no analysis conducted. Static factor of safety was less than 1.0.
(2) Upper bound of range based on 84% confidence interval
(3) 5% in 50 years is approximately equivalent to the AASHTO hazard level of 7% in 75 years

Conclusions

The above summary indicates that the displacements ranged from zero to over 40 inches, depending on the assumptions made for soil properties, the design earthquake, and the steepness of the slope. This information allows the owner and the design engineers to decide on the approach to take for the widening of the road.

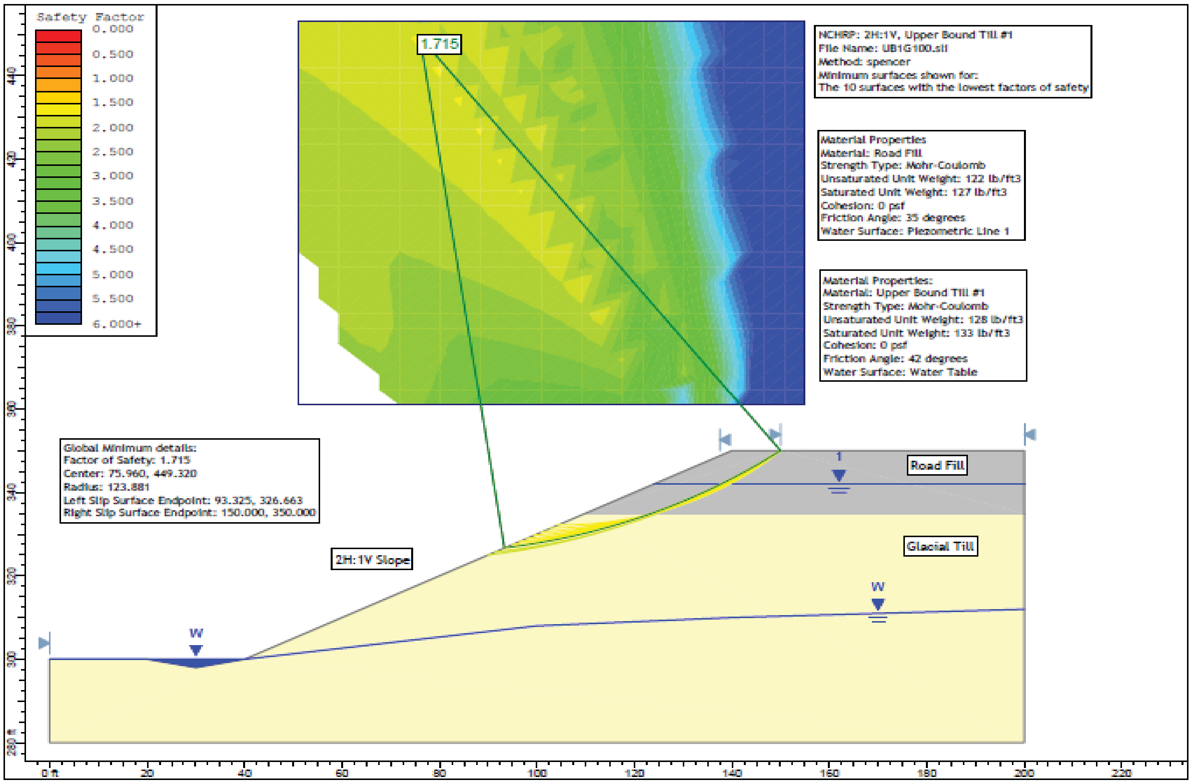

Figure 10. SLIDE results for 2H:1V slope, ϕ = 42 degrees, c = 0 psf, no seismic

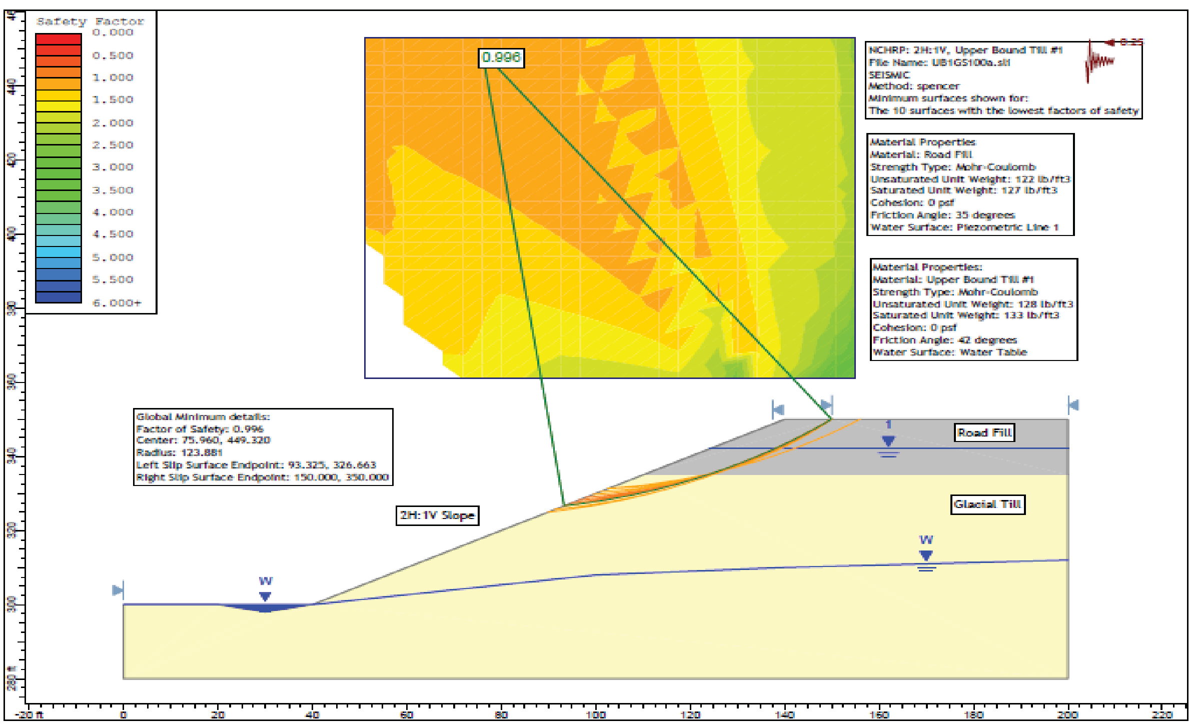

Figure 11. SLIDE results for 2H:1V slope, ϕ = 42 degrees, c = 0 psf, seismic

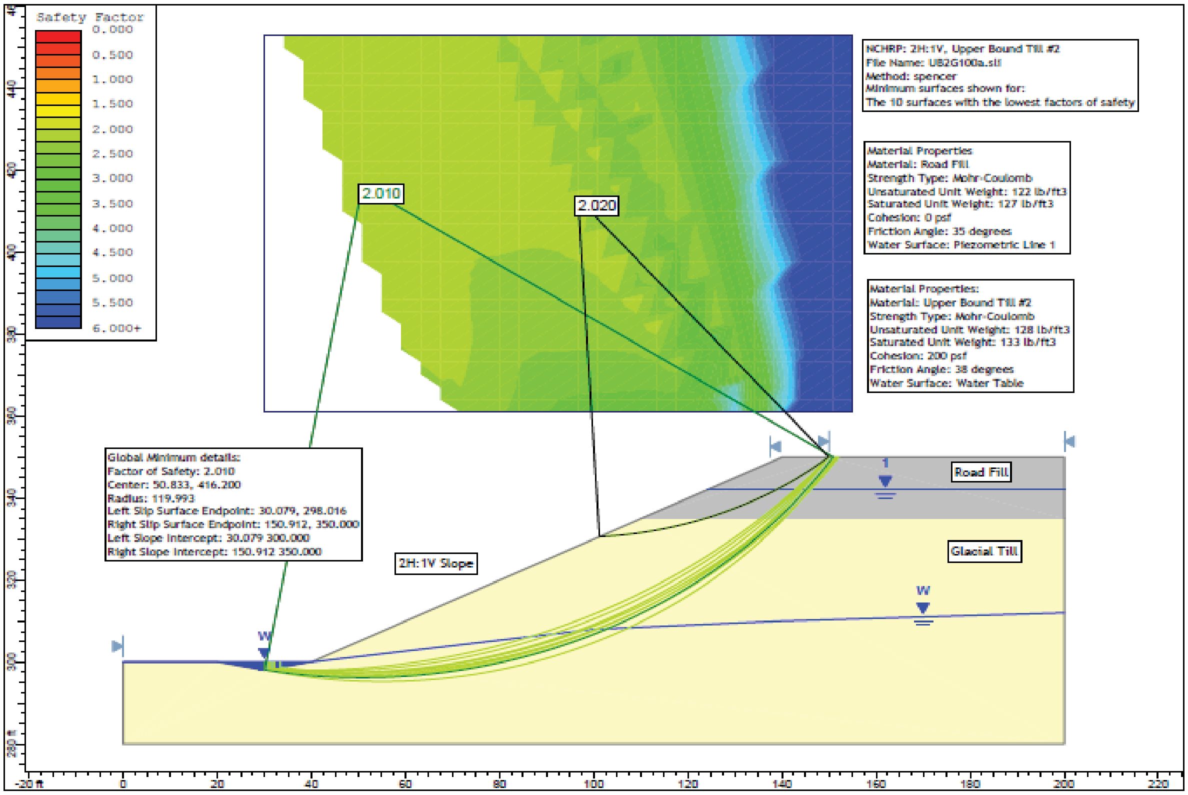

Figure 12. SLIDE results for 2H:1V slope, ϕ = 38 degrees, c = 200 psf, no seismic

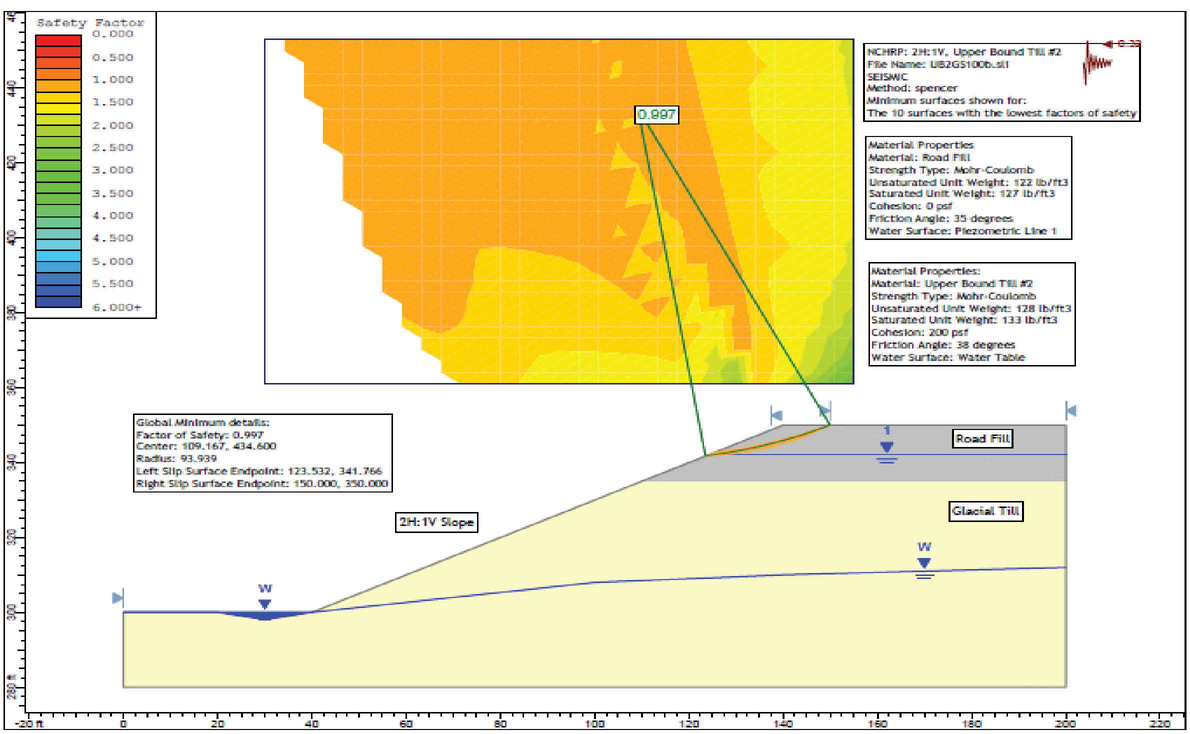

Figure 13. SLIDE results for 2H:1V slope, ϕ = 38 degrees, c = 200 psf, seismic

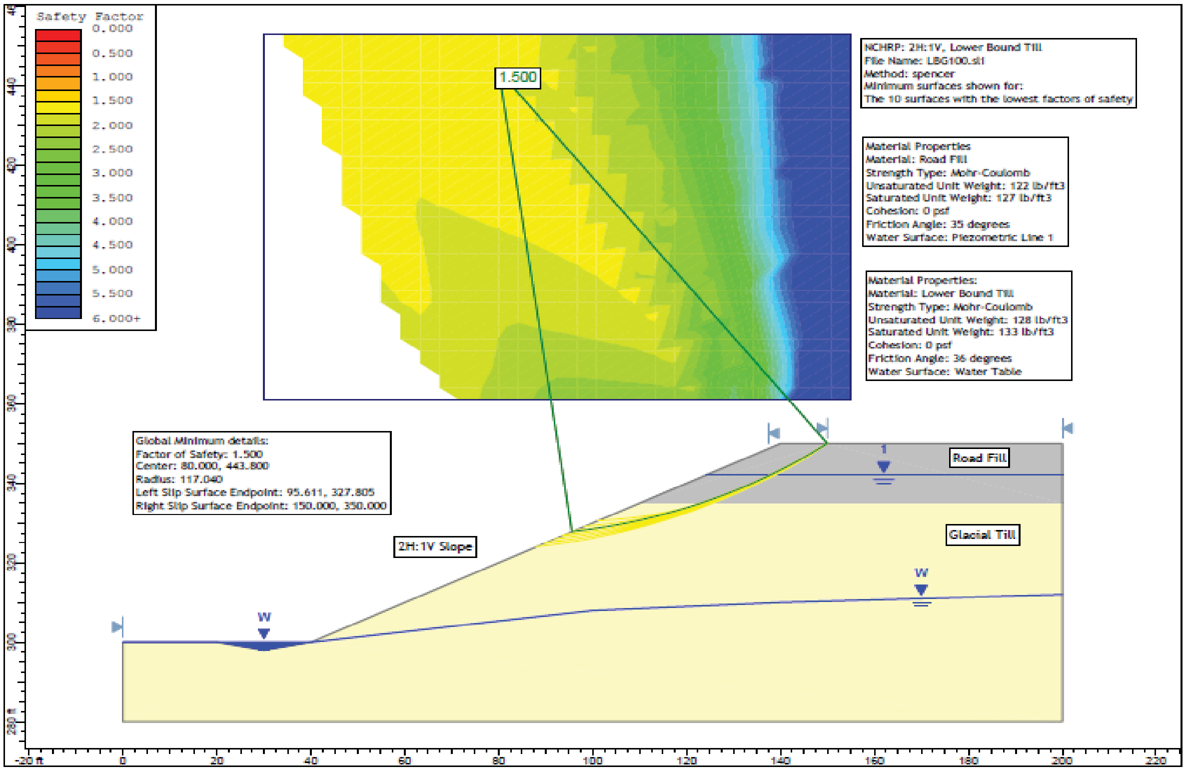

Figure 14. SLIDE results for 2H:1V slope, ϕ = 36 degrees, c = 0 psf, no seismic

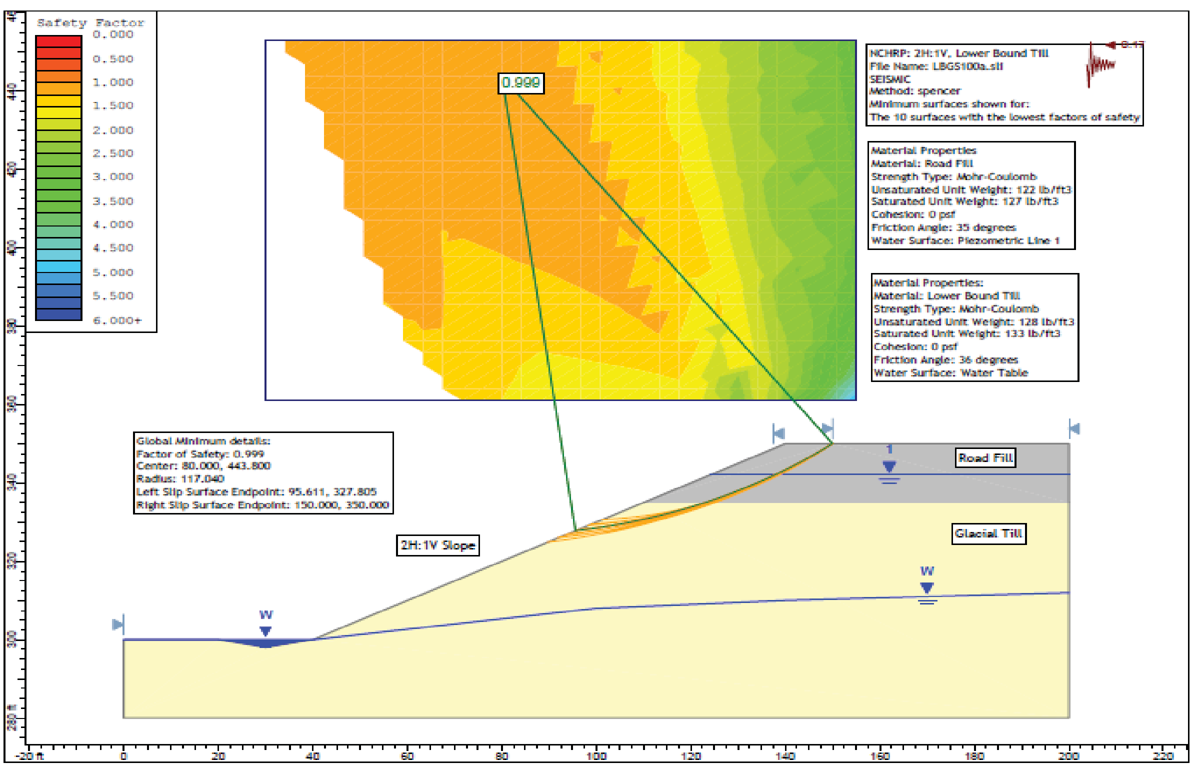

Figure 15. SLIDE results for 2H:1V slope, ϕ = 36 degrees, c = 0 psf, seismic

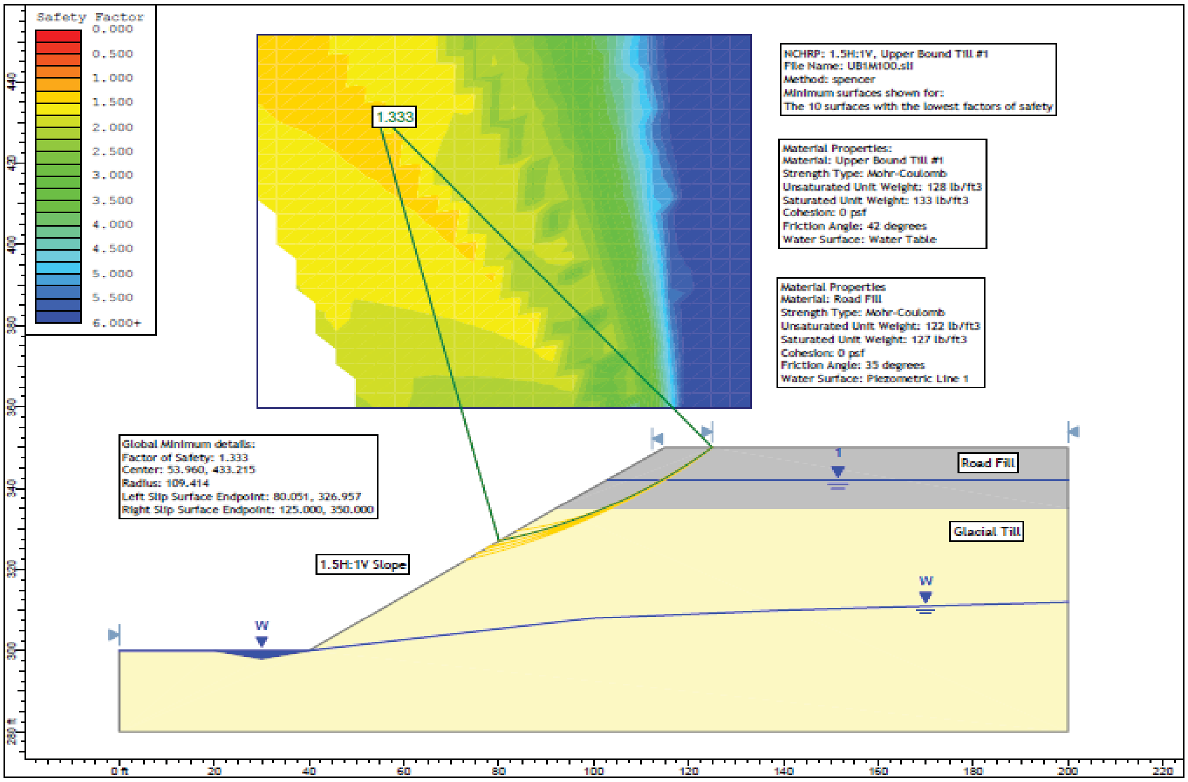

Figure 16. SLIDE results for 1.5H:1V slope, ϕ = 42 degrees, c = 0 psf, no seismic

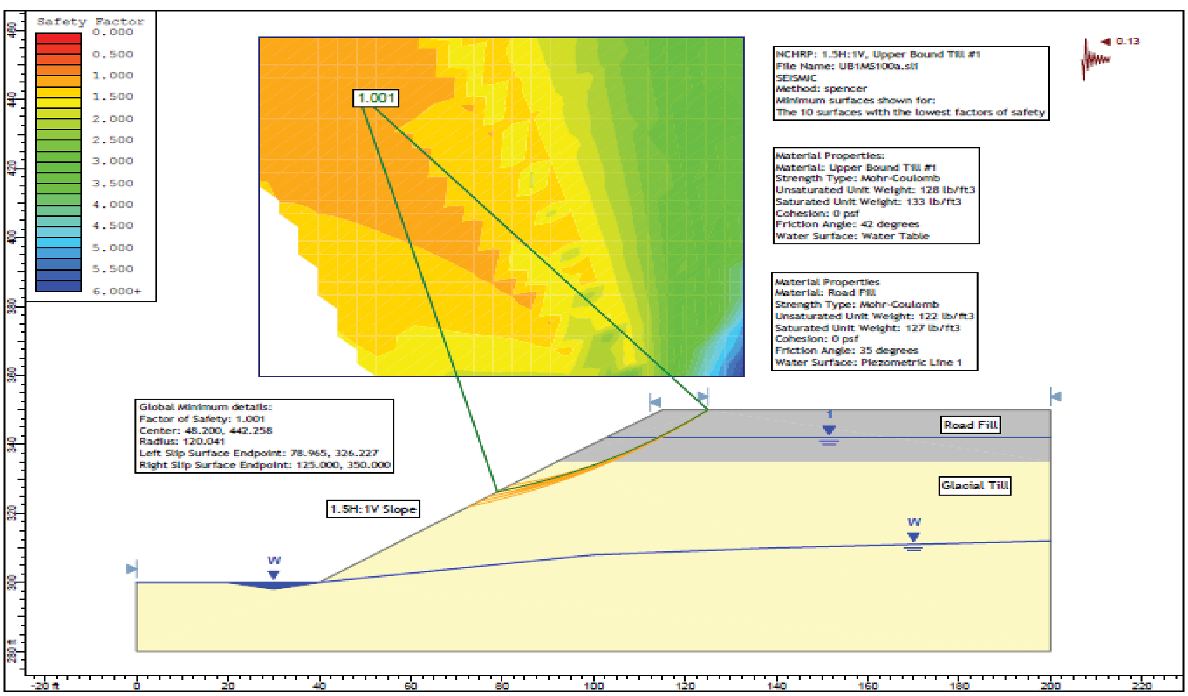

Figure 17. SLIDE results for 1.5H:1V slope, ϕ = 42 degrees, c = 0 psf, seismic

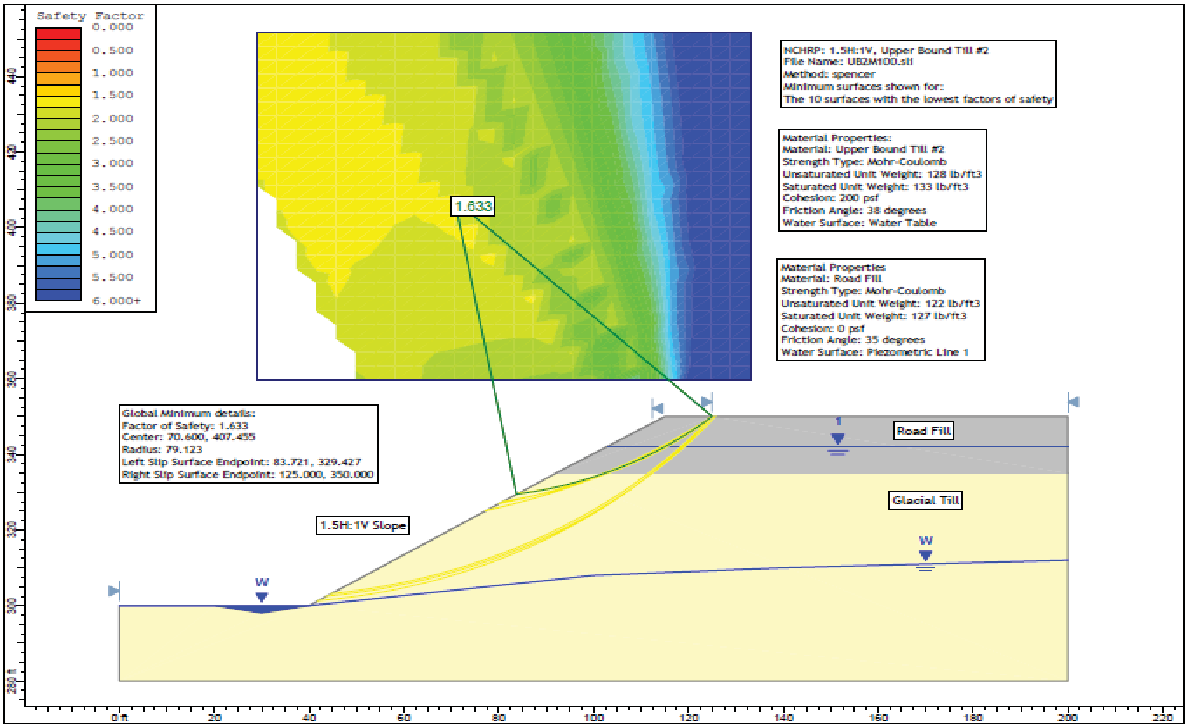

Figure 18. SLIDE results for 1.5H:1V slope, ϕ = 38 degrees, c = 200 psf, no seismic

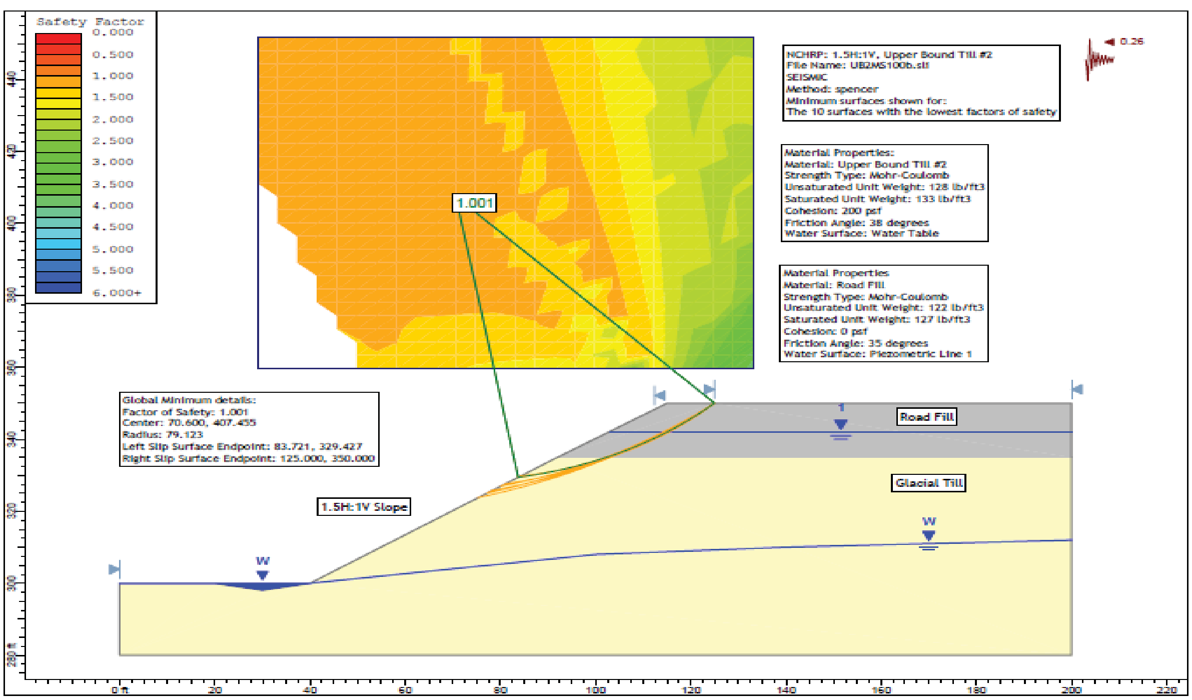

Figure 19. SLIDE results for 1.5H:1V slope, ϕ = 38 degrees, c = 200 psf, seismic

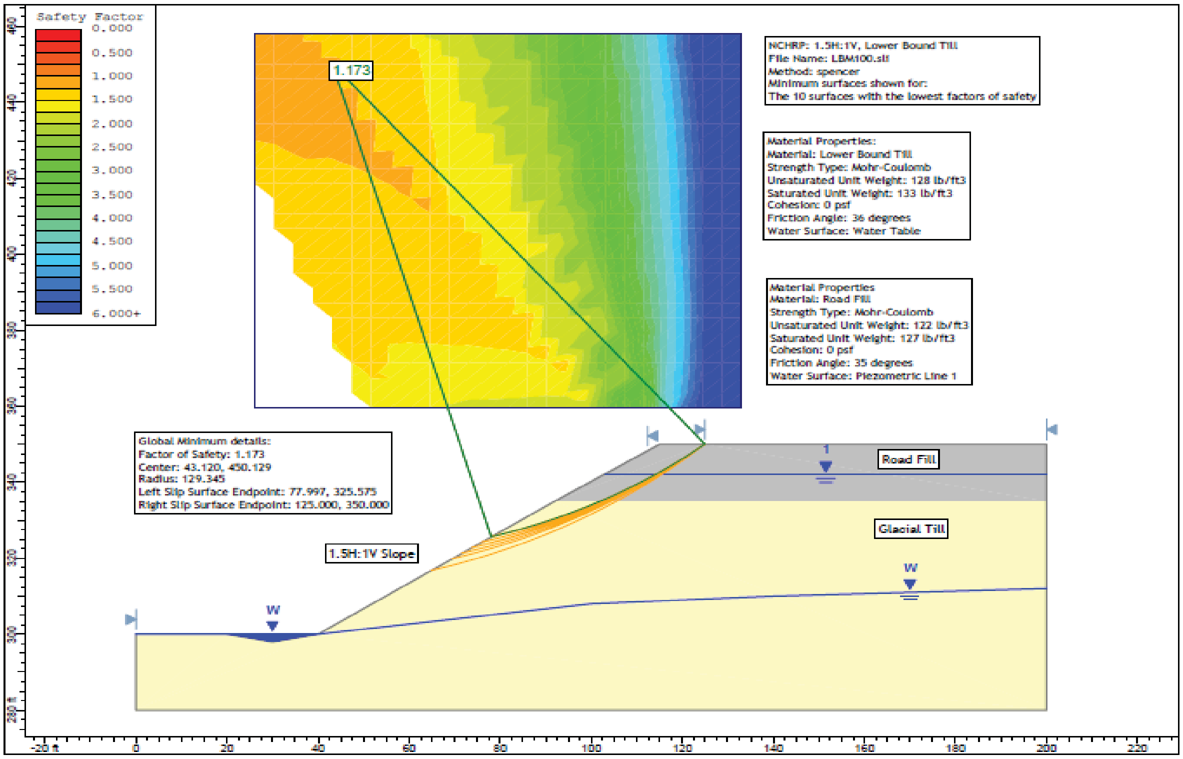

Figure 20. SLIDE results for 1.5H:1V slope, ϕ = 36 degrees, c = 0 psf, no seismic

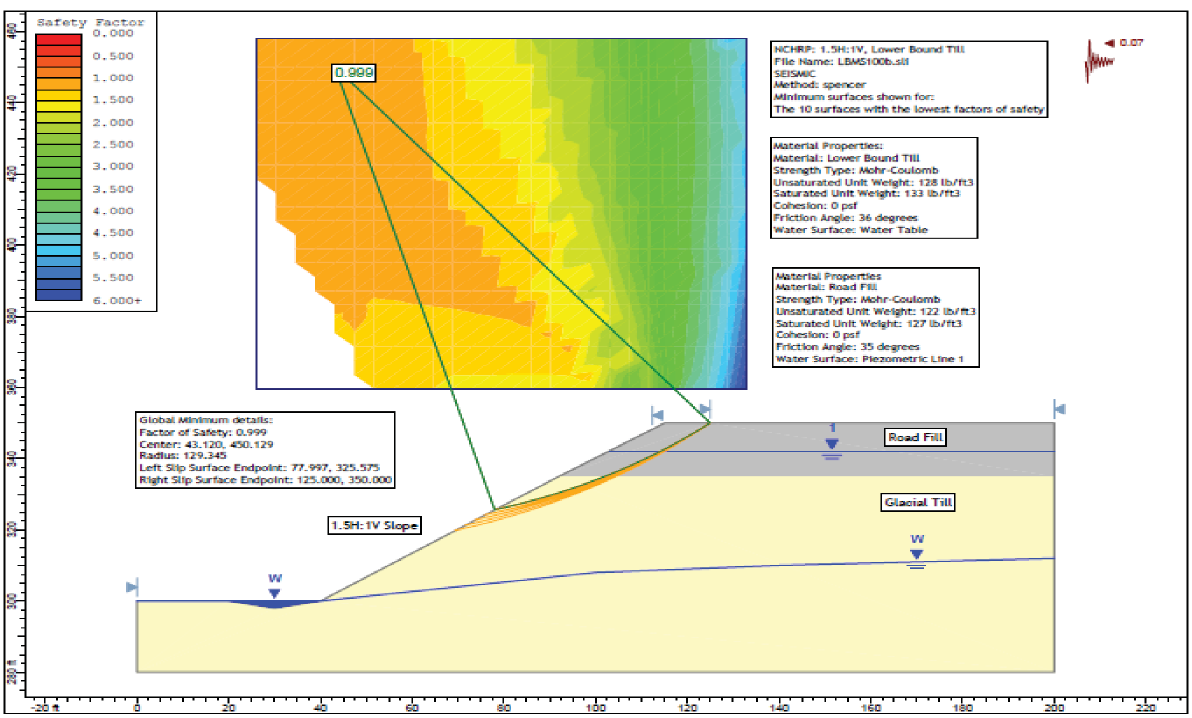

Figure 21. SLIDE results for 1.5:1V slope, ϕ = 36 degrees, c = 0 psf, seismic

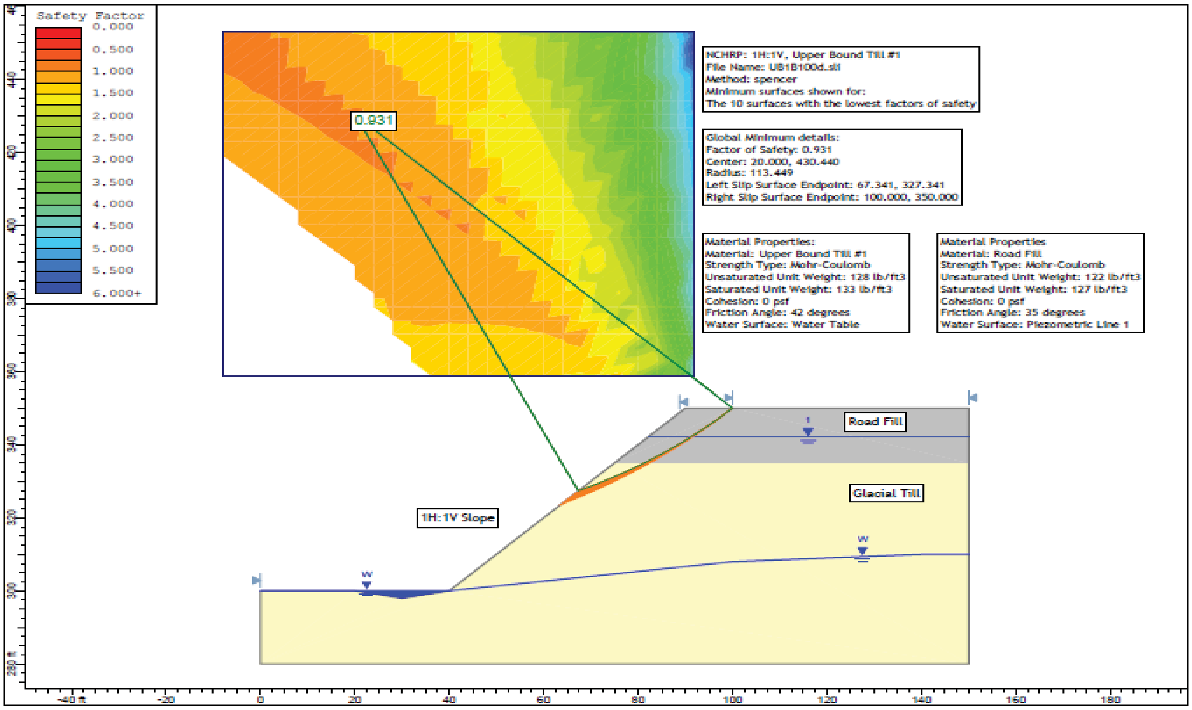

Figure 22. SLIDE results for 1H:1V slope, ϕ = 42 degrees, c = 0 psf, no seismic

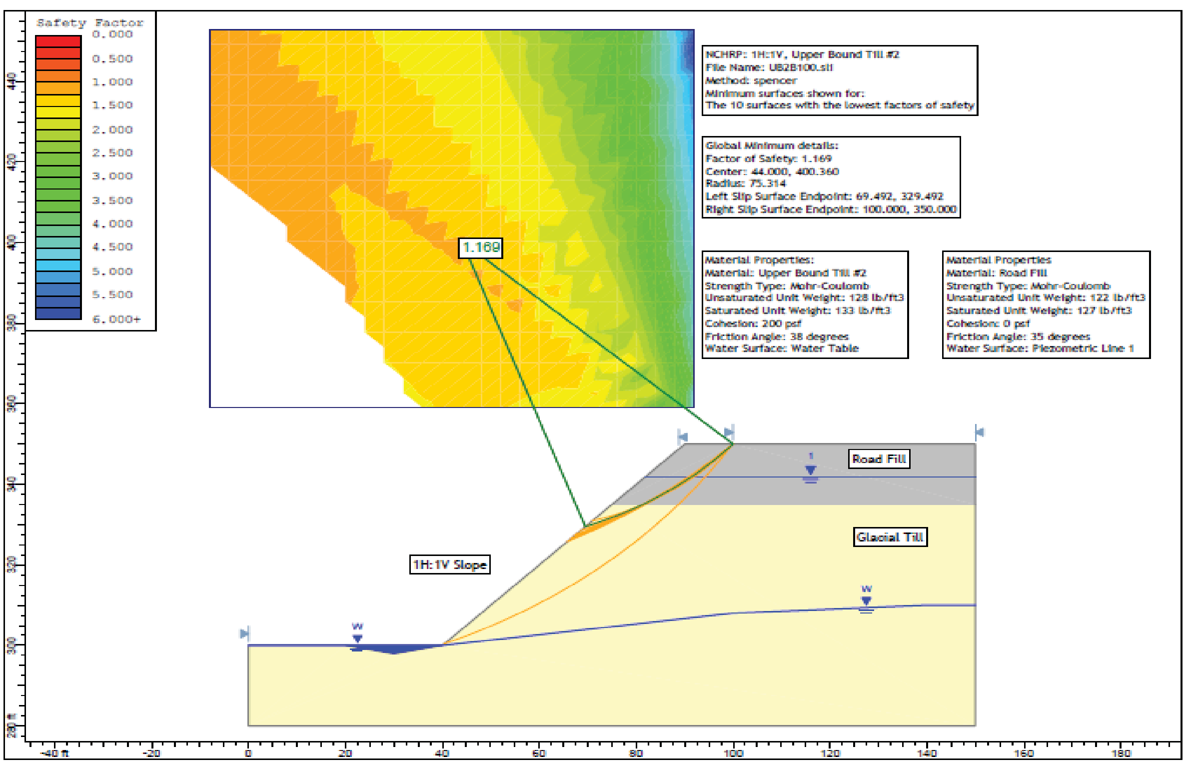

Figure 23. SLIDE results for 1H:1V slope, ϕ = 38 degrees, c = 200 psf, no seismic

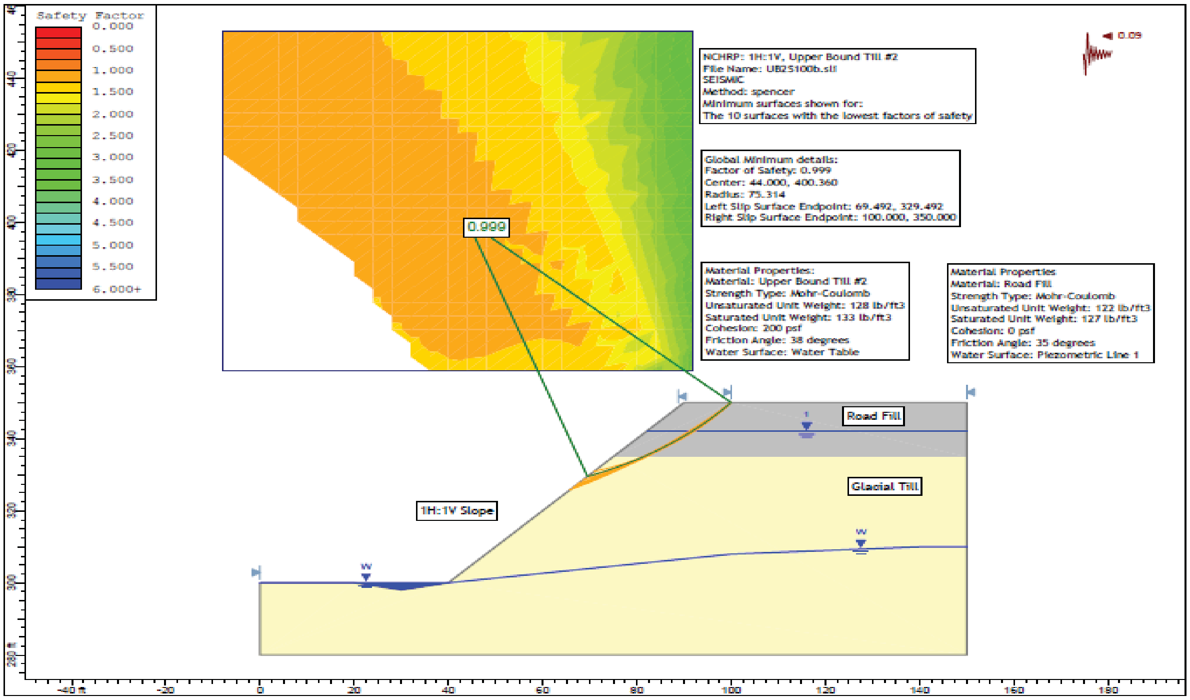

Figure 24. SLIDE results for 1H:1V slope, ϕ = 38 degrees, c = 200 psf, seismic

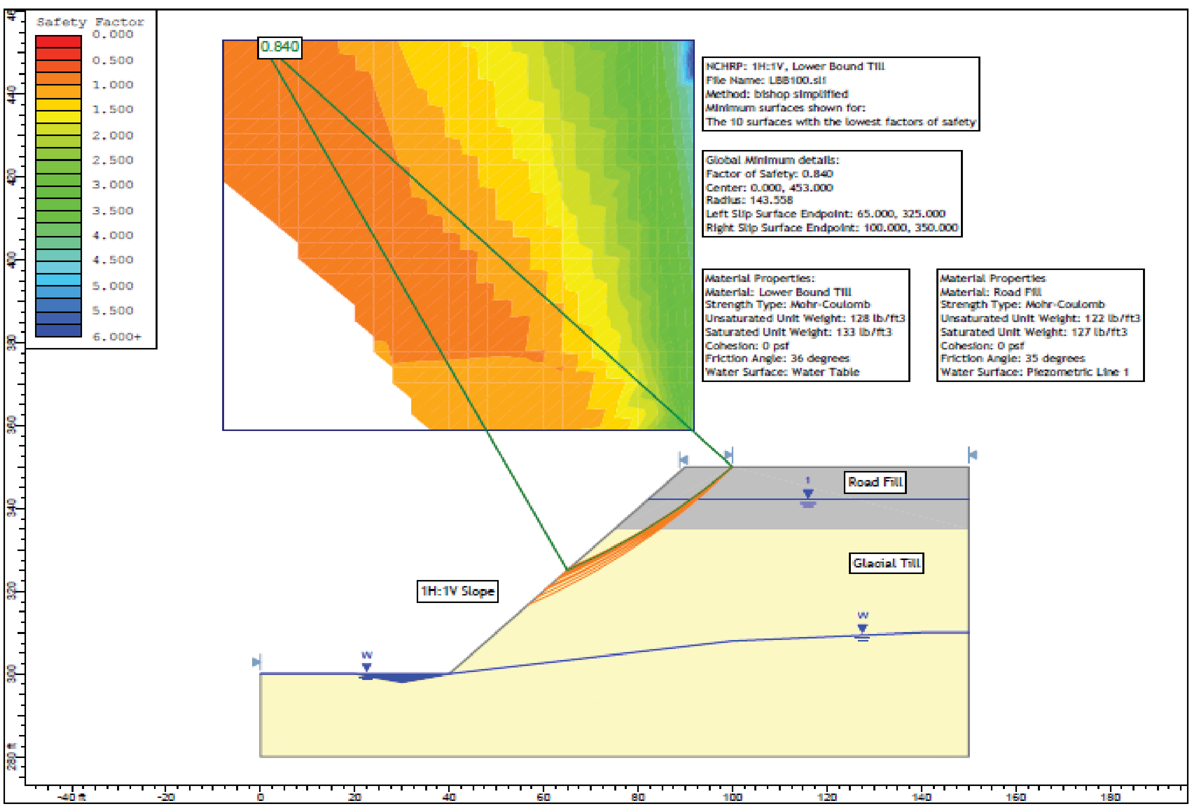

Figure 25. SLIDE results for 1H:1V slope, ϕ = 36 degrees, c = 0 psf, no seismic

Example Buried Circular Pipe Problem

Introduction

The following example demonstrates the application of the procedures outlined in Section Z of the proposed Specifications for the seismic design of buried structures. The example considers corrugated metal pipes (CMP) undergoing ovaling deformations when subjected to ground shaking effects. Pipes with two different diameters were considered: 10 feet and 3 feet. The pipes were assumed to be constructed in a firm/stiff ground, represented by an effective Young's modulus of E_m=3,500 psi. Furthermore, three sites with different levels of seismic activity were included in this example, similar to those assumed in the Example Embankment Slope Problem discussed earlier. The overall site condition is characterized as Site Class D (i.e., firm ground) using the site classification categories table (i.e., Table X.4-1) defined in Section X of the proposed Specification.

The following subsections summarize (1) the CMP material properties and geometry of the corrugated steel metal pipes and the soil properties used in the example, (2) the seismicity for the three sites considered, (3) the general methodology followed, (4) the derivation of the results of the seismic evaluations, and (5) preliminary conclusions made from these analyses.

CMP Material Properties and Geometry and Soil Properties

Two steel corrugated metal pipes with varying sizes (10-ft and 3-ft diameters) were examined in this example. The various material properties and geometry/dimensions of the pipes are summarized in the Table below.

Table 1. CMP Material Properties and Geometrical Parameters

Steel CMP Properties/Geometry	d=10 ft Diameter Steel CMP	d=3 ft Diameter Steel CMP
Corrugation Pitch (inch)	5	2.67
Corrugation Depth (inch)	1	0.5
Pipe Thickness (inch)	0.168	0.168
Young's Modulus, E_l (psi)	2.9E+07	2.9E+07
Moment of Inertia, I_l (ft^4/ft)	0.0000145 (=0.301 in^4/ft)	0.0000033 (=0.069 in^4/ft)
Sectional Area, A_l (ft^2 per ft)	0.0152 (=2.186 in^2/ft)	0.0148 (=2.133 in^2/ft)
Poisson's Ratio, ν_l	0.3	0.3

The 10-foot diameter CMP was constructed with a soil cover depth (i.e., soil overburden thickness) of 10 feet; and for the 3-foot diameter CMP the soil cover depth is 5 feet, as depicted in Figure 1.

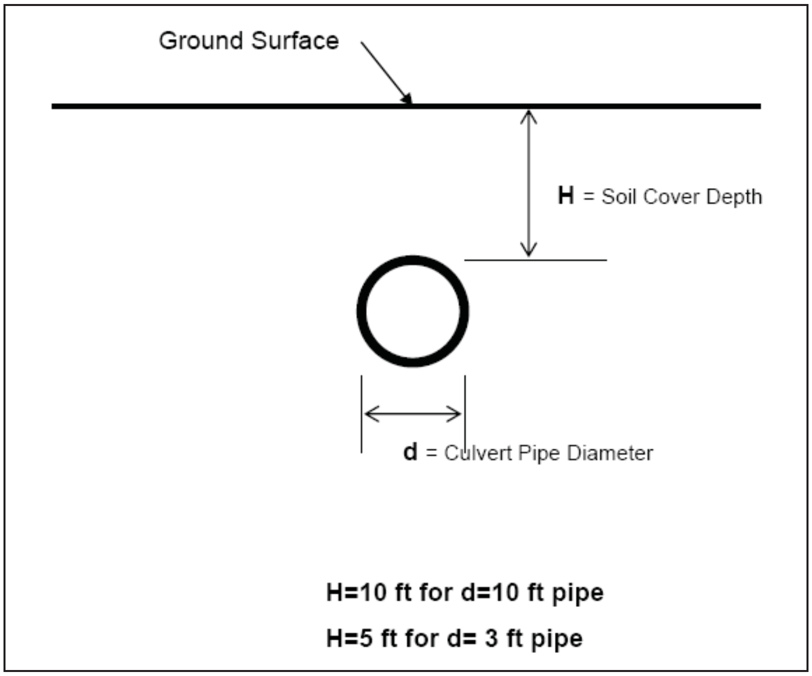

Figure 1. Pipe Burial Depth Configuration

The density/consistency of the soil surrounding the pipes is described as firm/stiff, and its engineering properties were characterized as follows in this study:

- Effective Young's Modulus, E_m = 3,500 psi = 504,000 psf;
- Poisson's Ratio, $\nu_m = 0.3$;
- Effective Shear Modulus, $G_m = E_m/2(1 + \nu_m)$ = 1,346 psi = 193,824 psf ;
- Total Unit Weight, γ_m = 115 pcf.

Seismicity

Following the approach taken for the *Example Embankment Slope Problem,* three sites with different levels of seismic activity were included in this example problem. Two of the sites are located in the western United States (WUS) – the Los Angeles area and the Seattle area. The third site is located in central and eastern United States (CEUS) - Charleston, South Carolina.

The overall subsurface free-field soil profile at the all three sites was assumed to be in the category of Site Class D (i.e., firm/stiff ground) using the site classification categories table defined in Section X of the proposed Specifications. From the ground motion parameters derived for the *Example Embankment Slope Problem,* the site adjusted peak ground accelerations coefficient (PGA) and the spectral acceleration coefficients at period T=1.0 second (S_1) for each site were summarized in Table 2 below. These parameters were determined from USGS/AASHTO Seismic Design Parameters for 2006

AASHTO Seismic Guidelines, based on a design earthquake event that is represented by an average return period of 1,000 years.

Table 2. Site Locations and Ground Motion Parameters (Based on USGS/AASHTO 1,000-yr Return Period Earthquake)

Site Coordinates		Locations (Region)	For Site Class "D"	
Longitude	Latitude		F_{pga}PGA	F_vS_1
-117.9750	34.0500	Los Angles, CA (WUS)	0.600	0.782
-122.2500	47.2700	Seattle, WA (WUS)	0.460	0.535
-079.2370	33.1000	Charleston, SC (CEUS)	0.298	0.237

Methodology

The methodology followed that outlined in Section Z of the proposed Specifications in evaluating the seismic effect due to ground shaking on the ovaling response of circular culverts/pipes. The general procedure involves the following three simple steps:

1. Estimate the maximum free-field ground shear strain – γ_{max}, (due to the vertically traveling shear waves) using design ground motion parameters and the stiffness properties of the ground;

2. Derive the soil-structure interaction factors – the Compressibility Ratio (C) and the Flexibility Ratio (F) using the culvert/pipe (in this case, steel CMP) material properties and geometrical data and soil properties;

3. Using the information derived from Steps 1 and 2 above, calculate the earthquake-induced maximum bending moment (M_{max}) and maximum thrust/hoop force (T_{max}) taking into account the soil-structure interaction effect.

Derivation of Results of Seismic Evaluations

Step 1: Estimate the maximum free-field ground shear strain (γ_{max}) using design ground motion parameters and the stiffness properties of the ground.

As discussed in the proposed Specifications, in general the maximum free-field ground shear strain (γ_{max}) can be roughly estimated using the equation, γ_{max} = PGV/C_{se} , provided that the structure in question is constructed a significant depth below the ground surface (Note: PGV is the peak ground velocity PGV at the depth of interest, and C_{se} is the effective shear wave traveling velocity of the soil surrounding the pipe). For most highway culverts/pipes, however, the burial depths are generally relatively shallow (i.e., within the upper 50 feet from the ground surface). Under this condition, it is more reasonable to estimate the maximum free-field shear strain using the following equations on the following page. [It should be noted that the maximum free-field shear strains can be more accurately estimated by performing a more refined free-field site response analysis, e.g., by performing the SHAKE analysis. The simplified procedure presented in this example was used in the absence of site-specific site response analysis].

$$\gamma_{max} = \tau_{max}/G_m$$

where:

τ_{max} = maximum earthquake- induced shear stress = (F_{pga} PGA) σ_v R_d

σ_v = γ_m (H + d)

= Total vertical soil pressure at the depth corresponding to the invert elevation of the pipe (γ_m is the total unit weight and H and d are as shown in Figure 1).

R_d = Stress Reduction Factor

= 1.0 - 0.00233z for z < 30 ft

= 1.174 - 0.00814z for 30 ft < z < 75 ft

z = Depth of interest = (H + d)

G_m = Effective shear modulus of the soil surrounding the pipes

= 1,346 psi = 193,824 psf

Using the procedure and parameters/properties discussed above, the resulting maximum free-field shear strains for the two pipes located in three different locations are summarized in Tables 3a and 3b, for the d=10 feet diameter CMP and d=3 feet diameter CMP cases, respectively.

Table 3a. Estimated Maximum Free-Field Ground Shear Strains (for d = 10 ft Diameter Pipe Case)

Location	**Los Angeles, CA**	**Seattle, WA**	**Charleston, SC**
F_{pga} PGA	0.600	0.460	0.298
r_m	115 pcf	115 pcf	115 pcf
H + d	20 ft	20 ft	20 ft
$\sigma_v = r_m$ (H + d)	2,300 psf	2,300 psf	2,300 psf
R_d (for z=20’)	0.953	0.953	0.953
τ_{max} = (PGA/g) σ_v R_d	1,315 psf	1,008 psf	653 psf
G_m	193,824 psf	1,824 psf	193,824 psf
$\gamma_{max} = \tau_{max}/G_m$	0.00678	0.00520	0.00337

Table 3b. Estimated Maximum Free-Field Ground Shear Strains (for d = 3 ft Diameter Pipe Case)

Location	Los Angeles, CA	Seattle, WA	Charleston, SC
F_{pga} PGA	0.600	0.460	0.298
r_m	115 pcf	115 pcf	115 pcf
H + d	8 ft	8 ft	8 ft
$\sigma_v = r_m$ (H + d)	920 psf	920 psf	920 psf
R_d (for z=8')	0.981	0.981	0.981
$\tau_{max} = (F_{pga}$ PGA) σ_v R_d	542 psf	415 psf	269 psf
G_m	193,824 psf	193,824 psf	193,824 psf
$\boldsymbol{\gamma_{max} = \tau_{max}/G_m}$	0.00280	0.00214	0.00139

Step 2: Derive the soil-structure interaction factors – the Compressibility Ratio(C) and the Flexibility Ratio (F) using the culvert/pipe material properties and geometrical data and soil properties.

Follow the procedure outlined in Section Z in the proposed Specifications, the two relative stiffness parameters [(Compressibility Ratio (C) and Flexibility Ratio (F)] can be derived using the following formula to account for the soil-structure interaction effects. The results are presented in Tables 4a and 4b for the d=10 feet diameter CMP and d=3 feet diameter CMP cases, respectively.

$$C = \{E_m (1-\nu_l^2) R\} / \{E_1 A_l (1+\nu_m)(1-2\nu_m)\}$$

$$F = \{E_m (1-\nu_l^2) R^3\} / \{6 E_l I_l (1+\nu_m)\}$$

Table 4a. Compressibility Factor "C" and Flexibility Factor "F" (for d = 10 ft Diameter Pipe Case)

	For All Three Sites
R	5 ft
E_m (soil)	504,000 psf (3,500 psi)
ν_m (soil)	0.3
A_l	0.0152 ft^2/ft
I_l	0.0000145 ft^4/ft
E_l	4.18+09 psf (2.9E+07 psi)
ν_l	0.3
C	0.069
F	121.4

Table 4b. Compressibility Factor "C" and Flexibility Factor "F" (for d = 3 ft Diameter Pipe Case)

	For All Three Sites
R	1.5 ft
E_m (soil)	504,000 psf (3,500 psi)
ν_m (soil)	0.3
A_l	0.0148 ft^2/ft
I_l	0.0000033 ft^4/ft
E_l	4.18+09 psf (2.9E+07 psi)
ν_l	0.3
C	0.021
F	14.4

The results in Tables 4a and 4b show that in both cases the pipes are very flexible relative to the surrounding ground. As it can be seen, the flexibility ratios of F=121.4 for the 10-foot diameter pipe and F=14.4 for the 3-foot diameter pipe are significantly greater than 1.0, suggesting the surrounding ground is much stiffer than the lining. The distortion (i.e., the ovaling) of the pipes, therefore, will conform to the ground.

The computed compressibility ratios (0.069 and 0.021 for the 10-foot and 3-foot pipe cases respectively), on the other hand, are significantly lower than 1.0. This suggests that the ring compression stiffness of the pipe is significantly greater than the ground and hence tends to resist the ground strains in the compression/extension senses.

Step 3: Calculate the earthquake induced maximum bending moment (M_{max}) and maximum thrust/hoop force (T_{max})

Using the estimated maximum ground shear strain (γ_{max}) from Step 1 and the two soil-structure interaction parameters derived from Step 2 above (C and F values), the maximum bending moment (M_{max}) and the maximum thrust/hoop force (T_{max}) can be calculated using the following equations as outlined in Section Z in the proposed Specification.

$$M_{max} = \{(1/6)\, k_1 [\, E_m / (1 + \nu_m)\,]\, R^2\, \gamma_{max}\}$$

(full-slip interface condition)

$$T_{max} = \{\, k_2\, [\, E_m / 2\,(1 + \nu_m)\,]\, R\, \gamma_{max}\}$$

(no-slip interface condition)

where:

$$k_1 = 12\,(1-\nu_m) / (2F+5-6\nu_m)$$

The results are summarized in Table 5a for the d=10 foot pipe case and Table 5b for the d=3 foot pipe case.

Table 5a. Maximum Bending Moments and Thrust/Hoop Forces (for d = 10 ft Diameter Pipe Case)

Location	Los Angeles, CA	Seattle, WA	Charleston, SC
γ_{max}	0.00678	0.00520	0.00337
E_m (soil)	504,000 psf	504,000 psf	504,000 psf
ν_m (soil)	0.3	0.3	0.3
R	5 ft	5 ft	5 ft
C	0.069	0.069	0.069
F	121.4	121.4	121.4
k_1	0.034	0.034	0.034
k_2	1.158	1.158	1.158
M_{max}	374 ft-lb/ft	287 ft-lb/ft	186 ft-lb/ft
T_{max}	7,610 lb/ft	5,837 lb/ft	3,783 lb/ft

Based on the results for the 10-foot diameter pipe, as well as the pipe properties (i.e., values of A_1, I_1, and the corrugation profile depth), the bending stresses resulting from the seismically induced M_{max} range from ±7.5 ksi for the pipe in Los Angeles, ±5.7 ksi in Seattle, to ±3.7 ksi in Charleston. The thrust/hoop stresses due to the seismically induced T_{max} range from ±3.5 ksi in Los Angeles, ±2.7 ksi in Seattle, to ±1.7 ksi in Charleston.

Table 5b. Maximum Bending Moments and Thrust/Hoop Forces (for d = 3 ft Diameter Pipe Case)

Location	Los Angeles, CA	Seattle, WA	Charleston, SC
γ_{max}	0.00280	0.00214	0.00139
E_m (soil)	504,000 psf	504,000 psf	504,000 psf
ν_m (soil)	0.3	0.3	0.3
R	1.5 ft	1.5 ft	1.5 ft
C	0.021	0.021	0.021
F	14.4	14.4	14.4
k_1	0.2625	0.2625	0.2625
k_2	1.1994	1.1994	1.1994
M_{max}	107 ft-lb/ft	82 ft-lb/ft	53 ft-lb/ft
T_{max}	976 lb/ft	746 lb/ft	485 lb/ft

Similarly, for the 3-foot diameter pipe, the bending stresses resulting from the seismically induced M_{max} range from ±9.4 ksi in Los Angeles, ±7.2 ksi in Seattle, to ±4.6 ksi in Charleston. The thrust/hoop stresses due to the seismically induced T_{max} range from ±0.46 ksi in Los Angeles, ±0.35 ksi in Seattle, to ±0.23 ksi in Charleston.

Concluding Comments

The results from the example indicate that in general the steel corrugated pipes are considered flexible relative to ground stiffness from the pipe ovaling response standpoint. From the analysis it can also be shown that the maximum free-field shear strains of the ground (due to vertically traveling shear waves) decreases with decreasing depth.

The seismically induced forces/stresses presented in this example are incremental to other normal loading cases. For evaluating the seismic behaviors of the structures the seismic forces/stresses should be combined with appropriate other loading cases using the Extreme Event Load Combination I, as specified in Table 3.4.1-1 in Section 3 of the AASHTO *LRFD Bridge Design Specifications*.

Example Buried Rectangular Culvert Problem

Introduction

The following example demonstrates the application of the procedure outlined in Section Z of the proposed Specifications for the seismic design of a buried rectangular structure. The example considers a reinforced concrete box culvert undergoing racking deformations when subjected to ground shaking effects. The specific features of the problem are described as follows:

- A concrete box culvert with a 20-foot width and a 10-foot height.
- Assumed to be constructed in a firm/stiff ground, represented by an effective Young's modulus of E_m = 3,500 psi.
- Three sites with different levels of seismic activity similar to those assumed in the *Example Embankment Slope Problem*, as well as in the *Example Buried Pipe Problem* discussed earlier.
- Overall site condition characterized as Site Class D (i.e., firm ground) using the site classification categories table (i.e., Table X.4-1) defined in Section X of the proposed Specifications.

The following subsections summarize (1) the material properties and geometry of the reinforced concrete box culvert and the soil properties used in the example, (2) the seismicity for the three sites considered, (3) the general methodology followed, (4) the derivation of the results of the seismic evaluations, and (5) preliminary conclusions made from these analyses.

Concrete Box Culvert Material Properties and Geometry and Soil Properties

The various material properties and geometry/dimensions of the reinforced concrete box culvert are summarized in Table 1. The box culvert was constructed with a soil cover depth (i.e., soil overburden thickness) of 10 feet, as depicted in Figure 1.

The density/consistency of the soil surrounding the culvert is described as firm/stiff, and its engineering properties are characterized as follows in this study:

- Effective Young's Modulus, E_m = 3,500 psi = 504,000 psf;
- Poisson's Ratio, ν_m = 0.3 ;
- Effective Shear Modulus, $G_m = E_m/2(1+\nu_m)$ = 1,346 psi = 193,824 psf ;
- Total Unit Weight, γ_m = 115 pcf.

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Table 1. Concrete Box Culvert Material Properties and Geometrical Parameters

Width of Box Culvert (ft)	20
Height of Box Culvert (ft)	10
Box Culvert Wall/Top/Base Thickness (ft)	Roof Slab: 1 Invert Slab: 1 Walls: 1
Box Culvert Young's Modulus, E_l (psi)	4.0E+06 psi
Culvert Moment of Inertia, I_l (ft^4/ft)	Roof Slab: 0.083 Invert Slab: 0.083t Walls: 0.083
Box Culvert Poisson's Ratio, ν_l	0.2

Note that in the frame analysis the culvert liner is modeled as beam element with the Young's Modulus modified as $E_l/(1- \nu_l^2)$ = 4.17+06 psi to account for the increased stiffness under the two-dimensional plane-strain condition.

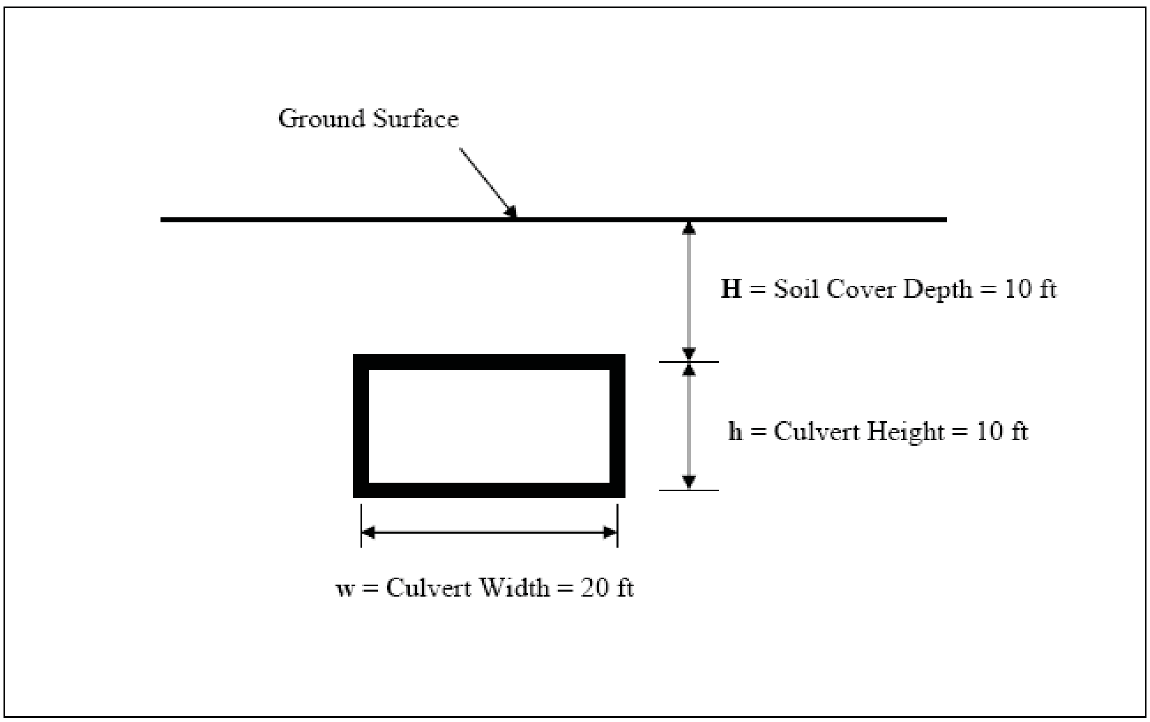

Figure 1. Culvert Burial Depth Configuration

Seismicity

Following the approach taken for the *Example Embankment Slope Problem* as well as the *Example Buried Pipe Problem*, three sites with different levels of seismic activity were included in this example problem. Two of the sites are located in the western United

States (WUS) – the Los Angeles area and the Seattle area. The third site is located in central and eastern United States (CEUS) - Charleston, South Carolina.

The overall subsurface free-field soil profile at all three sites was assumed to be in the category of Site Class D (i.e., firm/stiff ground) using the site classification categories defined in Section X of the proposed Specifications. From the ground motion parameters derived for the *Example Embankment Slope Problem*, the site-adjusted peak ground accelerations coefficient (i) and the spectral acceleration coefficients at a period T=1.0 second (S_1) for each site were as summarized in Table 2. These parameters were determined from USGS/AASHTO Seismic Design Parameters for 2006 AASHTO Seismic Guidelines, based on a design earthquake event that is represented by an average return period of 1,000 years.

Table 2. Site Locations and Ground Motion Parameters (Based on USGS/AASHTO 1,000-yr Return Period Earthquake)

Site Coordinates		Locations (Region)	For Site Class "D"	
Longitude	Latitude		$F_{pga}PGA$	F_vS_1
-117.9750	34.0500	Los Angles, CA (WUS)	0.600	0.782
-122.2500	47.2700	Seattle, WA (WUS)	0.460	0.535
-079.2370	33.1000	Charleston, SC (CEUS)	0.298	0.237

Methodology

The methodology followed procedures outlined in Section Z of the proposed Specifications for evaluating the seismic effect due to ground shaking on the racking response of rectangular culverts. The general procedure involves the following steps:

1. Estimate the maximum free-field ground shear strain – the γ_{max} (due to the vertically traveling shear waves) using design ground motion parameters and the stiffness properties of the ground, and then determine the differential free-field relative displacements ($\Delta_{free\text{-}field}$) corresponding to the top and the bottom elevations of the rectangular/box structure;

2. Determine the racking stiffness (K_S) of the box structure from a simple structural frame analysis using the material properties and geometrical data for the culvert;

3. Derive the soil-structure interaction factor - flexibility ratio (F_{rec}) of the box structure using the racking stiffness (K_S) of the culvert, geometrical data, and the stiffness properties of the surrounding soil;

4. Based on the flexibility ratio obtained from Step 3 above, determine the racking ratio (R_{rec}) for the box structure using empirical relationship or design chart;

5. Determine the actual racking deformation of the box structure (Δ_S) using the racking ratio (R_{rec}) derived from Step 4 and the free-field relative displacements ($\Delta_{free\text{-}field}$) derived from Step 1;

6. Compute the seismic demand in terms of internal forces [bending moments (M) axial forces (T) and shear forces (V)] by imposing Δ_S (derived from Step 5 above) upon the structure using the same frame analysis model used in Step 2.

Derivation of Results of Seismic Evaluations

Step 1: Estimate the maximum free-field ground shear strain (γ_{max}) using design ground motion parameters and the stiffness properties of the ground, and then determine the differential free-field relative displacements ($\Delta_{free\text{-}field}$) corresponding to the top and the bottom elevations of the rectangular/box structure.

As discussed in the proposed Specifications, in general the maximum free-field ground shear strain (γ_{max}) can be roughly estimated using the equation, $\gamma_{max} = PGV/C_{se}$, provided that the structure in question is constructed a significant depth below the ground surface (Note: PGV is the peak ground velocity at the depth of interest, and C_{se} is the effective shear wave traveling velocity of the soil surrounding the pipe). For most highway culverts, however, the burial depths are generally relatively shallow (i.e., within the upper 50 feet from the ground surface). Under this condition, it is more reasonable to estimate the maximum free-field shearing strain using the following equations. [It should be noted that the maximum free-field shear strains can be more accurately estimated by performing a more refined free-field site response analysis, e.g., by performing a SHAKE analysis. The simplified procedure presented in this example was used in the absence of site-specific site response analysis].

$$\gamma_{max} = \tau_{max}/G_m$$

where:

τ_{max} = maximum earthquake-induced shear stress = $(F_{pga}PGA)\ \sigma_v\ R_d$

$\sigma_v = \gamma_m (H + h)$

= total vertical soil pressure at the depth corresponding to the invert elevation of the culvert (γ_m is the total unit weight, and H and h are as shown in Figure 1).

R_d = stress reduction factor

= $1.0 - 0.00233z$ for $z < 30$ ft

= $1.174 - 0.00814z$ for 30 ft $< z < 75$ ft

z = depth of interest = (H + h)

G_m = effective shear modulus of the soil surrounding the structure

= 1,346 psi = 193,824 psf

Using the procedure and parameters/properties discussed above, the resulting maximum free-field shear strains for the box culverts located in the three different geographic areas are summarized in Table 3.

Table 3. Estimated Maximum Free-Field Ground Shear Strains

Location	Los Angeles, CA	Seattle, WA	Charleston, SC
$F_{pga}PGA$	0.600	0.460	0.298
r_m	115 pcf	115 pcf	115 pcf
H + h	20 ft	20 ft	20 ft
$\sigma_v = r_m$ (H + h)	2,300 psf	2,300 psf	2,300 psf
R_d (for z=20')	0.953	0.953	0.953
$\tau_{max} = (F_{pga}PGA)\ \sigma_v\ R_d$	1,315 psf	1,008 psf	653 psf
G_m	193,824 psf	1,824 psf	193,824 psf
$\gamma_{max} = \tau_{max}/G_m$	0.00678	0.00520	0.00337

Based on the maximum free-field shear strains (γ_{max}), the differential free-field relative displacements corresponding to the top and the bottom elevations of the rectangular/box structure can be estimated using the following equation and are presented in Table 4.

$$\Delta_{free\text{-}field} = h \cdot \gamma_{max}$$

Table 4. Estimated Maximum Free-Field Soil Racking Displacements ($\Delta_{free\text{-}field}$)

Location	Los Angeles, CA	Seattle, WA	Charleston, SC
$\Delta_{free\text{-}field} = h \cdot \gamma_{max}$	0.81"	0.62"	0.40"

Step 2: Determine the racking stiffness (K_S) of the box structure from a simple structural frame analysis using the material properties and geometrical data for the culvert.

For practical purposes, the racking stiffness can be obtained by applying a unit lateral force at the roof level, while the base of the structure is restrained against translation, but with the joints free to rotate. The structural racking stiffness is then calculated as the ratio of the applied force to the resulting lateral displacement. Using the culvert material properties and geometrical data presented in Table 1, the results from the simple frame analysis are presented in the Table 5. (Note that in the frame analysis the culvert liner is modeled as beam element with the Young's Modulus modified as $E_l/(1-\nu_l^2)$ = 4.17+06 psi to account for the increased stiffness under the two-dimensional plane-strain condition.)

Table 5. Box Culvert Racking Stiffness

Unit Lateral Force Applied at Roof Level	Resulting Lateral Racking Displacement	Racking Stiffness Ks $K_s = p/\delta$
p = 1 kip	$\delta = 0.03$”	Ks = 33.3 kips/in (= 400 kips/ft)

Step 3: Derive the soil-structure interaction factor - flexibility ratio (F_{rec}) of the box structure using the racking stiffness (K_S) and geometrical data of the culvert and the stiffness properties of the surrounding soil.

Following the procedure outlined in Section Z in the proposed Specifications, the flexibility ratio (F_{rec}) of the box culvert structure can be calculated using the following formula:

$$F_{rec} = (G_m / K_s) \cdot (w/h) = 0.97$$

The flexibility ratio represents the “relative racking stiffness” of the ground to the structure. When $F_{rec} = 0.97$ (i.e., very close to 1.0), it suggests that the racking stiffness of the surrounding ground is about the same as that of the culvert, and therefore, it is expected that the actual culvert racking displacement will be about the same as that of the free-field.

Step 4: Based on the flexibility ratio obtained from Step 3 above, determine the racking ratio (R_{rec}) for the box structure using empirical relationship or design chart.

The racking ratio (R_{rec}) for the culvert structure is estimated using Figure Z.7-8 or the following expression (refer to the proposed Specifications Section Z):

$$R_{rec} = 2F_{rec} /(1 + F_{rec}) = 0.985$$

Step 5: Determine the actual racking deformation of the box structure (Δ_S) using the racking ratio (R_{rec}) derived from Step 4 and the free-field relative displacements ($\Delta_{free-field}$) derived from Step 1.

The actual racking displacements of the culvert structures at the three different sites can be computed using the following equation. Results are presented in Table 6 below.

$$\Delta_S = R_{rec} \cdot \Delta_{free\text{-}field}$$

Table 6. Actual Racking Displacements of Culvert Structure

	Los Angeles, CA	Seattle, WA	Charleston, SC
$\Delta_S = R_{rec} \cdot \Delta_{free\text{-}field}$	0.80”	0.61”	0.39”

Step 6: Compute the seismic demand in terms of internal forces [bending moments (M) axial forces (T) and shear forces(V)] by imposing Δ_S (derived from Step 5 above) upon the structure using the same frame analysis model used in Step 2.

Using the same frame analysis model mentioned in Step 2 above (as depicted in Figure Z.7-9 of the proposed Specifications), the computed maximum culvert liner internal forces (for each structural member) due to the applied racking displacement Δ_S (at the roof slab level) are summarized in Table 7.

Table 7. Maximum Culvert Liner Internal Forces of Each Structural Member

	Los Angeles, CA	Seattle, WA	Charleston, SC
Δ_S	0.80”	0.61”	0.39”
M_{max} (Bending)	Slabs: 67 kip-ft/ft Walls: 67 kip-ft/ft	Slabs: 51 kip-ft/ft Walls: 51 kip-ft/ft	Slabs: 33 kip-ft/ft Walls: 33 kip-ft/ft
T_{max} (Axial)	Slabs: 13.3 kips/ft Walls: 6.7 kips/ft	Slabs: 10.1 kips/ft Walls: 5.1 kips/ft	Slabs: 6.5 kips/ft Walls: 3.2 kips/ft
V_{max} (Shear)	Slabs: 6.7 kips/ft Walls: 13.3 kips/ft	Slabs: 5.1 kips/ft Walls: 10.1 kips/ft	Slabs: 3.2 kips/ft Walls: 6.5 kips/ft

Based on the results for the simple frame analysis, the maximum seismically induced bending moment M_{max} range from ±67 kip-ft/ft for the culvert in Los Angeles, ±51 kip-ft/ft in Seattle, to ±33 kip-ft/ft in Charleston. The seismically induced maximum shear and axial forces range from ±13.3 kips/ft in Los Angeles, ±10.1 kips in Seattle, to ±6.5 kips in Charleston.

Additional Comments

The seismically induced structural displacements and internal forces presented in this example are incremental to other normal loading cases. For evaluating the seismic behaviors of the structures, the seismic forces must be combined with appropriate other loading cases using the Extreme Event Load Combination I, as specified in Table 3.4.1-1 in Section 3 of the AASHTO *LRFD Bridge Design Specifications*.