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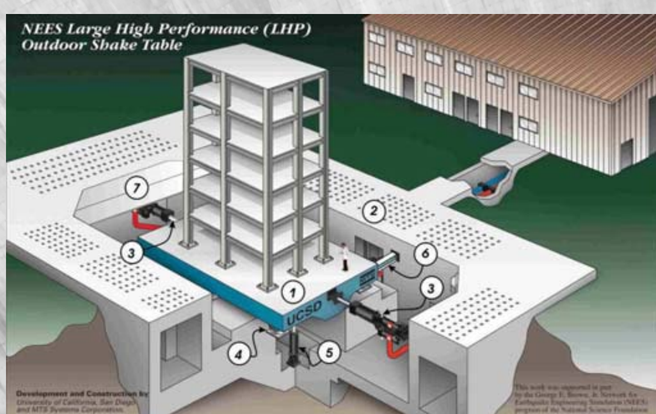
Managing Water in the West

Report DSO-10-01

Soil-Structure Interaction Scoping Phase 1

Dam Safety Technology Development Program



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Steve Dominic, P.E., Bureau of Reclamation, Denver CO, 86-68180
Ming Liu, P.E., Bureau of Reclamation, Denver CO, 86-68110

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14. ABSTRACT
This ongoing research will help answer the questions that continue to arise as Reclamation is faced with detailed analysis and modifications of spillways and other earth-retaining structures subjected to significant seismic loading. The objective of this research is a better analytical tool to predict the seismic lateral earth pressures for configurations that include groundwater, cohesion, nonhorizontal zone, and compaction/in-place density effects under various ground accelerations. This report represents the conclusion to the first phase (Scoping Phase 1) of this research project. The ultimate project purpose is to complete a full scale shake table test of a concrete cantilever retaining wall.

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Reclamation, soil-structure interaction, dynamic, seismic, retaining walls, Mononobe-Okabe, Woods, shake table, model, earthquake, full scale test, numerical analysis, finite element analysis, NEES, LHPOST, concrete, spillways

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Dam Safety Technology Development Program

prepared by

Steve Dominic, P.E.
Ming Liu, P.E.

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Civil Engineering Services Division
Materials Engineering and Research Laboratory Group
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Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

BUREAU OF RECLAMATION
Technical Service Center
Materials Engineering and Research Laboratory Group, 86-68180

Report DSO-10-01

Soil-Structure Interaction Scoping Phase 1

Dam Safety Technology Development Program
Denver, Colorado

Stephen J. Dominic, P.E.

Prepared: Steve Dominic, P.E.
Civil Engineer, Materials Engineering and Research Laboratory Group 86-68180

Checked: Ming Liu, P.E.
Civil Engineer, Structural Analysis Group 86-68110

Technical Approval: John Baals, P.E.
Group Manager, Structural Analysis Group 86-68110

Larry K Nuss

Peer Review: Larry Nuss, P.E.
Structural Engineer, Structural Analysis Group 86-68110

9/10/2009
Date

REVISIONS					
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Introduction

This report presents a compilation of historical and recent technical references regarding dynamic soil-structure interaction (SSI) and outlines a preliminary full scale testing program for further investigation of soil-structure interaction of concrete retaining walls under large magnitude seismic loading. This report further documents completion of the first phase (Scoping Phase 1) of a proposed five-phase research project funded under the Dam Safety Technology Development Program.

The first part of this Scoping Phase 1 consisted of a literature review and compilation of technical references to SSI focused specifically on concrete retaining structures subjected to dynamic lateral earth pressures. The literature review included widely accepted historical methods as well as recently completed studies focused on numerical modeling and physical testing. One of the primary objectives of the literature review was to determine if a full scale concrete retaining wall shake table test has previously been performed and documented within the United States research community or overseas, and to identify sources of existing test data on the measured response of concrete retaining walls under seismic loads. A compilation of technical references regarding dynamic SSI was completed and is included as a compact disc (CD) (appendix A) in this report.

The second part of this Scoping Phase consisted of evaluating laboratory capabilities at the Bureau of Reclamation (Reclamation) and other research facilities to determine if dynamic testing of a full scale cantilever concrete retaining wall is feasible and to then develop an appraisal-level design configuration for testing such a full scale model based on the capabilities of the identified testing facilities. An appraisal-level cost estimate for the proposed testing based on the developed design configuration was also completed and is presented in appendix C.

The final (third) part of this Scoping Phase 1 consisted of further development of Reclamation's own finite element capabilities specific to soil-structure interaction. Specifically, independent finite element models were created using both LS-DYNA [125] and FLAC [124] for purposes of comparison of results and independent validation. The results of these finite element studies will be documented separately in a report scheduled for completion as part of Planning Phase 2 in fiscal year (FY) 2010.

Additional phases of the project are planned in subsequent fiscal years with annual progression toward a full scale physical model test some time during FY2013. Subsequent project phases are briefly outlined below:

- Planning Phase 2—FY2010
    - Procurement of a practicing PhD research consultant
    - Cost-benefit analysis
    - Identification of funding sources for final design engineering
    - Identification of funding sources for construction
    - Documentation of recent Reclamation SSI finite element study results
- Feasibility Phase 3—FY2011
    - Laboratory site visit
    - Feasibility-level design development
    - Detailed cost estimates including reduced-scope testing alternatives
    - Draft testing procedure including static and dynamic load requirements as well as soil types and properties
    - Detailed instrumentation and data acquisition requirements
    - Pretest finite element analysis studies
- Final Design Phase 4—FY2012
    - Final design level plans and specifications
    - Final testing procedure including static and dynamic load requirements as well as soil types and properties
    - Final instrumentation and data acquisition requirements
    - Final design level bid schedule and cost estimate
    - Final contract documents and bid package
- Laboratory Testing Phase 5—FY2013
    - Contractor evaluation and procurement
    - Construction

- Full scale shake table tests and data collection
- Data postprocessing
- Posttest finite element analysis studies
- New design guidelines for design and evaluation of concrete cantilever retaining walls subjected to strong seismic ground motions
- Final report and closeout

Problem and Background

Reclamation has numerous spillways and other earth-retaining structures founded on soil and backfilled with various embankment materials, some in high seismic areas. Queries of Reclamation's spillway database indicate that there are approximately 35 gated spillway structures in Reclamation's inventory located in high seismic regions. Recent finite element analyses of spillway walls completed by Reclamation indicate dynamic loads on the walls due to seismic lateral earth pressures may be greater than or less than values predicted by traditional analytical methods, such as Mononobe-Okabe or Wood. This would depend on site-specific parameters such as foundation conditions and embankment alignment as well as assumed model parameters such as numerical energy source and boundary conditions. However, verifying the computed seismic earth pressures from various finite element computer programs such as FLAC [124] and LS-DYNA [125] has historically been problematic and unvalidated with physical model testing. Recent small scale centrifuge model tests completed by others and supported by numerical analyses suggest that dynamic earth pressures are substantially less than those predicted by the traditional analytical methods [7]. Further, seismic performance of retaining walls, with a few exceptions, has generally been satisfactory, even for wall systems originally designed with no consideration of seismic loading.

Additional complications exist due to the limitations of traditional analytical methods. Specifically, inherit assumptions of the Mononobe-Okabe or Wood methods often result in difficulties analyzing and designing soil-structure systems that are prevalent in, not only Reclamation's spillway inventory, but the infrastructure inventory of many other federal agencies, such as other Department of the Interior agencies, the United States Army Corps of Engineers, the Natural Resources Conservation Service, and the Federal Highway Administration. Specifically, the Mononobe-Okabe method is limited to small to medium ground accelerations requiring that the seismic inertial angle (ψ) be less than or equal to the backfill's angle of internal friction (φ). Wood's method is not limited by the magnitude of ground accelerations; however, Wood's method assumes a nonyielding wall system with no groundwater within the backfill and no soil cohesion. The existing field conditions for many of Reclamation's spillway walls are inconsistent with these fundamental assumptions.

Efforts to either validate historical analytical methods or to develop new guidelines for dynamic SSI have increased substantially over the last several decades. Studies including physical model testing in conjunction with numerical analyses are prevalent throughout the technical literature. Physical model testing has traditionally consisted of small scale model centrifuge testing with cohesionless sand backfill. The primary reason for focused efforts on scaled model testing is economic. Specifically, as presented herein, costs are nontrivial for developing and executing a full scale shake table test of a cantilever concrete retaining wall, and boundary effects associated with full scale testing are problematic. Conversely, small scale model centrifuge testing is far less expensive; however, sand backfill is exclusively used for such tests because of scaling effects associated with soil cohesion. Numerical analyses performed to verify physical model test results are often not completed independently of the model testing. Convenient dismissal of nonconforming physical model test data or massaging of numerical model boundary conditions at the soil-wall interface to validate research results is common practice. As a result, no industry standard guidelines or methodologies have been developed to supplant the traditional Mononobe-Okabe and Wood methods.

Experience has shown the importance of the dynamic SSI issue to Reclamation. Bradbury Dam is a good example of the potential far-reaching impacts regarding quantification of seismic lateral earth pressures. Bradbury Dam is a Reclamation-owned dam located on the Santa Ynez River approximately 25 miles northwest of Santa Barbara, California. The dam is a zoned earthfill structure, 279 feet high, with a crest length of 3,350 feet, and crest width of 40 feet at elevation 766.0. Between 1994 and 1995, dam safety modifications were constructed including downstream modifications of the embankment to eliminate the potential for failure of the dam due to earthquake-induced liquefaction of the foundation alluvium. In addition, the spillway crest structure was demolished and replaced with a new crest structure to accommodate the embankment modifications and to substantially increase seismic-load-carrying capabilities above those used for the original design in the early 1950s. The new crest structure was designed in the early 1990s using a deterministic approach consistent with standard engineering design practices at the time. As such, an effective pseudo-dynamic horizontal acceleration of 0.7g was used for the new crest structure design.

Since construction of the new crest structure was completed in 1995, earthquake engineering has evolved significantly and transitioned from a deterministic approach to a probabilistic approach. Identification, evaluation, and documentation of potential seismic sources have expanded exponentially with the progression of the Internet and the personal computer. As a result, the current probabilistic seismic hazard for Bradbury Dam is significantly greater than the hazard used for design and construction of the 1995 modifications. Specifically, the current seismic hazard data points include:

- 3,100-year event (3.2×10^{-4}) = 0.7g peak ground acceleration (PGA)
- 10,500-year event (9.5×10^{-5}) ≥ 1.0g PGA
- 50,000-year event (2.0×10^{-5}) = 1.6g PGA

Risk analyses recently completed using the current seismic hazard indicate that risks are above Reclamation's public protection guidelines [122], and a second rehabilitation of the spillway crest structure 15 years following construction of the first rehabilitation is very possible.

There are an estimated 35 gated spillway structures in Reclamation's inventory located in high seismic zones as part of embankment dam projects that could potentially require future remediation if seismic hazards continue to increase as they have over the past several decades. Several recent design modifications have incorporated extremely conservative, limit state approaches to dynamic SSI. Specifically, rehabilitation projects located in high seismic areas, such as Deer Creek Dam and Echo Dam, have utilized lateral earth loads based on static, limit state, passive, lateral earth pressure coefficients to avoid the limitations of traditional dynamic methods and conflicting results from dynamic numerical modeling. For a typical 15-foot high cantilever retaining wall with cohesionless backfill and a pseudo-dynamic horizontal acceleration of 0.6g, the ratio of total computed shear at the base of the wall stem using passive limit state pressures to that computed using the Mononobe-Okabe method (static active pressure plus dynamic pressure) is 1.8.

One approach to mitigate this problem is to determine if seismic lateral earth pressure loads resulting from increased seismic hazards are less than those determined using traditional methods or, in certain instances, less than those predicted by high numerical analyses. Completion of a full scale, concrete retaining wall, shake table test could provide valuable insight in this regard and potentially save millions of dollars in rehabilitation construction costs for Reclamation spillway walls.

This SSI research project, at the completion of all four phases, will help answer the questions that continue to arise as Reclamation is faced with detailed analysis and modifications of spillways and other earth-retaining structures in areas of increased or high seismicity. This research is for developing a better analytical tool to predict the seismic lateral earth pressures for configurations that include groundwater effects, cohesion effects, nonhorizontal zone effects and compaction/in-place density effects under various ground accelerations.

The project will likely culminate with a full scale model testing program to verify analytical results and either establish new evaluation and design standards for SSI or confirm criteria established by historical methods.

Literature Search and Review

The problem of retaining soils is one of the oldest in geotechnical engineering. Some of the earliest and most fundamental principles of soil mechanics were developed to allow rational design of retaining walls. Lateral earth pressures are those imparted by soils onto vertical or nearly vertical supporting surfaces of retaining structures. Two of the pioneers in the effort to quantify and evaluate lateral earth pressures were Coulomb and Rankine. Many others have since made significant contributions to our knowledge of static earth pressures; however, the work of Coulomb and Rankine was so fundamental that it still establishes the methodology for earth pressure calculations and retaining wall design today.

The magnitude of lateral earth pressures is mostly related to the movement of the retaining structure. Minimum (active) and maximum (passive) earth pressures occur when the retaining structure moves away from and into the soil mass, respectively. At-rest earth pressures fall in between these two extremes and occur when the retaining system does not move or is prevented from moving.

Coulomb [17] first studied the problem of lateral static earth pressures on retaining structures. He used force equilibrium to determine the magnitude of the soil thrust acting on the wall for the minimum active and maximum passive conditions. Known as the wedge theory, the analysis assumes a linear slip plane within the soil mass and assumes full mobilization of the shear strength along the assumed failure surface without considering the existing state of stress in the soil. Since the problem is statically indeterminate, a number of potential failure surfaces must be considered in the analysis to identify the critical failure surface. Interface friction between the wall surface and the backfill can be included as an analysis parameter.

Rankine [70] later developed a simpler procedure than Coulomb's wedge theory. Rankine's procedure makes simplifying assumptions about the stress conditions and the strength envelope of the soil mass behind the wall resulting in a statically determinate problem and, as a result, allowing for direct computation of the static earth pressures acting on retaining structures. Specifically, the Rankine theory of lateral earth pressures for active and passive conditions is used to estimate the state of stress within the soil mass that, because of displacement of the wall into (passive) or away from (active) the soil, is transformed from an at-rest state to a state of plastic equilibrium. Unlike the in Coulomb analysis, the orientation of the assumed linear slip plane within the soil mass can be computed directly because the problem is statically determinant. The Mohr-Coulomb shear strength relationship defines the shear stress along the slip plane at failure.

In addition to Coulomb's wedge theory and Rankine's theory, numerous other researchers and engineers have developed relationships for active and passive earth pressure, based on the assumption of a logarithmic failure surface. This logarithmic spiral procedure results in values similar to those of Coulomb's wedge theory and Rankine's theory for both active and passive earth pressures when the friction angle between the backfill and the wall surface (δ) is zero (see figure 1). For wall interface friction angles greater than zero, the wedge method and the logarithmic spiral procedure result in nearly the same value for active earth pressures. For all wall friction angle (δ) values, the logarithmic spiral procedure results in values reasonably similar to those of Rankine for passive earth pressures; however, the accuracy of the passive earth pressure computed using the wedge method diminishes with increasing values of wall interface friction (δ), due to the fact that the boundary of the failure block becomes increasingly curved [123].

The evaluation of seismically induced lateral earth pressures on retaining structures, however, represents a significantly more challenging problem than that of active and passive conditions. Specifically, retaining structures that performed satisfactorily for many years based on design practices consistent with Coulomb-Rankine lateral earth pressure theory did not perform as well during earthquakes. In fact, some earthquakes have permanently deformed retaining structures. In some cases, the observed deformations were small, while in others, deformations were significant causing serious damage. As a result, the engineering community recognized that properly designing earth-retaining systems for seismic loading had become essential.

The dynamic response of even the simplest type of retaining wall is a complex soil-structure interaction. Wall movements and dynamic earth pressures depend on the response of the soil underlying the wall, the response of the backfill, the inertial and flexural response of the wall itself, and the nature of the input motions. Seismic earth pressures are traditionally estimated from the Mononobe-Okabe (M-O) method (Okabe [63]; Mononobe and Matsuo [60]). However, the excellent performances of some retaining walls during major earthquakes, such as San Fernando (1971), Loma Prieta (1989), Northridge (1994), Kobe (1995), Chi-Chi (1999), Athens (1999), and Wenchuan (2008), have indicated that the M-O method may overestimate seismic earth pressures. This apparent disconnect between theory and field performance along with the extents and complexity of the problem has resulted in a significant amount of research over the past several decades regarding the problem of seismically induced lateral earth pressures. While many of these theoretical, experimental, and analytical studies have been completed on the subject of seismic earth pressures, to date, there seems to be no general agreement on a seismic design or evaluation method for retaining structures [7]. Given the importance of the seismically induced lateral earth pressure problem in the design of retaining structures in seismically active areas, it is curious that development of a standardized approach or industry-accepted guideline has not evolved from all the research efforts in this field of study.

The Scoping Phase of this study started with an extensive literature review of previous analytical, numerical, and experimental work related to dynamic earth pressures. The number of studies and publications on the topic is staggering. Google search results on the topic of soil-structure interaction illustrate this:

- "Soil AND structure AND interaction" found 432,000 results
- "Soil AND structure AND interaction AND seismic" found 87,000
- "Soil AND structure AND interaction AND seismic AND dams," 77,600

Refined Internet sources, such as deep web databases and Internet research center libraries, help reduce the overwhelming data hits on the subject, but the number of applicable papers, theses, presentations, and technical manuals on seismic earth pressures is still staggering. Over 100 technical papers and over 10 technical criteria manuals were collected for these Scoping Phase 1 studies and are included on a CD library (appendix A) to this report. Review of the literature was considered a vital first task in the Scoping Phase 1 to:

- Capture and summarize methods for evaluation and design of retaining walls subjected to seismic lateral earth pressure
- Evaluate and compare the vast number of studies that have recently been completed using numerical methods, typically involving finite element analyses
- Compare the scope, results, and conclusions of physical testing in regards to dynamic soil-structure interaction to determine if any previously completed testing applies to the specific problem of Reclamation spillway walls or if full scale shake table testing is justified

Assess the performance of existing retaining walls under seismic loading based on documented field observations compared with predicted performance based on historical design criteria.

Analytical Methods

As Stadler [85] and Sitar [7, 84] suggest, analytical solutions for the dynamic earth pressures problem can be divided into three broad categories depending on the magnitude of the expected wall deflection. These categories include rigid-plastic, elastic, and elasto-plastic methods. Relatively large wall deflections are usually assumed for rigid-plastic methods, while very small deflections are assumed for elastic methods. Elasto-plastic methods, appropriate for moderate wall deflections, are usually developed using finite element analyses and are therefore presented under the numerical methods section of this report.

It is important to note that, due to the significant number of variables and parameters involved with dynamic soil-structure interaction, analytical methods are generally based on idealized assumptions and simplifications that do not necessarily represent the real seismic behavior of the retaining structure-soil mass system. Therefore, many researchers believe that analytical methods often result in overly conservative estimates of dynamic earth pressures [7, 84].

Rigid-Plastic Methods

Rigid-plastic methods generally assume large wall deflections and are either force based or displacement based. The most commonly used force-based rigid-plastic methods are the M-O and Seed and Whitman [79] methods. Displacement methods are generally based on the Newmark [62] or modified Newmark sliding block.

Okabe [63], in 1924, and Mononobe and Matsuo [60], in 1929, performed the pioneering work for dynamic soil-structure interaction in Japan following the Great Kwanto Earthquake. The method proposed by these researchers is known as the Mononobe-Okabe (M-O) method and, along with its derivatives, has generally been considered the most commonly used approach to determine seismically induced lateral earth pressures. The M-O method is based on Coulomb's wedge theory of static lateral earth pressures and was originally developed for gravity walls retaining cohesionless backfill.

The M-O theory includes the effects of earthquakes through the use of constant horizontal acceleration and constant vertical acceleration, expressed as a fraction of the acceleration of gravity. The acceleration is assumed to act on the soil mass comprising Coulomb's active or passive wedge. The M-O theory further assumes that wall movements are sufficient to fully mobilize the shear resistance along the backfill wedge slip plane, which is consistent with the Coulomb wedge theory.

The M-O theory computes the net static and dynamic force acting on the retaining structure. For positive horizontal accelerations (soil accelerates toward the wall), the net dynamic active force (P_{AE}) is greater than the net static active force (P_a), and the net dynamic passive force (P_{PE}) is less than the net static passive force (P_p). Thus, compared with static conditions, the seismic earth pressures increase from the driving side soil mass and decrease from the resisting side soil mass. The M-O force diagram is presented in figure 1. A limitation of the M-O method in higher seismic regions is that the soil angle of internal friction (φ) must be greater than the seismic inertial angle (ψ), which is a function of the horizontal acceleration. The M-O equations yield negative radicals (complex numbers) under such large seismic accelerations. A summary of the fundamental M-O assumptions is presented below (Seed and Whitman [79]):

- The wall yields sufficiently when subject to active pressures as shown in table 1.
- The backfill is cohesionless.
- The soil is assumed to satisfy the Mohr-Coulomb failure criterion.
- When the minimum active pressure is attained, a soil wedge behind the wall is at the point of incipient failure, and the maximum shear strength is mobilized along the potential slip plane.
- Failure in the backfill occurs along a slip plane surface that is inclined at some angle with respect to the horizontal backfill passing through the toe of the wall.
- The soil wedge behaves as a rigid body, and accelerations are constant throughout the mass.
- Equivalent static horizontal and vertical forces, Wk_h and Wk_v, are applied at the center of gravity of the wedge and represent the earthquake forces. Parameters k_h and k_v represent gravitational accelerations in the soil wedge.
- Liquefaction is not a consideration for the backfill.
- The backfill is completely above or completely below the water table, unless the ground surface is horizontal, in which case the backfill can be partially saturated.
- The ground surface is planar, not irregular or broken.
- Any surcharge is uniform and covers the entire soil surface.
- The soil angle of internal friction must be greater than the seismic inertial angle—$\varphi \geq \psi$ (nonsloping backfills).

Table 1.—Required wall movements for development of M-O pressures

	Values of y/H	
Backfill type	Active	Passive
Dense sand	0.001	0.01
Medium dense sand	0.002	0.02
Loose sand	0.004	0.04

Notes: y = horizontal displacement at the top of the wall; H = height of wall

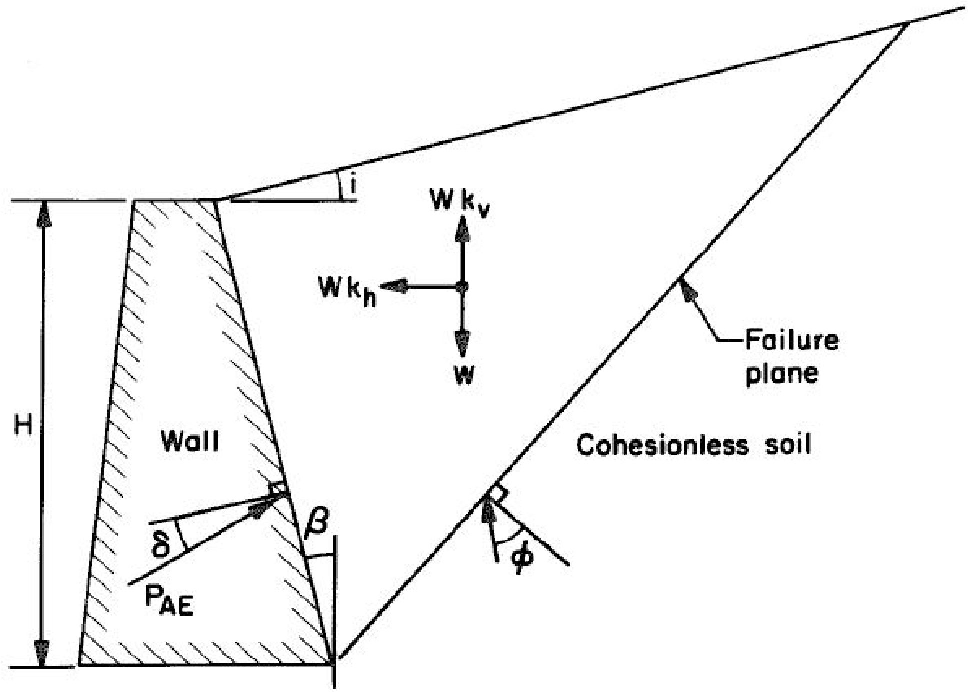

Figure 1.—Forces considered in Mononobe-Okabe analysis [7].

Based on the M-O method, the active lateral thrust can be determined by the static equilibrium of the soil wedge shown in figure 1. The maximum dynamic active thrust per unit width of the wall, P_{AE}, is determined by optimizing the angle of the failure plane to the horizontal plane, and is given by:

$$P_{AE} := \frac{1}{2}\cdot\gamma\cdot H^2\cdot(1 - k_v)\cdot K_{AE}$$

$$K_{AE} := \frac{\cos(\phi - \psi - \beta)^2}{\cos(\psi)\cdot\cos(\beta)^2\cdot\cos(\delta + \beta + \psi)\left(1 + \sqrt{\dfrac{\sin(\phi + \delta)\cdot\sin(\phi - i - \psi)}{\cos(\delta + \beta + \psi)\cdot\cos(i - \beta)}}\right)^2}$$

where,

P_{AE} = maximum dynamic active force per unit width of the wall
K_{AE} = total lateral earth pressure coefficient
γ = unit weight of the soil
H = height of the wall
φ = angle of internal friction of the soil
δ = angle of wall friction
i = slope of ground surface behind the wall
β = slope of the wall relative to the vertical
k_h = horizontal wedge acceleration divided by g

k_v = vertical wedge acceleration divided by g

$$\psi = \operatorname{atan}\left(\frac{k_h}{1-k_v}\right)$$

The M-O method gives the total active thrust acting on the wall but does not explicitly give the point of application of the thrust or the dynamic earth pressure distribution. The point of application of the M-O active thrust is assumed to be at H/3 above the base of the wall.

Seed and Whitman [79] performed a parametric study to evaluate the effects of changing the angle of wall friction (δ), the friction angle of the soil (φ), the backfill slope (β) and the vertical acceleration (k_v) on the magnitude of dynamic earth pressures. The results of the Seed and Whitman studies concluded that the maximum total earth pressure acting on a retaining wall (P_{AE}) can be divided into two components: the initial static active pressure (P_a) and the dynamic increment due to the seismic base motion (ΔP_{AE}). As a result, Seed and Whitman [79] suggested that the static, incremental dynamic, and total lateral earth pressure coefficients for dynamic soil-structure interaction are related as follows:

$$K_{AE} = K_a + \Delta K_{AE}$$

where the dynamic earth pressure increment coefficient ΔK_{AE} is approximately equal to ¾ k_h for the case of a vertical retaining wall with horizontal backfill slope and a soil internal friction angle of 35°. Other significant findings of the Seed and Whitman [79] studies included:

- The determination that the point of application of the dynamic incremental thrust force (ΔP_{AE}) should be between one half to two thirds the wall height above the base

- The peak ground acceleration occurs for an instant and does not have sufficient duration to cause significant wall movements. As a result, a recommended reduced ground acceleration of about 85% of the peak value should be used for the seismic design of retaining walls.

- Many walls that are adequately designed exclusively for static earth pressures will automatically have the capacity to withstand earthquake ground motions of substantial magnitudes, and, in many cases, special seismic earth pressure provisions may not be required for satisfactory performance during significant seismic events.

Displacement-based methods have generally been developed for gravity retaining walls and are typically based on the Newmark [39] and modified Newmark sliding block model. The displacement-based methods involve calculating an acceleration coefficient value based on the amount of permissible displacement of the wall. This reduced acceleration coefficient is then used in conjunction with the M-O method to determine the dynamic thrust. Wall inertial effects are usually accounted for in displacement-based methods based on determination of the acceleration coefficient value. Richards and Elms [72] observed that inertial forces on gravity retaining walls can be significant and concluded that the M-O method provides adequate estimates of seismic earth pressures provided that wall inertial effects are properly accounted for as a separate load. Other examples of such methods are Zarrabi [105] and Jacobson [43].

Elastic Methods

Elastic methods were originally developed and applied for the design of basement walls that would be expected to experience very small displacements under seismic loading and, as such, can be considered as rigid, nonyielding walls. The fundamental assumption for the elastic methods is that the relative soil-structure displacement generates soil stresses in the elastic range of the material. Elastic methods are usually based on elastic wave solutions and are thought to represent upper-bound dynamic earth pressures and, as a result, produce seismic loads greater than those of the M-O method. Wood's method [99] is the most widely accepted and widely used method under the category of elastic methods. Other elastic method studies include Matsuo and Ohara [59], Tajimi [91], and Scott [77].

Wood's method is based on linear elastic theory and on idealized representations of the wall-soil structural system [7]. Wood performed an extensive study on the behavior of rigid retaining walls subject to earthquake loading and provided chart solutions for the cases of arbitrary horizontal forcing of the rigid boundaries and for a uniform horizontal body force. Wood determined normal mode solutions for the cases of uniform soil modulus and soil modulus varying with depth. Wood's solutions are slowly convergent for practical problems and, as a result, Wood presented approximate procedures based on findings from the normal mode solutions. The normal stress distributions along the back of the wall were found to be related to Poisson's ratio and the lateral extent of the backfill behind the wall. Wood's method predicts a total dynamic thrust approximately equal to $2\gamma Hk_h$ acting at a height equal to 0.58H above the base of the wall. Figure 2 presents Wood's formulation for the case of a uniform horizontal body force.

Numerical Methods

Numerical modeling efforts have been applied to verify the seismic design methods in practice and to provide new insights into the problem of dynamic SSI. Various assumptions have been made, and several numerical codes have been used (e.g., PLAXIS, FLAC, SASSI, LS-DYNA, and GT STRUDL) in an attempt to further evolve the solution to the problem. While elaborate finite element techniques and constitutive models are available in the literature to determine the seismic lateral soil pressure for design, simple methods for quick prediction or prescriptive development of dynamic soil pressures are rare [7]. Moreover, while some of the numerical studies reproduced experimental data quite successfully; independent predictions of the performance of retaining walls are not available [7]. Specifically, most studies consist of performing small scale

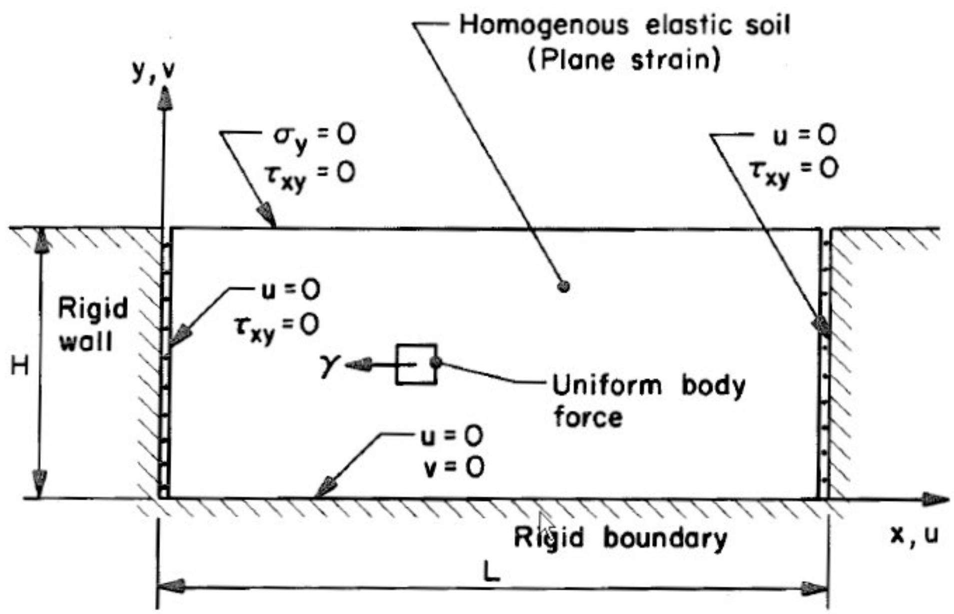

Figure 2.—Wood [99] rigid problem.

centrifuge model testing followed by a detailed finite element study, the purpose of which is to match the results of the physical testing. With the sophistication of current finite element codes in terms of nonlinear contact surface and nonlinear material types, changing parameters to match physical results is not uncommon. This is especially true at the soil mass-concrete interface. Another technique used to declare one-to-one correspondence between physical testing results and finite element results is selective sampling of physical data. Selective sampling consists of removing or ignoring physical test data that do not fit well or match up with numerical results. Hence, the predictive capability of the various finite element studies is not clear. While discussion of all numerical studies completed in the field of dynamic SSI would be impossible due to the vast number of studies, selected studies on numerical methods in the field of dynamic SSI are summarized in the following paragraphs.

Clough and Duncan [14] performed the first true numerical study using finite element methods to model static SSI behavior where the interface effects between the structure and the surrounding soil were included. Wood's studies [99] included modeling a rigid retaining wall–soil system using linear plane-strain elements and found a favorable comparison of the numerical results with analytical calculations for rigid walls. Aggour and Brown [2] conducted two-dimensional plane-strain finite element analyses on a 20-foot high cantilever retaining wall to study the effects of wall flexibility and backfill geometry on the dynamic earth pressure distribution. They concluded that greater wall flexibility reduces the total dynamic moments acting on the retaining structure and that the shape of the backfill has considerable effects on the frequencies of the soil-structure system [7].

Siddharthan and Maragakis [83] conducted finite element analyses to model the dynamic behavior of a flexible retaining wall supporting dry, cohesionless soil. They modeled soil nonlinear hysteretic behavior and validated their model by comparing results to recorded responses from a dynamic, small scale, model centrifuge experiment. Their studies concluded that high bending moments and low wall deflections occur for stiff retaining walls supporting loose, sandy backfills.

Steedman and Zeng [87] proposed a pseudo-dynamic numerical model for seismic earth pressures, taking into account dynamic amplification and phase shifting, and validated their model with results from a small scale model centrifuge experiment. They concluded that the dynamic thrust component (ΔP_{AE}) acts approximately one-third above the base of the wall. Veletsos and Younan [92] modeled flexible retaining walls and concluded that forces acting on flexible walls are much lower than those acting on rigid ones.

Green *et al.* [31] performed a series of nonlinear dynamic response analyses of a cantilever retaining wall–soil system using the FLAC computer code [124] and concluded that at very low levels of acceleration, the seismic earth pressures were consistent with those predicted by the M-O method. However, as accelerations were increased, seismic earth pressures were greater than those predicted by the M-O method [7]. This deviation was attributed to the flexibility of the retaining wall system and to the observation that the driving soil wedge does not respond monolithically, but rather responds as several wedges [31]. Gazetas *et al.* [28] performed a series of finite element analyses on several types of flexible retaining systems subjected to brief, moderately strong excitations. They concluded that as the degree of *realism* in the numerical model increased, the results were consistent with the frequently observed satisfactory performance of retaining systems during strong seismic shaking [7].

To investigate the characteristics of the lateral seismic soil pressure on building walls, Ostadan [67] performed a series of dynamic SSI analyses using the computer code SASSI [126] that resulted in predicted seismic earth pressures comparable to or greater than Wood's method, with the maximum earth pressure occurring at the top of the wall.

Madabhushi and Zeng [57] of Cambridge University performed finite element analyses of a gravity wall and a flexible cantilever retaining wall with dry and saturated backfills subjected to seismic loads using the computer code SWANDYNE developed by Chang [10]. The analysis results were verified by small scale model dynamic centrifuge tests. It was found that the total bending moments on the walls significantly increased above those predicted by the M-O method during seismic loading, and the residual moments after the earthquake were much higher than the static bending moments before the earthquake. Further, the effect of the seismic loading was much more severe for a cantilever retaining wall with saturated backfills than that of a cantilever retaining wall with dry backfills. Ling *et al.* [53] conducted parametric studies at Columbia University on the behavior of reinforced soil retaining walls under seismic loading, using a modified version of DIANA-SWANDYNE-II [127].

Reclamation has recently completed several numerical studies using the finite element software LS-DYNA [125] and FLAC [124]. Specifically, different researchers independently developed two finite element models, one using LS-DYNA and the other using FLAC. Time history analysis results compared favorably between the two models and resulted in dynamic soil loads consistent with those predicted by Wood's method but greater than those predicted by the M-O method. The results of these finite element studies will be documented separately in a report scheduled for completion as part of Planning Phase 2 in FY2010. Additional finite element evaluations by Reclamation are on-going as part of project work for Bradbury Dam, Echo Dam, and Scoggins Dam.

In summary, the numerical approaches in commercial software, such as FLAC [124], LS-DYNA [125] and SASSI [126], generally provide more conservative seismic earth pressures than the widely accepted M-O method. This is primarily due to the conservative assumptions and constitutive soil models embedded in the commercial software [7]. However, the variability in results is significant, and general convergence of solutions resulting in a development of a new simplified method of design or evaluation is unlikely due to the variability of material models, contact surface algorithms and boundary condition assumptions inherent to all finite element studies.

Physical Testing

The physical testing in the field of seismic earth pressures on retaining walls can be divided into the four groups: small scale, 1g shake table tests, dynamic small scale centrifuge tests, medium to large scale shake table tests, and field tests.

Small Scale, 1g Shake Table Tests

A significant aspect to development of the widely accepted M-O method was the first known physical experiments that Mononobe and Matsuo performed in Japan for investigating seismic earth pressure on retaining walls [60]. The experiments used a rigid, small scale, 1g shake table. The 9-foot long by 4-foot wide by 4-foot or 6-foot high sand boxes that contained the relatively loose and dry sands were set on rollers as shown on figure 3. A winch driven by a 30-horsepower electric motor provided the horizontal simple harmonic motion of the shake table, and the hydraulic pressure gauges at the top of the sand boxes measured the resulting seismic earth pressures.

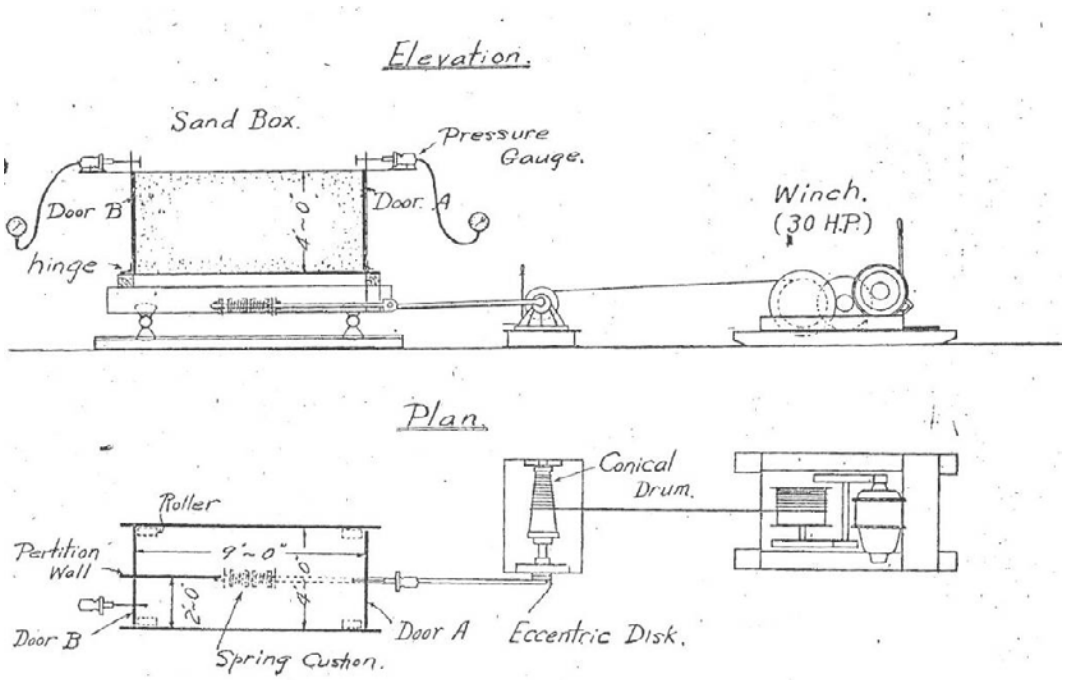

Fig. 6.

Figure 3.—Setup of Mononobe and Matsuo [60] experiments.

Matsuo [58], Ishii *et al.* [41], Matsuo and Ohara [59], Sherif *et al.* [82], Bolton and Steedman [8], Sherif and Fang [81], Steedman [86], Ishibashi and Fang [40], and others performed similar experiments using small scale, 1g shake tables. Although all of these experimental results generally supported the predictions of the M-O method, other researchers, such as Atik and Sitar [7], have identified the limitations of the small scale, 1g shake table. One limitation is the inability to replicate in-situ soil stress conditions, especially for granular backfills.

Dynamic Small Scale Centrifuge Tests

Bolton and Steedman [8, 9] and Steedman [86] conducted dynamic small scale centrifuge tests at Cambridge University in England on concrete and aluminum cantilever retaining walls supporting dense and dry sands. During the same time frame, Ortiz [66] at Caltech performed a series of dynamic small scale centrifuge tests on cantilever retaining walls with medium dense and dry sands. Although most of these dynamic small scale centrifuge test results generally supported the magnitude of total lateral earth force predicted by the M-O method, conclusions regarding the location of the resultant above the base of the wall varied from one-half to one-third of the wall height, instead of the two-thirds wall height level predicted by the M-O method. Steedman and Zeng [87, 88] and Zeng [105] suggested that, based on their dynamic small scale centrifuge tests, the dynamic amplifications or attenuations of the ground motions and the phase shifting between the wall and supported backfills play an important role in determination of the magnitudes and distributions of seismic earth pressures.

Stadler [85] and Dewoolkar *et al.* [18] at the University of Colorado performed a series of dynamic centrifuge tests on cantilever retaining walls supporting dry, medium dense sands, and saturated, liquefiable cohesionless backfills, respectively. The important conclusions from their research are summarized as:

- Excess pore pressures in liquefiable backfills contribute significantly to seismic earth pressures.
- Maximum dynamic thrust is a linear function of the magnitude of the average base shaking acceleration.
- If the backfills liquefy completely, the residual thrust at the end of shaking is independent of the flexural stiffness of the wall and is about 50 percent of the active static thrust.
- The distribution of total lateral earth pressures is approximately triangular, but the distribution of seismic earth pressures varies from up-triangular to down-triangular. The line of action of total lateral earth pressures varies between 0.6 and 0.8 of the wall height during shaking.
- Flexible walls may experience significant bending stresses due to inertia effects of the wall, in addition to the stresses caused by the changes in total lateral earth pressures.

Moreover, Stadler [85] suggested that the seismic acceleration coefficients used in the M-O method could be reduced by 20 to 70 percent of the horizontal peak accelerations to match the experimental results from the dynamic small scale centrifuge tests. This particular conclusion is consistent with the conclusions of Whitman [94].

Nakamura [61] of the University of California at Davis and Al Atik [7] of the University of California at Berkeley have made recent advances in dynamic centrifuge tests on gravity retaining walls as shown in figure 4. These more recent dynamic small scale centrifuge tests of flexible cantilever retaining walls supporting granular, dry, medium dense soils with level ground, and nonliquefiable backfill concluded:

- Contrary to the M-O method rigid wedge assumption, the part of the backfill that follows the movement of the retaining wall deforms plastically when sliding down.
- Also contrary to the M-O method, peak inertial forces from the wall and forces from the backfill occur at different times. That is, the seismic acceleration is transmitted instantaneously through the wall and then transmitted into the backfill.

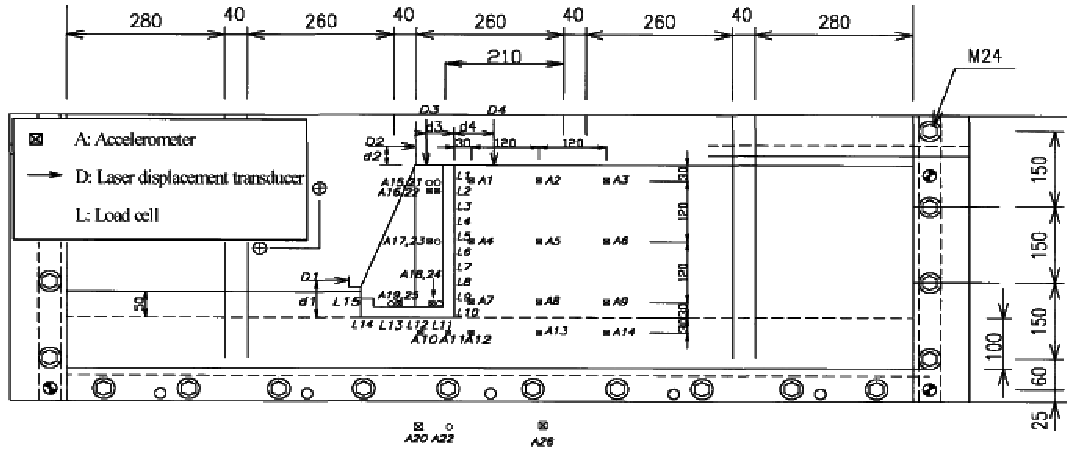

Figure 4.—Nakamura [61] centrifuge model, horizontal shaking direction (in mm).

- Seismic accelerations are not uniform on the wall or in the backfill.

- Seismic earth pressures are approximately zero when the inertial force of the wall is at its maximum, which indicates that the seismic earth pressure and inertial force are not in phase.

- Maximum seismic earth pressure increases with depth and can be reasonably approximated by a triangular distribution.

- The traditional M-O and the Seed and Whitman [79] methods overestimate the magnitude of the maximum induced seismic earth pressure.

Medium to Large Scale Shake Table Tests

While the literature review completed as part of this Scoping Phase 1 did not identify any previously documented medium to large scale shake table tests consistent with field configurations of Reclamation spillways, noteworthy medium to large scale dynamic SSI studies performed for other earth-retaining systems are summarized in the following paragraphs for completeness and future reference.

Richards [72] pioneered medium to large scale shake table testing of reinforced earth-retaining walls. Futaki *et al.* [106, 107] performed large scale shake table tests on a 20-foot high reinforced earth-retaining wall and a 16-foot high multi-anchor earth wall. Richards *et al.* [73] used a shake table of plan dimensions 20 feet by 10 feet at the State University of New York at Buffalo to verify the predicted threshold accelerations for seismically induced rotations or sliding of a small scale gravity-wall bridge abutment. In general, large scale shake table tests on reinforced soil-retaining walls appears to be prevalent in the literature as identified by Ling [53] who summarized the work of several researchers completing shake table tests for reinforced earth systems.

Hazarika *et al.* [36] performed a series of large scale underwater shake table tests on two gravity type model caissons: one with conventional sandy backfills (Case A), and another with tire chips cushion protection (Case B). The three-dimensional, 18.5-foot diameter underwater shake table at the Port and Airport Research Institute in Japan was installed in a 50-foot by 50-foot by 6.5-foot deep pool of water. It was observed that the incremental dynamic earth pressures acted in the opposite direction of the inertia forces for Case B (i.e., phase shifting) but in the same direction for Case A. This resulted in measured, horizontal, seismic earth pressures that were significantly less for Case B than Case A. Moreover, the seismic earth pressures in Case A did not stabilize immediately at the end of loading sequence due to residual earth pressures, but the seismic earth pressures did stabilize immediately at the end of loading in Case B. This implied that the residual earth pressures in Case B were much less than those in Case A.

Kagawa *et al.* [45] compared dynamic centrifuge tests with large scale shake table tests for three case studies completed for soil-pile-structure systems. The large one-dimensional shake table of plan dimensions 50-feet by 50-feet at the National Research Institute for Earth Science and Disaster Prevention in Japan was used. It was concluded that the two types of tests yielded comparable results for these case studies but also that key differences exist. For example, excess pore-water pressures were developed and redistributed differently in the two different test setups.

Field Tests

Several noteworthy free and forced vibration dynamic SSI tests have been conducted in the field. For example, Amano *et al.* [110] conducted free and forced vibration field tests on a pier at the Port of Kobe. Aliev *et al.* [109] investigated the effects of foundation soils on seismic earth pressures on retaining walls by performing field tests. Fukuoka and Imamura [108] used load cells to measure seismic earth pressures on a cantilever retaining wall and on a large concrete-block retaining structure in the field. Although quantifiable results were inclusive, the qualitative conclusions from these field test studies are:

- Retaining walls supported on rock foundations were determined to be more vulnerable to seismic damage.

- Retaining wall failures were attributed to the combined inertial effects of the wall body and increased seismic earth pressure of the backfills behind the walls.

Chang *et al.* [10] reported the measured seismic earth pressures on the embedded walls of the quarter-scale model reactor containment structure during several moderate earthquakes in Lotung, Taiwan. These measured seismic earth pressures were similar to or lower than those estimated by the Mononobe-Okabe method.

Elgamal *et al.* [20] at Rensselaer Polytechnic Institute conducted a full scale vibration test to measure the low amplitude, dynamic characteristics of a reinforced concrete cantilever wall-backfill system. An eccentric-mass shaker with a capacity of providing up to 5.0 kips of horizontal shaking force within the frequency range from 0.5 to 30 Hz was used to vibrate a 140-foot (length) × 14-foot (height) × 16-in. (thickness) concrete retaining wall. It was observed that the wall and adjacent backfill zone generally moved together in phase. Farther away from the wall, out-of-phase wall-backfill motions were measured at higher resonant frequencies. Moreover, the reinforced concrete wall behaved more like a clamped cantilever plate than a cantilever beam under dynamic loadings.

While several field tests have been performed to evaluate dynamic earth pressures, results have generally been limited to qualitative observations with inconclusive results in terms of quantifiable data. A field test on a Reclamation spillway wall using either Reclamation's eccentric-mass shaker or the University of Texas' eccentric mass shaker would be expected to have similar results. Specifically, introduction of dynamic loading from the top of a spillway wall does not represent conditions during an earthquake with the dynamic source imparted to the base of the wall instead of the top of the wall. Difficulties with proper instrumentation and data collection due to the limited access to the entire height of the wall stem are another limitation to this field test approach.

Field Performance

According to Atik and Sitar [7], limited information is available on the field performance of retaining structures in recent major earthquakes due to the lack of well documented failures of retaining structures in nonliquefiable backfills. As discussed in Gazetas *et al.* [28], the performance of retaining structures and basement walls during earthquakes greatly depends on the presence of liquefaction-prone, loose, cohensionless backfills. Case histories from recent major earthquakes (such as San Fernando in 1971, Loma Prieta in 1989, Northridge in 1994, Kobe in 1995, Chi-Chi in 1999, Kocaeli in 1999, and Athens in 1999) show that retaining structures supporting loose, saturated, liquefiable, cohesionless soils are quite vulnerable to strong seismic shaking. On the other hand, flexible retaining walls supporting dry cohesionless sands or saturated clayey soils have performed particularly well during earthquakes. It is important to note that some of these retaining walls were not designed for seismic loading and that others were designed for base accelerations not more than 20 percent of the peak accelerations that they actually experienced during actual earthquakes. Selected case histories describing the behavior of retaining structures with nonliquefiable backfills are presented in this section.

Clough and Fragaszy [15] investigated the seismic performance of open channel floodway structures in the Greater Los Angeles area during the 1971 San Fernando earthquake. The floodway structures studied consisted of open U-shaped channels with wall tops set flush to the ground surface as shown in figure 5. The backfill soil consisted of dry, medium dense sand with an estimated friction angle of 35 degrees. No seismic load cases were included in the design. The cantilever walls were damaged during the earthquake, with the typical mode of failure as shown in figure 5. From the pseudo-static analyses (i.e., the Mononobe-Okabe method) and shear-wave propagation field tests, it was concluded (as shown in figure 6) that ". . . conventional factors of safety used in design of retaining structures for static loadings provide a substantial strength reserve to resist seismic loadings. The floodways sustained peak accelerations of up to 0.5g with no damage even though no seismic loads were explicitly considered in the design." The relationship between wall damage and ground acceleration obtained by Clough and Fragaszy [15] is shown in figure 6.

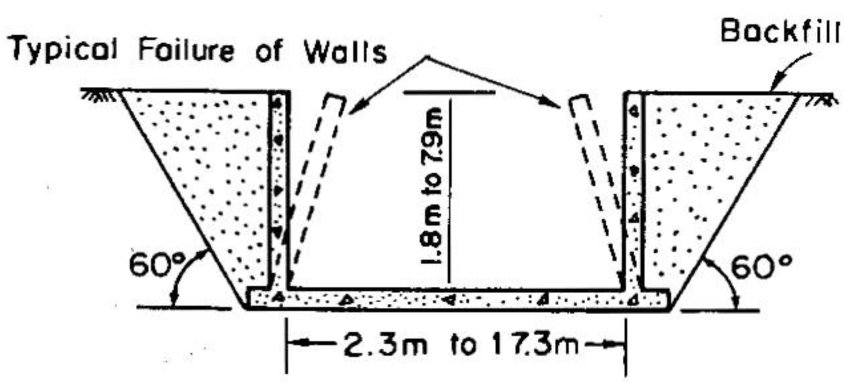

Figure 5.—Section through open channel floodway and typical mode of failure due to earthquake shaking [15].

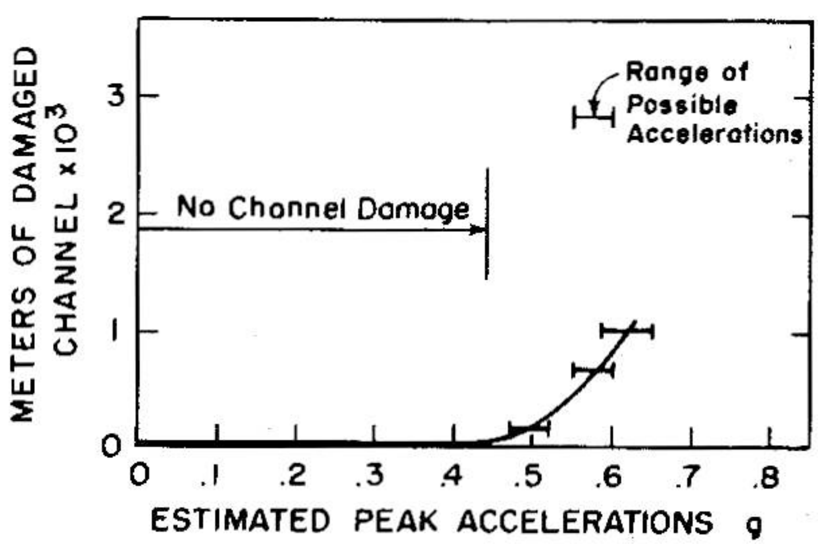

Figure 6.—Relationship between channel damage and peak accelerations [15].

During the 1995 magnitude 7.0 Kobe earthquake in Japan, many types of retaining structures, mostly located along railway lines, were subjected to strong ground motions. Gravity-type retaining walls such as masonry, unreinforced concrete, and leaning types were heavily damaged. On the other hand, reinforced concrete walls experienced only limited damage.

Koseki *et al.* [48] presented preliminary evaluations of the internal and external stability of several damaged retaining walls during the Kobe earthquake. The aim of their study was to improve the current design procedures that are mostly based on the M-O theory. Similarly to the suggestion from the dynamic centrifuge tests of Stadler [85], it was concluded that the horizontal seismic acceleration coefficients used in the Mononobe-Okabe method could be reduced up to 60 percent of the measured peak horizontal acceleration.

During the 1999 magnitude 7.6 Chi-Chi earthquake in Taiwan, flexible reinforced concrete walls and reinforced-soil retaining walls performed well. Ling *et al.* [52] studied cases of modular-block, geosynthetic-reinforced, soil retaining walls and reinforced slope failures during the Chi-Chi earthquake. They attributed part of these failures to the topography and geotechnical conditions in Taiwan, whereby many walls are located along slopes and mountains, and most walls that performed poorly were constructed with obvious lack of professional design oversite.

Fang *et al.* [25] investigated three gravity retaining walls damaged in the same Chi-Chi earthquake. The failure modes of the retaining walls included (a) shear failure of the wall stem base due to the insufficient frictional resistance at the untreated construction joint between the footing and the wall stem; (b) global bearing capacity failure of the foundation resulting in excessive settlements and tilting about the wall toe; and (c) overturning and subsequent global sliding failure of the wall due to a fault rupture underneath the footing. In all three cases, the forensic investigations concluded that the wall failures were the result of fundamentally unsound engineering practices of the designers or lack of quality control in the field during construction.

Gazetas *et al.* [28] reported that during the 1999 magnitude 5.9 Athens earthquake in Greece, several subway stations were under construction. Although the retaining structure of the Kerameikos Metro Station was not designed for any seismic loads, it was able to withstand nearly 0.5g of peak ground acceleration during the earthquake with no visible damage. Maximum wall displacements were estimated to have been on the order of a few centimeters.

Most recently, observations of the seismic responses of retaining walls during the Wenchuan earthquake (2008) in China showed excellent performance of all types of retaining structures [6, 7]. It should be emphasized that these retaining structures were designed for seismic events much weaker than the magnitude 7.9

earthquake, but no evidence of significant damage was observed in postearthquake investigations.

On the contrary, instances have been documented of retaining wall failures during seismic events. For example, figure 7 shows the failure of a chute wall panel in the spillway for Shi-Kang Dam during the 1999 magnitude 7.6 Chi-Chi earthquake in Taiwan. The wall is counterforted, and it appears that the collapse of the wall resulted from a shear failure in the counterforts. No specific details are available regarding the failure or regarding the design and quality control practices used during construction of the wall, but the figure does demonstrate the potential for failure of spillway walls during large earthquakes.

In summary, postearthquake investigations on the field performances of existing retaining walls indicate that the retaining walls supporting loose and saturated liquefiable backfills are quite vulnerable to strong ground motions, but, in general, flexible retaining walls supporting dry sands or saturated clays have performed very well during major earthquakes over the past decades, even if these flexible retaining walls were originally designed for no seismic loads at all.

Figure 7.—Failed spillway wall at Shih-Kang Dam.

Laboratory Facilities

Literature review results suggest that there is a significant knowledge gap in the understanding of dynamic SSI that could be filled with completion of a full scale shake table test of a cantilever concrete retaining wall. Recent studies do not appear to converge on both numerical results and small scale centrifuge tests.

Specifically, supporting numerical analyses are almost never completed independent of dynamic small scale centrifuge tests, and conclusions vary significantly from one study to the next. Field testing has proven inconclusive in terms of obtaining useful quantifiable results. A full scale shake table test could help considerably in filling this knowledge gap that has existed for decades. As such, the second part of these Scoping Phase 1 studies consisted of evaluating laboratory capabilities at Reclamation and other facilities around the country to determine the feasibility of performing dynamic tests of a full scale cantilever concrete retaining wall.

Reclamation Shake Table

In the late 1990s, Reclamation's Materials Engineering and Research Laboratory (MERL) designed and constructed a 15-foot by 20-foot shake table for the primary purpose of performing shake table tests on small scale concrete gravity and arch dam models. Unfortunately, the shake table was dismantled and the steel framing sold as surplus in 2005 shortly after completion of the dam model tests. Additional steel could be purchased and fabricated into a new shake table test fixture to be used along with the remaining actuators if funds were available to do so; however, the size of the shake table in the MERL would limit the practical size of a cantilever retaining wall test section to significantly less than the 15-foot high wall section being proposed for this research project. Specifically, the the backfill behind a full scale retaining wall consistent with Reclamation's spillway inventory would, at a minimum, need to extend well beyond the intersection of the anticipated soil failure wedge slip plane and the ground surface to properly simulate field conditions and limit boundary effects. In addition to size limitations of the shake table, the MERL is not currently furnished with data acquisition resources to the extent that would be required for such a full scale test. Rather, a well organized network of existing facilities specializing in earthquake engineering and simulation would be better suited for a full scale retaining wall test.

Network for Earthquake Engineering Simulations

The Network for Earthquake Engineering Simulations (NEES) is a network of 15 experimental facilities created and funded under an all-encompassing earthquake research project funded by the National Science Foundation (NSF). The developmental phase of the NEES extended from 2000 to 2004 and consisted mainly of construction of new earthquake engineering laboratories or the significant enhancement of existing facilities. In addition, a sophisticated network of information technology infrastructure was developed to establish data repositories, to allow for remote operation using telepresence tools, and to develop simulation software for specific test projects. The NSF provided approximately $80 million for the development phase of the NEES. Completion of the development phase in 2004 culminated with formation of the NEES

Consortium, Inc., as the governing, nonprofit body of the NEES. The mission statement of the NEES Consortium is simply to improve understanding of earthquakes and their effects.

The operational phase of the NEES is ongoing and consists mainly of collaboratory research projects in addition to upgrading and maintenance of equipment sites for execution of laboratory tests. The NSF funds NEES research projects through a competitive process of peer-reviewed research proposals. The operational phase of the NEES started in 2005 and is fully funded through the year 2014. The NSF budgets approximately $30 million a year for the NEES, including $20 million for additional equipment, construction, and upgrades plus $10 million for research projects. One hundred thirty research projects are in progress at NEES facilities. An estimated one quarter of these research projects are SSI related; however, none of them are specific to cantilever concrete retaining walls of the size and type of Reclamation's spillway inventory. Organizations sponsoring NEES research projects include CalTran, the Pennsylvania Department of Transportation, and the U.S. Geological Survey. After the current operational phase expires in 2014, NEES expects additional funding for continued sponsorship of earthquake engineering research projects, but probably with a reduced network of facilities.

Accepted NEES proposals include funding through the NSF for the use of most services and equipment that accompany the identified facility to be used for the proposed research project with a few exceptions. Specifically, any required modifications to existing facilities or equipment to meet specific project needs would require separate funding by the sponsoring organization or principal investigator. Construction of test specimens or experiment-specific fixtures would also require separate funding sources from the sponsoring organization. The only other requirement for receiving NSF funding for use of the identified NEES facility is that the data and results of any NEES research project must be shared with the technical community via the NEES data repository.

A list of the 15 NEES facilities is shown in table 2.

Table 2.—The 15 NEES facilities

Location	Description
Brigham Young University	Permanently instrumented field sites for study of soil-foundation-structure interaction
Cornell University	Large displacement soil-structure interaction facility for lifeline systems
Lehigh University	Real time multi-directional testing facility for seismic performance simulation of large scale structural systems
Oregon State University	Oregon State multi-directional wave basin for remote tsunami research
Rensselaer Polytechnic Institute	Upgrading, development, and integration of next generation earthquake engineering experimental capability at Rensselaer's 100-g-ton geotechnical centrifuge
University at Buffalo, State University of New York	Large scale, high performance testing facility and versatile high performance shake table facility toward real time hybrid seismic testing
University of California, Berkeley	Reconfigurable reaction wall-based earthquake simulation facility
University of California, Davis	NEES geotechnical centrifuge facility
University of California, Los Angeles	Field testing and monitoring of structural performance
University of California, San Diego	Large, high performance outdoor shake table (LHPOST)
University of Colorado at Boulder	Fast hybrid test platform for the seismic performance evaluation of structural systems
University of Illinois at Urbana-Champaign	Multi-axial, full-scale substructuring testing and simulation facility
University of Minnesota	A system for multi-axial subassemblage testing (MAST)
University of Nevada, Reno	Biaxial multiple shake table research facility
The University of Texas at Austin	Large scale mobile shakers and associated instrumentation for dynamic field studies of geotechnical and structural systems

University of California, San Diego—Camp Elliott Facilities

Of the 15 facilities under the NEES umbrella, the Camp Elliott facility, operated by the University of California, San Diego (UCSD), offers a unique capability for the purpose of completing dynamic testing of a full scale spillway retaining wall for Reclamation. Specifically, the Camp Elliott facility, located 9 miles east of the main UCSD campus as shown on figure 8, is home to the largest shake table in the United States. Figure 9 shows the project layout of the Camp Elliott facilities.

The LHPOST enables large, full scale testing of structures and soil-foundation-structure interacting systems using simulated near source ground motions. The LHPOST steel platen table design consists of a three-piece assembly with each assembly piece having plan dimensions of 13.3 feet by 25 feet giving a total shake table area of 40 feet by 25 feet. The vertical payload capacity of the shake table is 4.5 million pounds, and the frequency bandwidth capability is 0 to 20 Hz. A larger shake table measuring 50-feet by 50-feet in plan is operational at the National Research Institute for Earth Science and Disaster Prevention in Japan; however, logistics and additional expenses of performing a full scale retaining wall test in Japan would be problematic given that the LHPOST would meet the requirements for a full scale test of a Reclamation spillway wall.

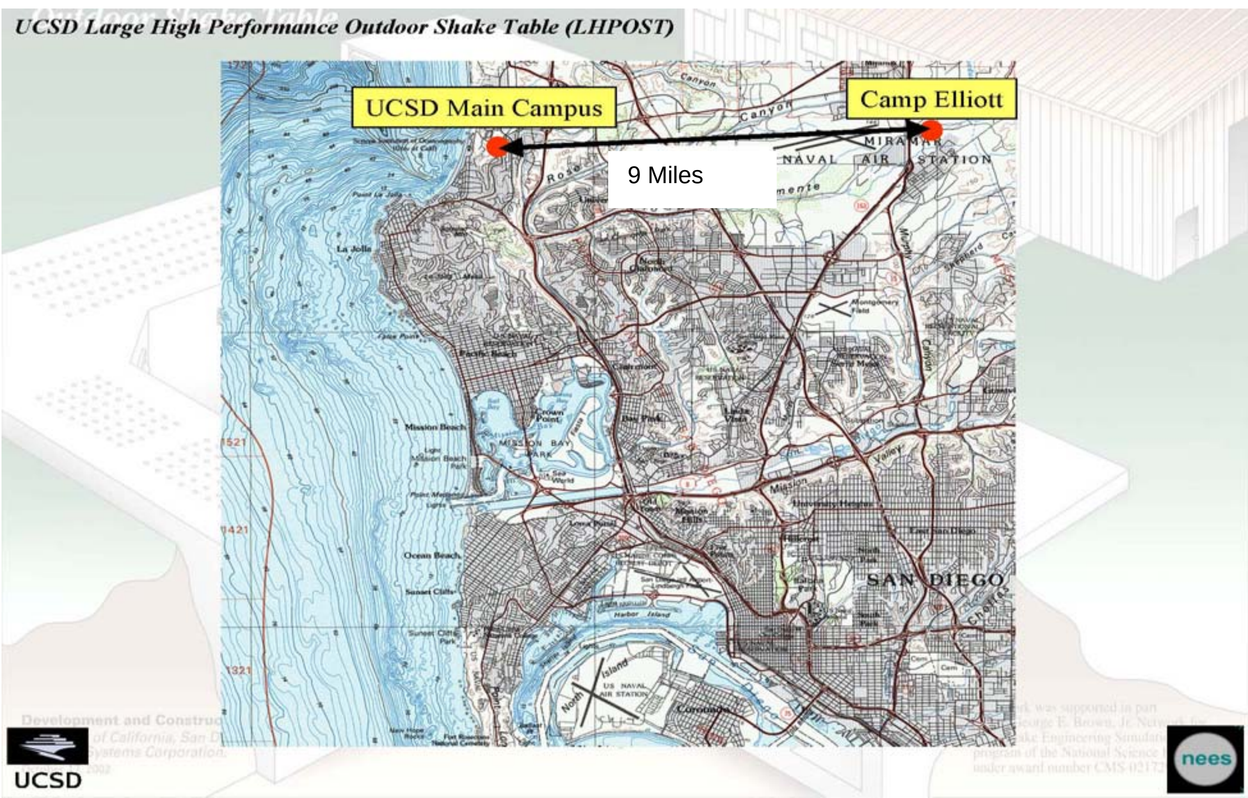

Figure 8.—Location map of Camp Elliott, the home of LHPOST.

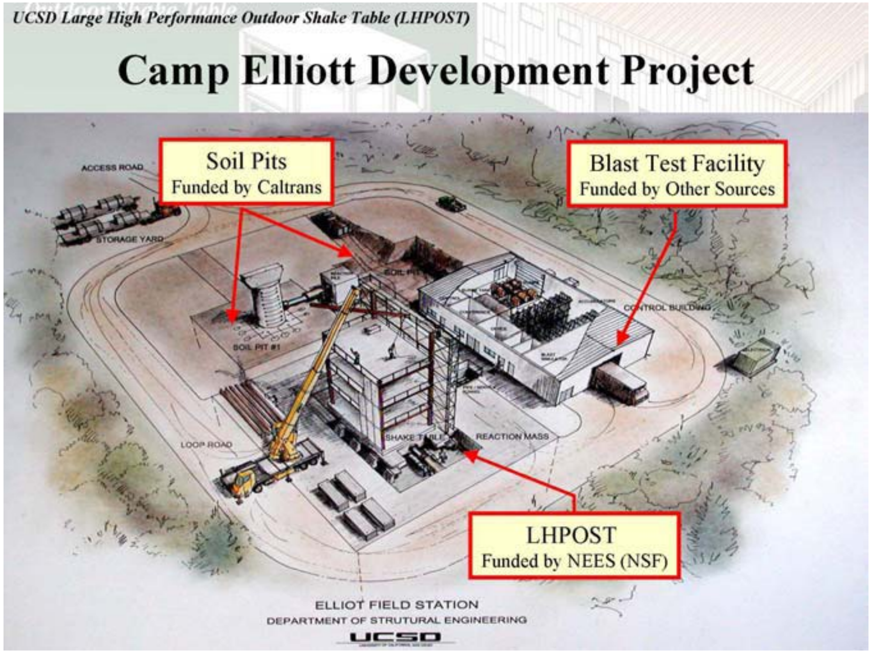

Figure 9.—Project layout of Camp Elliott, the home of LHPOST.

The LHPOST is a uniaxial (horizontal-X) system; however, the shake table is scheduled to be upgraded to a triaxial system by December 2009. Table 3 indicates the expected performance characteristics of the LHPOST after the 2009 scheduled upgrades. Figure 10 shows a cut-away rendering of the LHPOST. The steel platen design of the shake table surface is shown on figure 11.

Table 3.—LHPOST shake table performance characteristics

Direction	Acceleration	Peak velocity, ft/s	Displacement, ft
Horizontal-X	±3.0g	5.0	2.46
Horizontal-Y	±1.5g	2.95	1.23
Vertical	±1.0g	1.64	0.49

Another important feature of the LHPOST is the shake table controller/hydraulic system that has the ability to simulate an entire suite of ordinary historical ground motions, such as those generated by the Northridge, Loma Prieta, and Landers earthquakes, scaled or modified for site-specific conditions. This is an important capability in regards to simulations of highly variable ground motions for Reclamation spillways located in characteristically different seismic areas across the western United States with variable foundation conditions.

Finally, the LHPOST features a local area network (LAN) that is a sophisticated network of hydraulic controllers, data acquisition systems, and video systems that can be used free of charge to researchers under funding by the NSF. This is another important capability for full scale testing of a Reclamation spillway wall

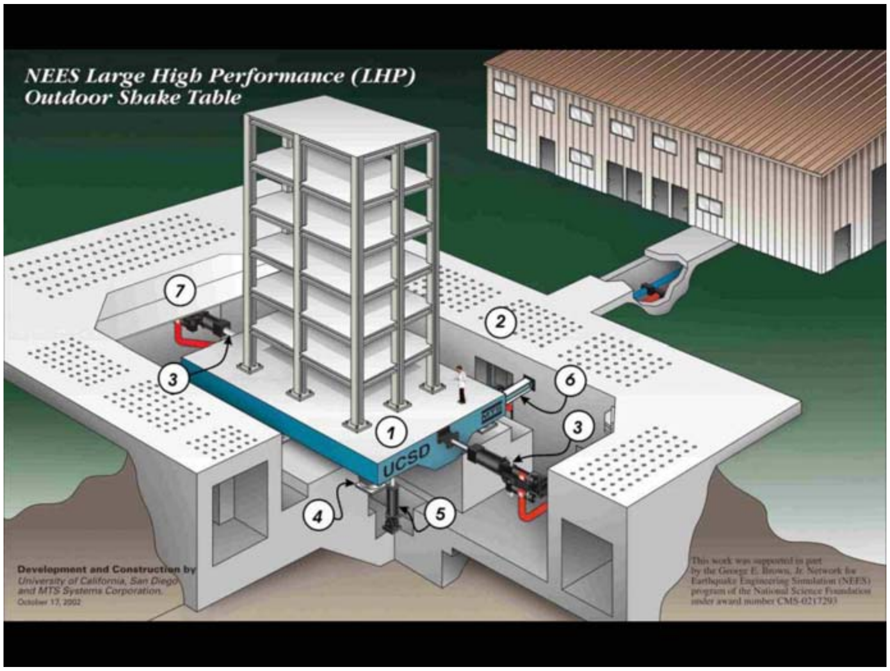

Figure 10.—Cut-away rendering of LHPOST.

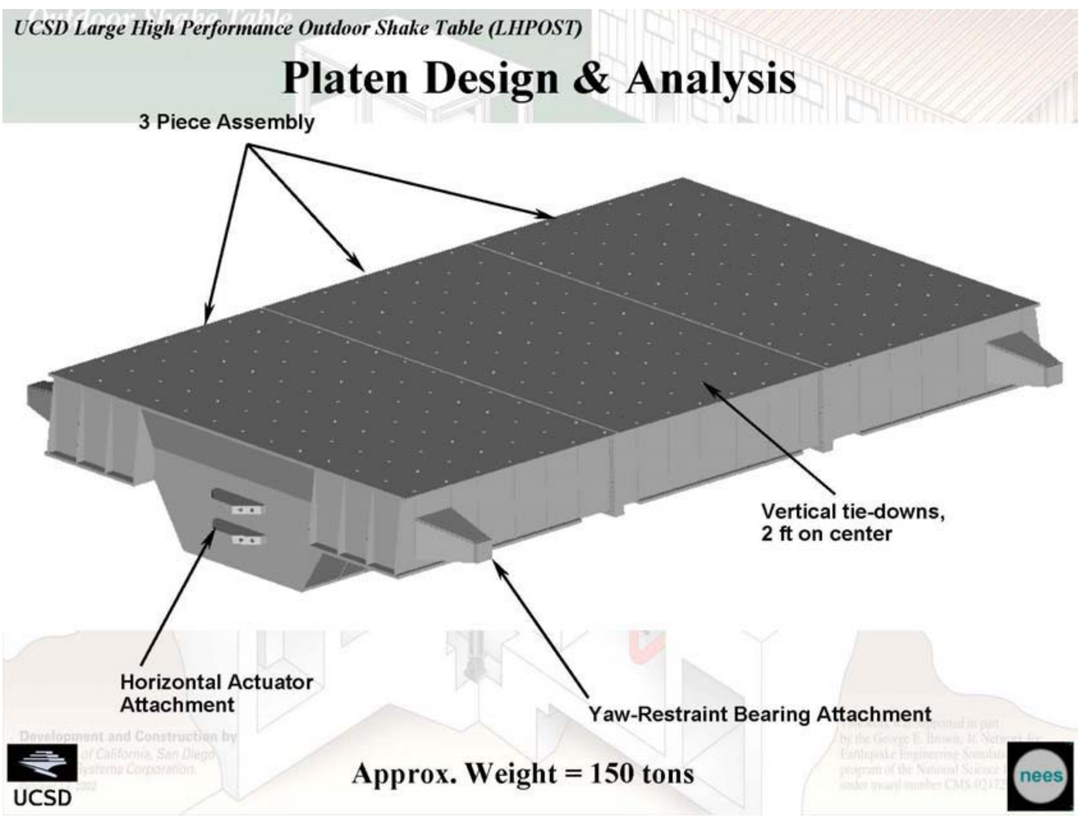

Figure 11.—LHPOST steel platen configuration.

due to the amount of instrumentation that will be required to accurately capture the dynamic soil pressure on the face of the wall along with the structural response of the wall itself. Significant amounts of data will need to be collected and postprocessed for various soil types, phreatic surface levels, and loading conditions that will be evaluated as part of the proposed testing scenarios discussed below. A configuration diagram of the LHPOST LAN is shown on figure 12.

Reclamation has conducted favorable consultations with UCSD regarding the possibility of completing a full scale retaining wall test with the LHPOST. UCSD has provided valuable insight and recommendations to the proposed model configuration presented below.

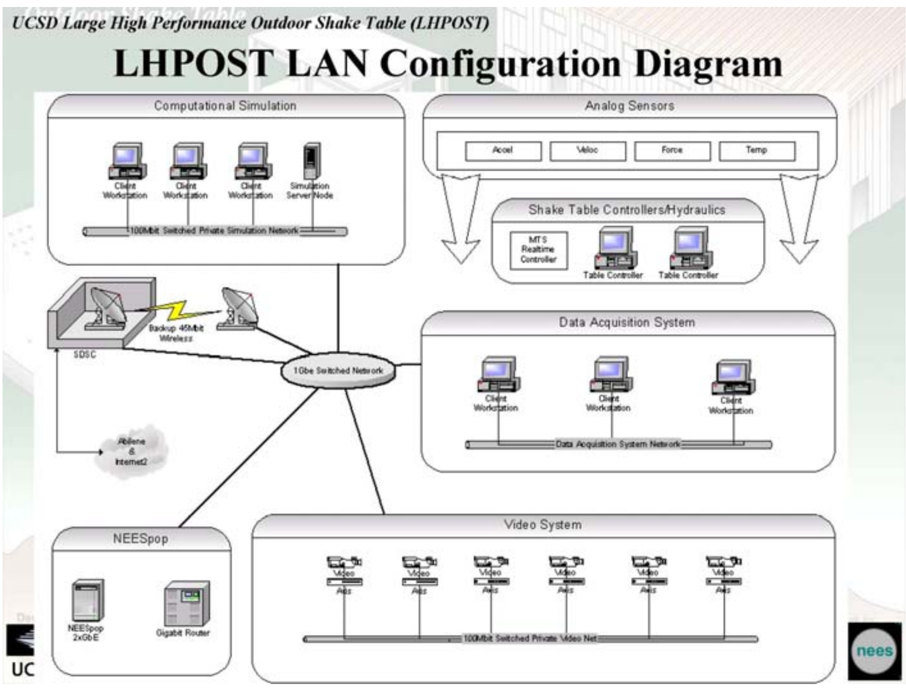

Figure 12.—LHPOST LAN controller and data acquisition diagram.

Proposed Physical Testing Model

Model Configuration

The proposed full scale model configuration is presented in figures 13 and 14. The plan view of figure 13 shows the 40-foot by 25-foot layout of the LHPOST steel platen table with superimposed test configuration. The table shake direction is parallel to the 40-foot dimension. A cantilever concrete retaining wall will be constructed at one end of the shake table to serve as the test subject, and a gravity retaining wall of considerably greater stiffness will be constructed at the other end

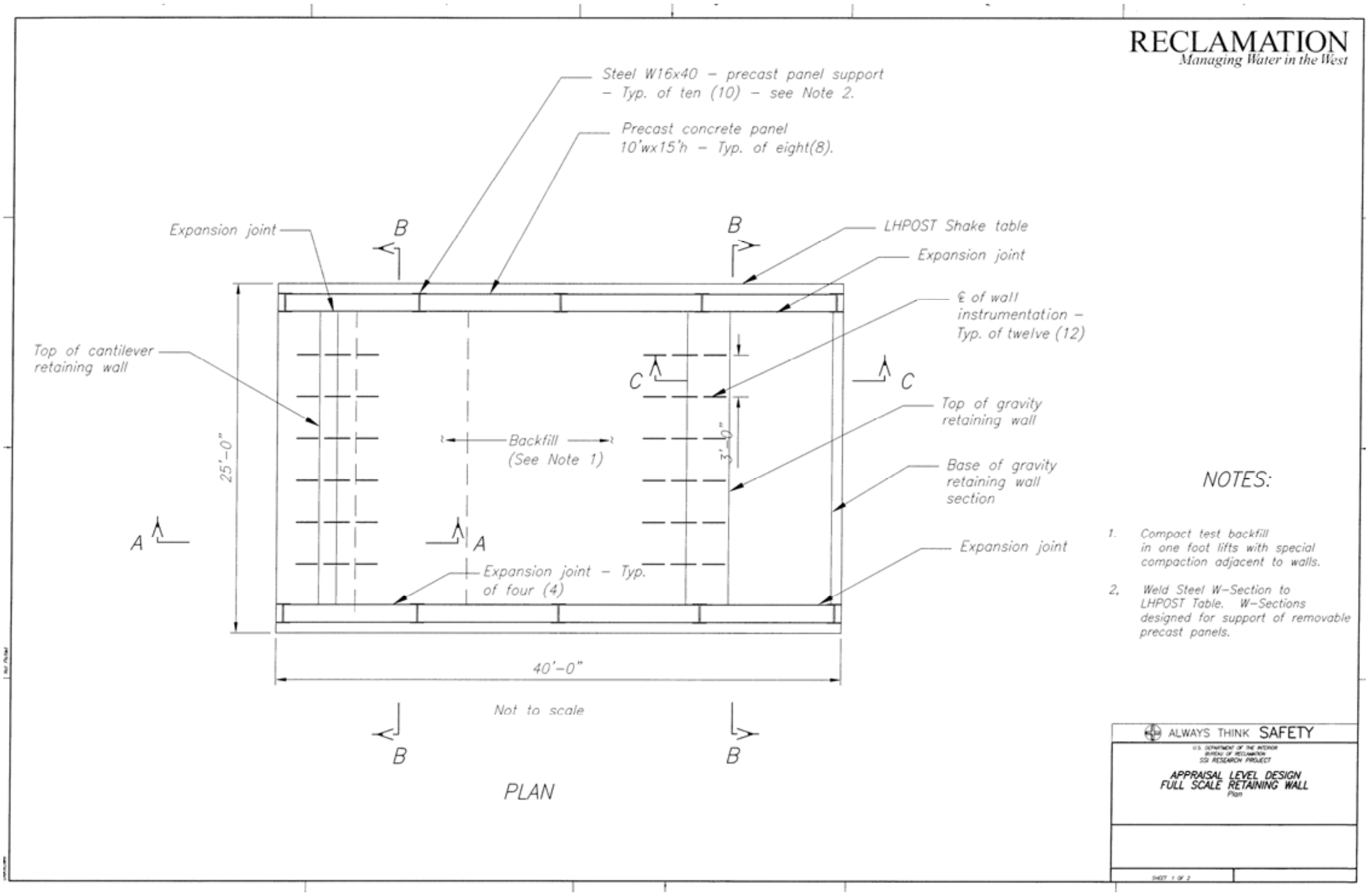

Figure 13.—Plan view of proposed full scale model configuration.

of the shake table intended only to provide a stiff boundary element for the test. Backfill will be placed in 1-foot lifts between the two retaining walls using small roller compactors. Special compaction using hand tampers will be required within 7 feet of the wall sections similar to typical specification requirements for construction of Reclamation spillway walls.

A removable system of precast concrete panels approximately 10 feet wide by 15 feet tall will retain the backfill in the nonshaking direction. W16x40 steel columns rigidly connected to the top of the shake table via the interface plate discussed below will support the precast concrete panels. The W16x40 column supports will allow for removal of individual precast panels for construction access and subsequent removal and replacement of different backfill material types for various test scenarios. A neoprene sheet will be installed between the backfill and the precast panels in an attempt to limit friction between the two features in the nonshaking direction.

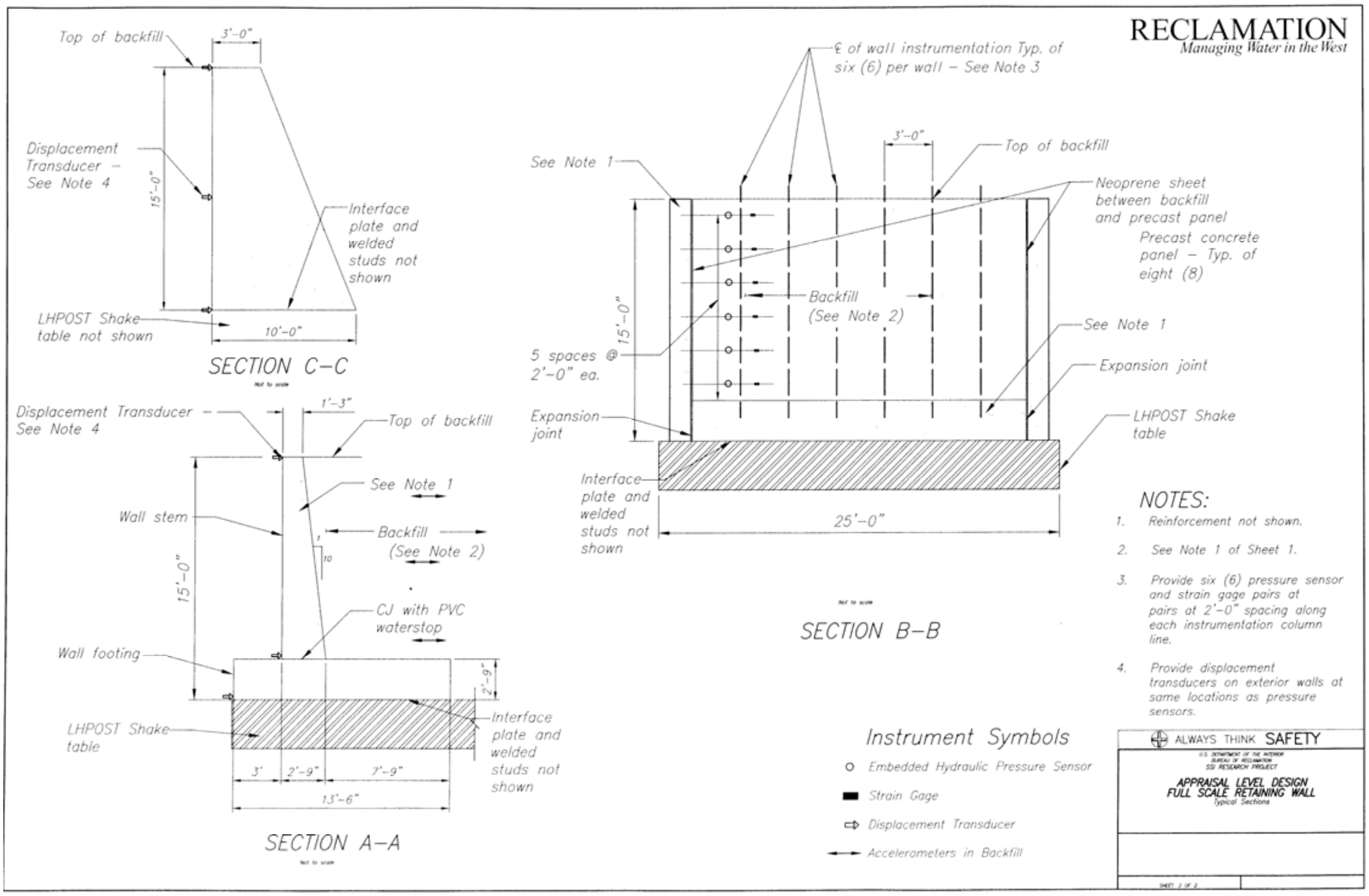

Figure 14.—Typical sections of proposed full scale model configuration.

The proposed cantilever concrete retaining wall is shown in Section A-A on figure 14. The wall will extend 15 feet from the table interface at the base of the footing to the top of the wall stem. A total footing width of 13.5 feet is anticipated with a thickness of 2.75 feet. The wall stem will include a batter on the backfill face of the wall with a stem thickness of 1.25 feet at the top of the wall and a thickness of 2.75 feet at the base of the stem. A polyvinyl chloride (PVC) waterstopped construction joint will be included at the wall stem-footing interface to simplify construction and limit seepage through the base of the wall during test runs with saturated backfill conditions. Primary (vertical and transverse) reinforcement has been included based on traditional retaining wall structural analyses (appendix B), and temperature and shrinkage reinforcement has been included in the longitudinal direction (into and out of the page).

The proposed gravity wall is shown in Sections B-B and C-C on figure 14 and will also be 15 feet high with a 3-foot thick width at the top of the wall and a 10-foot thick width at the base of the wall. Primary (vertical and transverse) reinforcement has been included based on traditional gravity wall structural analyses (appendix B), and temperature and shrinkage reinforcement has been included in the longitudinal direction (into and out of the page).

The proposed connection details between the retaining walls and the LHPOST steel platen table (see appendix B) are based on the recommendations from UCSD. Special considerations will need to be given to this detail as (a) the retaining walls must be restrained from sliding off the shake table during the testing, (b) the connections must be as rigid as possible to minimize shake table boundary effects, and (c) the retaining walls must be removed from the shake table without damaging the LHPOST steel platen table after completion of the testing. As a result, a ⅜-inch thick steel interface plate will be provided. The steel interface plate will be anchored to the LHPOST table using threaded anchor rods in a 2-foot by 2-foot grid configuration. The cantilever wall footing and the base of the gravity wall will then be anchored to the interface plate using Nelson-type welded studs. The studs will be welded to the interface plate and embedded in the concrete of the retaining walls to provide the shear resistance necessary to eliminate sliding of the walls.

Testing Scenarios

The proposed testing scenarios will consist of variations in backfill material types, phreatic surface levels, and loading conditions. A matrix of proposed test scenarios is presented in figure 15.

Many practicing engineers and researchers in the field of SSI regard cohesion as a significant parameter in variability of seismic lateral earth pressures. As stated earlier, cohesion is generally considered an unscalable material parameter in terms of similitude for small scale models and, as a result, dynamic small scale centrifuge tests have historically been completed using cohesionless Nevada sand material. However, most backfill material used for Reclamation retaining structures has at least some cohesion and, for cases of spillway crest structure walls backfilled with zone 1 material, could have a significant amount of cohesion. As a result, three alternative backfill types are proposed for this initial (appraisal-level) development of the testing procedure:

- Material Type I—Nevada sand to compare with previous centrifuge model test results

- Material Type II—Local silty sand (SM) with low to moderate cohesion representative of embankment shell material backfilled against Reclamation spillway chute and approach channel walls

- Material Type III—Imported clay (CL) with high cohesion representative of embankment core material backfilled against Reclamation spillway control structure walls

Another key parameter to be considered for Reclamation hydraulic retaining structures is the phreatic surface level or saturated pore pressure within the backfill material. Specifically, phreatic surface levels can vary substantially along

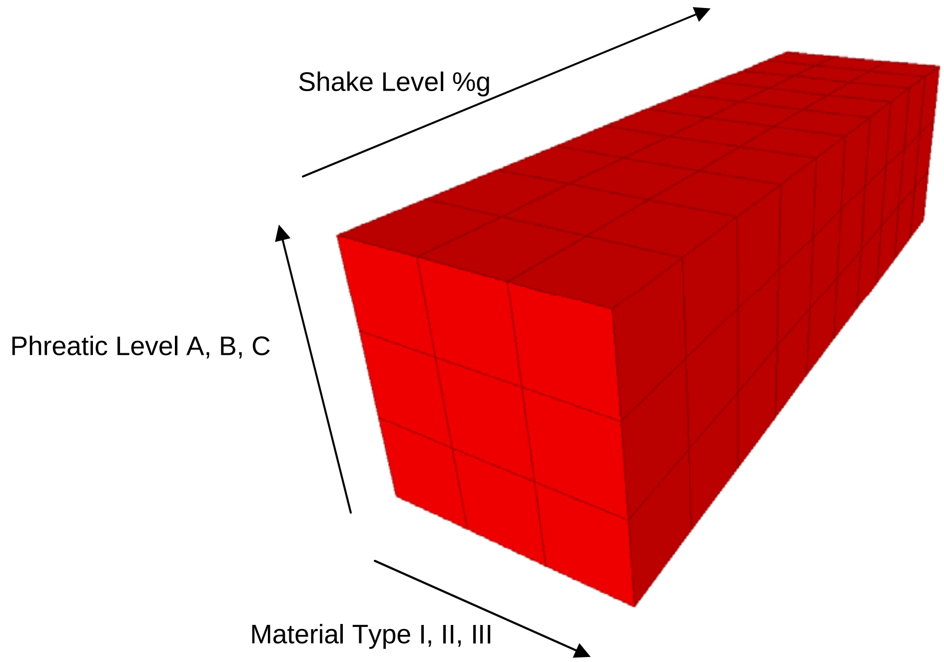

Figure 15.—Shake table test scenarios matrix.

the profile of a spillway wall from completely saturated upstream of the control crest structure to little or no saturation downstream of the crest structure. The influence of phreatic surface levels within backfill material on seismic lateral earth pressures has been overlooked in SSI research. As a result, three alternative phreatic surface levels are proposed for this initial (appraisal-level) development of the testing scenarios:

- Phreatic Level A—0-foot phreatic surface height
- Phreatic Level B—7-foot phreatic surface height
- Phreatic Level C—14-foot phreatic surface height

The final key parameter considered for test scenarios is the level of seismic loading. Numerous shaking levels would be tested along with test runs using various time histories. This would be the easiest of the three key parameters to vary as part of the suite of test scenarios to be executed. Specifically, the LHPOST hydraulic controllers and data acquisition systems are able to execute test runs with various shake inputs and collect test data for each run very quickly. As a result, an incremental seismic loading level up to 1.5g with a frequency range from 1 to 10 Hz was assumed for this level of study. This means that incremental shaking tests would be performed at 0.1g, 0.25g, 0.35g, 0.50g, 0.75g, 1.0g, 1.15g, 1.25g, and 1.50g. Sinusoidal shaking inputs are typically used for baseline analyses, but various time history shaking inputs that are scaled and modified from real earthquake records, such as the Loma Prieta (1989), Northridge (1994), and Landers (1992) earthquakes, will play an important role in this research project. The proposed test run plans for seismic loading levels and shaking inputs will be developed as part of subsequent phases of this research project.

These three key parameters provide a method for establishing a designation for each specific test run. For example, a test run utilizing local SM backfill material (i.e., Material Type II) with a phreatic surface height of 7 feet (i.e., Phreatic Level B) and a maximum shaking level of 1.25g would be designated Test IIB125.

Instrumentation Requirements

Instrumentation requirements are assumed to be significant, as it would make no sense to shortcut data collection in an attempt to save money on a test program as thorough and expensive as the one proposed. Traditional instruments such as strain gauges, displacement transducers, and accelerometers would be utilized along with new, state-of-the-art instruments. Two such instruments considered for the purposes of this study are shape tape and tactile pressure sensors. Shape tape is a self-contained, fiber-optic-based, three-dimensional, bend and twist sensor system, as shown in figure 16, that consists of strip sensor tape that would be attached to each retaining wall to obtain real time deflections of the wall during shaking. Another state-of-the-art system is the tactile pressure sensor that consists of a self-contained pad that encloses a 4-foot by 4-foot grid of independent pressure sensors as shown on figure 17.

To obtain the desired earth pressure distributions in each direction along the face of the wall, six instrumentation lines at 3-foot spacing along the length of the retaining walls were assumed as shown on Section B-B of figure 14. Strain gauges, accelerometers, and tactile pressure sensors will be installed at 2-foot spacing along the height of the retaining walls at each of the six instrumentation line locations. The cantilever wall will include displacement transducers installed at the top and bottom of the wall stem and at the base of the footing as shown in Section A-A of figure 14. The gravity wall will also include displacement transducers at the top, mid-height, and bottom of the section. In addition, instrumentation will be required to monitor the behavior of the backfill during shake testing. As a result, a 6-foot by 5-foot grid of displacement transducers will be secured to the top surface of the backfill between the two walls, and a series of accelerometers will be placed within the backfill at three elevations and along three line locations in the direction of shaking.

Since the proposed test scenarios consist of three cycles of backfill placement and removal, a replacement instrument rate of 50 percent was assumed for all strain gauges and accelerometers, based on MERL experience with such instruments.

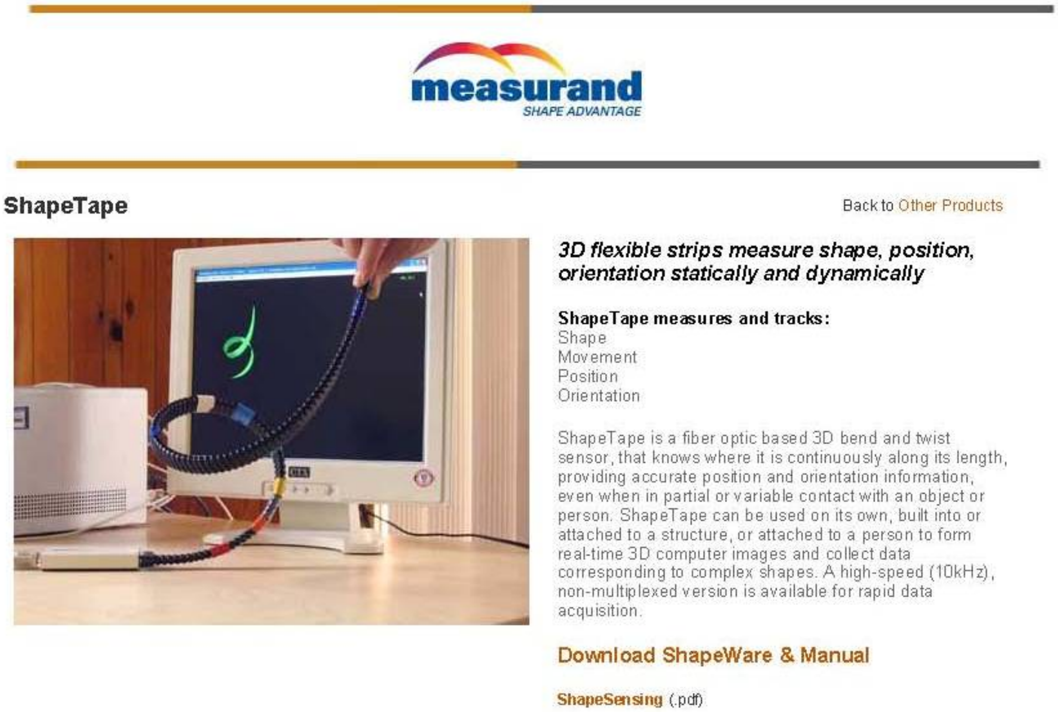

Figure 16.—Typical shape tape instrumentation for measuring real time deflections.

Figure 17.—Typical TactArray sensor for obtaining real time pressure data.

This rate is consistent with a reasonable backfill excavation and placement rate assuming the contractor will require some care in his earthfill work.

Finally, light steel frames spanning the 25-foot shake table direction will be required to serve as fixtures for mounting the displacement transducers to the top surface of the backfill for monitoring backfill movements as well as movements along the top of each retaining wall.

Special Considerations for Full Scale Testing

A full scale retaining wall test of the sophistication and complexity proposed would require a tremendous amount of careful evaluation and detailing beyond the scope of these Scoping Phase 1 studies. Part of the Planning Phase 2 work for this research project will consist of identification and procurement of a PhD researcher currently practicing in the field of SSI. The risk of missing key parameters cannot be overstated in the design of test model configuration, development of the test procedure, and determination of instrumentation requirements, along with the experience that may be required to interpret full scale testing results. Some of the special considerations identified in this Scoping Phase 1 work are discussed below.

Boundary conditions are always a concern with any physical model study; however, the potential impacts of not properly addressing boundary effects could be catastrophic for this particular type of full scale testing. Specifically, the proposed 15-foot height of the retaining walls is estimated to provide sufficient soil depth beyond the intersection of the soil wedge failure surface and the top of the backfill given the physical size limitations of the LHPOST shake table.

End conditions in the direction of shaking are additional boundary conditions requiring significant evaluation. A gravity retaining wall was selected to stiffly oppose the cantilever retaining wall. Specifically, to simulate the stiffness of backfill conditions in the field, an infinitely stiff wall at the opposite end of the shake table backfill would be ideal to model the limit of elastic/plastic compression of the backfill and to prevent immediate resonant type deflection of dynamic energy back toward the cantilever retaining wall.

Another key consideration is three-dimensional effects due to the physical size limitation of the LHPOST steel platen table in the nonshaking direction. The effects of the precast panels retaining the backfill perpendicular to the shaking direction need special consideration to limit friction between the soil and the precast panels. To this end, a neoprene sheet will be provided at the interface between the backfill and the precast panels to minimize sliding friction as shown in Section B-B of figure 14.

Cost Estimate

Reclamation's construction and cost estimating group developed an appraisal-level cost estimate for all civil components of the proposed test model configuration. Reclamation's MERL developed an appraisal-level cost estimate for all instrumentation requirements.

The cost estimate prepared and provided here is intended to be used as a tool that serves as a foundation in realizing management objectives, budgetary requirements, and economic analysis. The costs provided in the estimate are intended to be for appraisal-level costs and strictly for use in evaluating the feasibility of undertaking the full scale test project being proposed.

Generally accepted industry criteria and engineering judgment were used to develop the cost estimates. Costs from similar type construction, when applicable, and some preliminary contacts with material suppliers and contractors were also used to develop the estimate.

The estimated construction costs are in 2009 dollars and are based on the assumption that the work would conservatively be contracted using a request-for-proposal (RFP), negotiated contract with a qualified 8a contractor. A more competitive procurement process could result in potential cost savings for the project. Estimated construction costs would need to be adjusted accordingly if construction were performed after 2009 or if another type of construction contract were used.

The estimated cost reflects a professional opinion of the likely costs to construct the project, subject to the limitations discussed herein. A number of factors affect actual contractor costs such as the project location; the supply and demand in the business area for this type of construction at the actual time bids are due or prices are negotiated; changes in material and equipment costs; and changes in labor rates. Therefore, conditions and factors at the time of procurement may result in bids that are significantly different than the estimate presented here. The cost estimate provided in this report is for project feasibility and financial planning purposes only and should not be used in bidding or contracting.

The quantities and cost estimate are based on the appraisal-level design layout shown in figures 13 and 14 and included in appendix B.

Construction pricing for project features is based on the quantity estimates for each item identified on the estimate worksheet of appendix C. Pricing was accomplished with unit pricing from both published and internal Reclamation historical databases, factored for project location and other project-specific criteria. Material pricing, where necessary, was obtained from vendor verbal quotation, current cost estimates, owner-provided information, and cost estimator experience. The logic, methods, and procedures for developing cost estimates are typical for the construction industry. An experienced construction cost estimator with construction and hard-dollar contract bid experience prepared this cost estimate.

Costs for the following items were not included in the cost estimate:

- Engineering costs for completion of Planning Phase 2, Feasibility Phase 3 and Final Design Phase 4 of the project
- Environmental studies and permitting
- Construction management
- Construction oversight
- Operation and maintenance
- Escalation to FY2013 dollars when Laboratory Testing Phase 5 is scheduled for completion

Costs for a full time Reclamation engineer to monitor and coordinate instrumentation and data acquisition activities during actual performance of the full scale testing have been included in the cost estimate.

Detailed assumptions used in development of the cost estimate include:

- f'_c = 4,000 lb/in^2 at 28 days for all structural concrete
- Grade 60 reinforcement for structural concrete
- F_y = 50 ksi for structural steel members
- Fillet weld all around the perimeter of W16x40 steel columns at their connections to the steel shake table interface plate
- Individual test run cycles (scenarios) will include placement of backfill in 1.0-foot lifts with roller compaction using small rollers except in areas adjacent to the retaining walls.
- Special compaction using hand plate tampers will be required within 7 feet of the retaining walls.
- Following completion of the shake table test for each material type, backfill will be partially removed to allow for removal of precast panels followed by removal of the remainder of backfill.

- Concrete demolition, removal, and disposal of the retaining walls will be required following completion of all test scenarios.

- The general contractor or a subcontractor will install all instrumentation; however; Camp Elliott personnel will establish data acquisition connections.

- NSF funding will be obtained for the use of the LHPOST at Camp Elliott through the NEES proposal process discussed previously. NSF funding covers the shake table, general facilities, data collection equipment, and utilization of data acquisition personnel

A construction contingency of 15 percent is included for this research project in case unanticipated issues or items arise such as change orders or variations in bid quantities resulting from changed or unexpected field conditions.

A total estimated field cost in 2009 dollars of $2.8 million is estimated for completion of the full scale, retaining wall, shake table test project as presented here. A detailed cost estimate worksheet and quantity breakdown is included in appendix C.

Summary of Scoping Phase I Findings

The findings of this Scoping Phase 1 study can be summarized as follows:

- Due to increased seismic hazards in high seismic areas, Reclamation spillway walls will continue to be a source of high risk and potentially costly mitigation efforts without addressing the considerable knowledge gap on quantification of seismic lateral earth pressures.

- Both established analytical methods and recent physical test model studies completed in conjunction with numerical analyses fail to independently and adequately validate research results in the field of SSI. Research results and conclusions have been inconsistent as shown in table 4. This is the primary reason why no industry-accepted and consistent guidelines have been established to either disprove or supplant the Mononobe-Okabe or Wood methods in the field of dynamic SSI.

- The LHPOST shake table at the NEES Camp Elliot facility, operated by UCSD and under funding by NEES via NSF, has the exclusive capabilities in the United States to perform a full scale dynamic test of a cantilever concrete retaining wall to develop a standardized method of determining seismic lateral earth pressures for design and evaluation of earth-retaining structures.

Table 4 - Comparison Summary of SSI Literature Search Results

		Researcher/Year	Green/2003	Ostadan/2005	Madabhushi & Zeng/200	Matsuo/1941	Ishii/1980	Matsuo & Ohara/1960	Sherif/1982	Bolton & Steedman/1982	Sherif & Fang/1984	Steedman/1984	Ishibashi & Fang/1987	Bolton & Steedman/1985	Ortiz/1982	Stadler/1996	Whitman/1990	Nakamura/2006	Atik/2008	Chang/1990[1]	Clough & Fragaszy/1977	Koseki/1998
Test Method	Centrifuge Model				•									•	•	•	•	•	•			
	Small/Medium Shake Table Model					•	•	•	•	•	•	•	•									
	Field/EQ Test																			•	•	•
g's	Peak Acceleration					1	1	1	1	1	1	1	1									
Backfill	Saturated Backfill				•																	
	Dry Backfill		•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•
Analysis	Software/Method		FLAC	SASSI	SWANDYNE														OpenSees			
Comparison of Results	< M-O															•	•	•	•	•	•	•
	= M-O		•			•	•	•	•	•	•	•	•	•	•					•		
	> M-O		•		•																	
	< Woods																					
	= Woods			•																		
	> Woods			•																		

Notes:
1. Field tests performed on embedded walls of nuclear reactor containment structure.

- A conceptual design configuration consisting of opposing cantilever and gravity retaining walls is proposed for physical testing as shown on the figures presented in appendix B. Each retaining wall is 15 feet tall and bounded in the nonshaking direction by removable, precast concrete panels that provide easy access for placement and removal of backfill materials.

- Proposed testing procedures would be extensive to include key parameters associated with Reclamation spillway wall field conditions, including:

  - Various backfill material types with varying degrees of cohesion

  - Various phreatic surface levels within the backfill

    - Various magnitudes and frequencies of the shaking inputs including time history runs

- Special considerations to be further evaluated for subsequent phases of this research project include:

    - Detailed numerical analyses and engineering evaluations of boundary conditions and boundary effects associated with the proposed test model configuration

    - Sensitivity studies on all instruments, including various sensors and data acquisition

    - Interface connection design details between the walls and the shake table steel platens

- The estimated cost in the 2009 dollars to construct and execute the proposed full scale test program is $2.8 million, which includes the costs of:

    - Construction

    - Instrumentation procurement and installation

    - Observation and oversight of instrumentation and data acquisition by a full time Reclamation engineer

    - Implementation including data collection

    - Removal and replacement of backfill for three cycles of material type testing

    - Demolition and removal on constructed test features

Planning Phase 2 Recommendations

Should, after careful consideration of the proposed test configuration presented herein, the dam safety research team members feel this test project be continued, the following tasks are proposed for the Planning Phase 2 to be completed in FY2010:

- Procurement of a practicing PhD research consultant

- Cost-benefit analysis

- Identification of funding sources for final design engineering
- Identification of funding sources for construction
- Documentation of recent Reclamation SSI finite element study results

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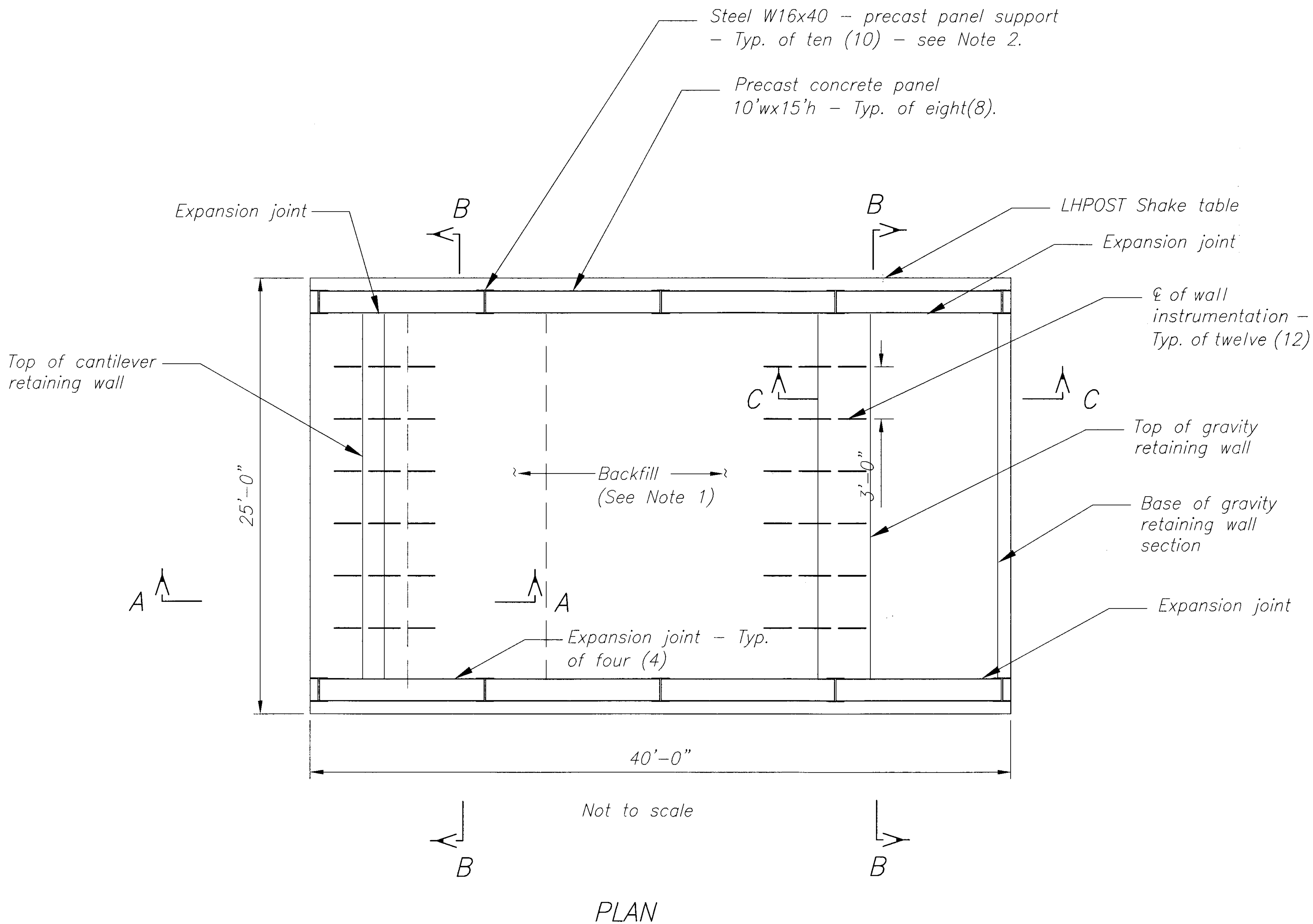
Appendix A—Reference Library CD

Appendix B—Appraisal-Level Design

RECLAMATION
Managing Water in the West



NOTES:

1. Compact test backfill in one foot lifts with special compaction adjacent to walls.

2, Weld Steel W-Section to LHPOST Table. W-Sections designed for support of removable precast panels.

ALWAYS THINK SAFETY

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SSI RESEARCH PROJECT

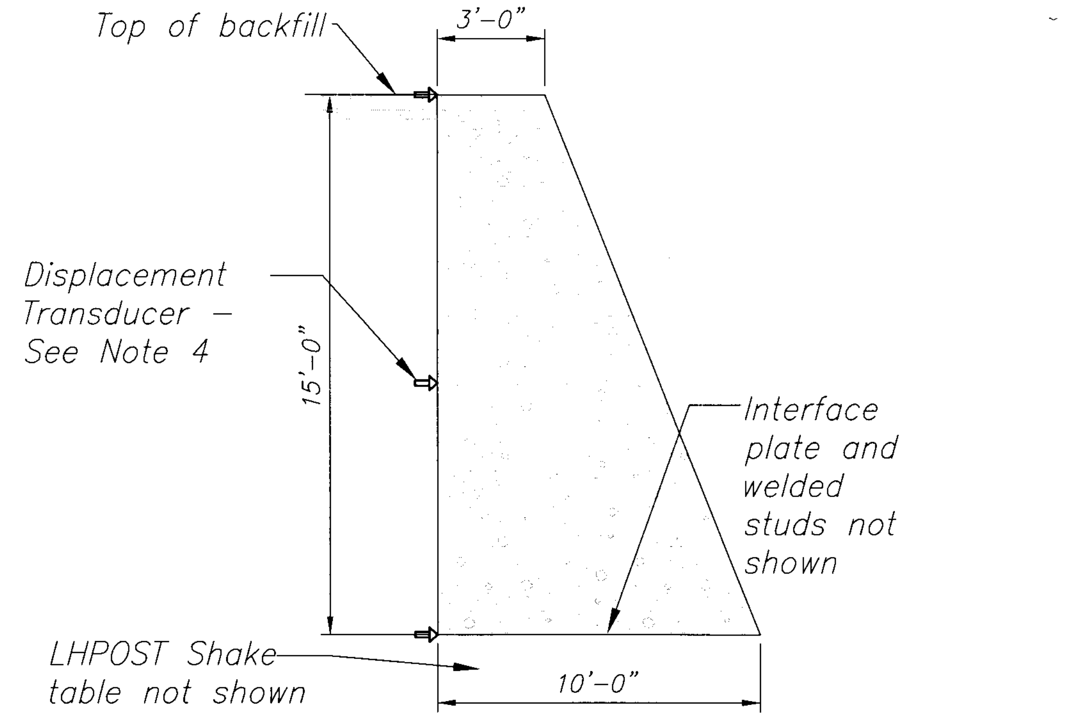
APPRAISAL LEVEL DESIGN
FULL SCALE RETAINING WALL
Plan

SHEET 1 OF 2

DATE AND TIME PLOTTED Not Plotted
PLOTTED BY Not Plotted

CAD SYSTEM AUTOCAD
CAD FILENAME UNKNOWN

RECLAMATION
Managing Water in the West



SECTION C–C
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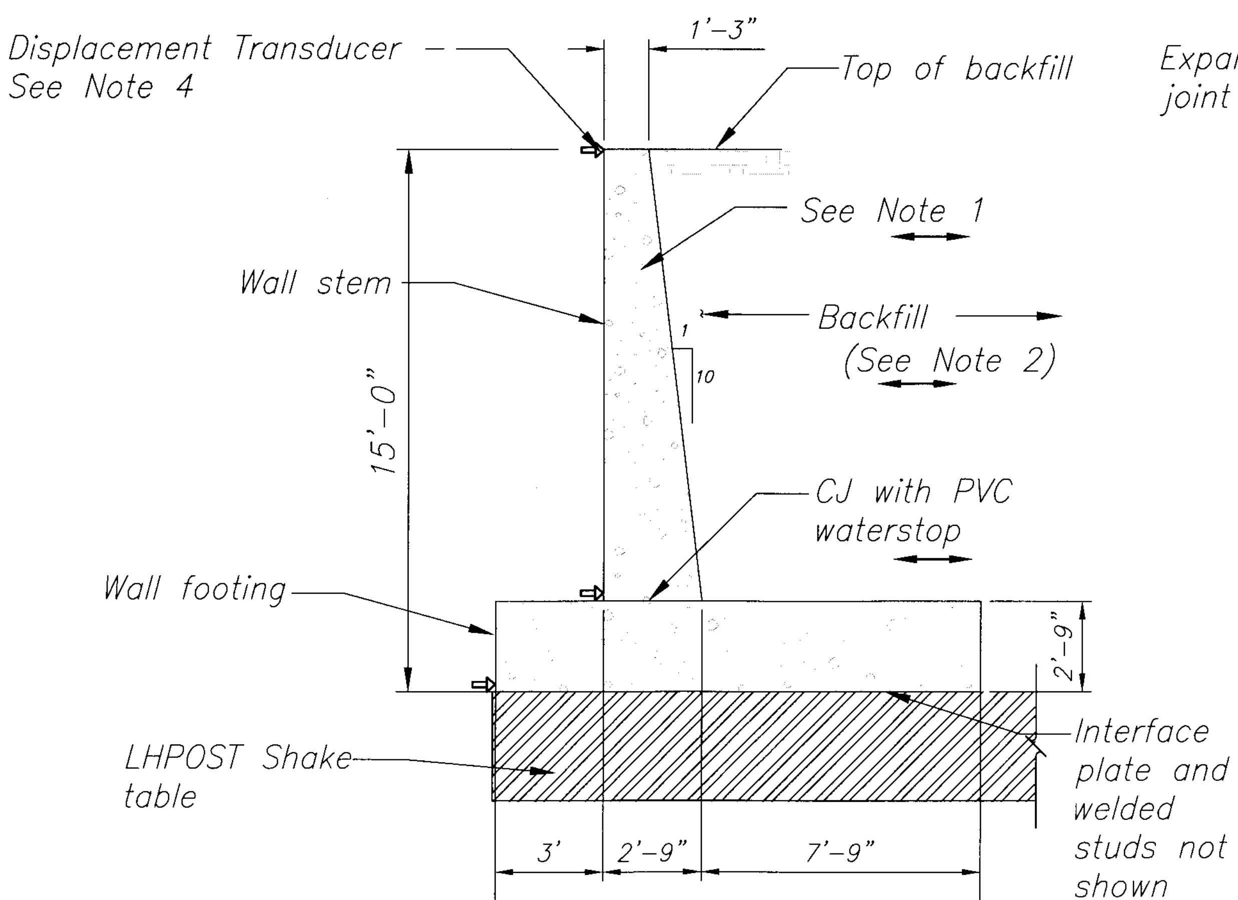

SECTION A–A
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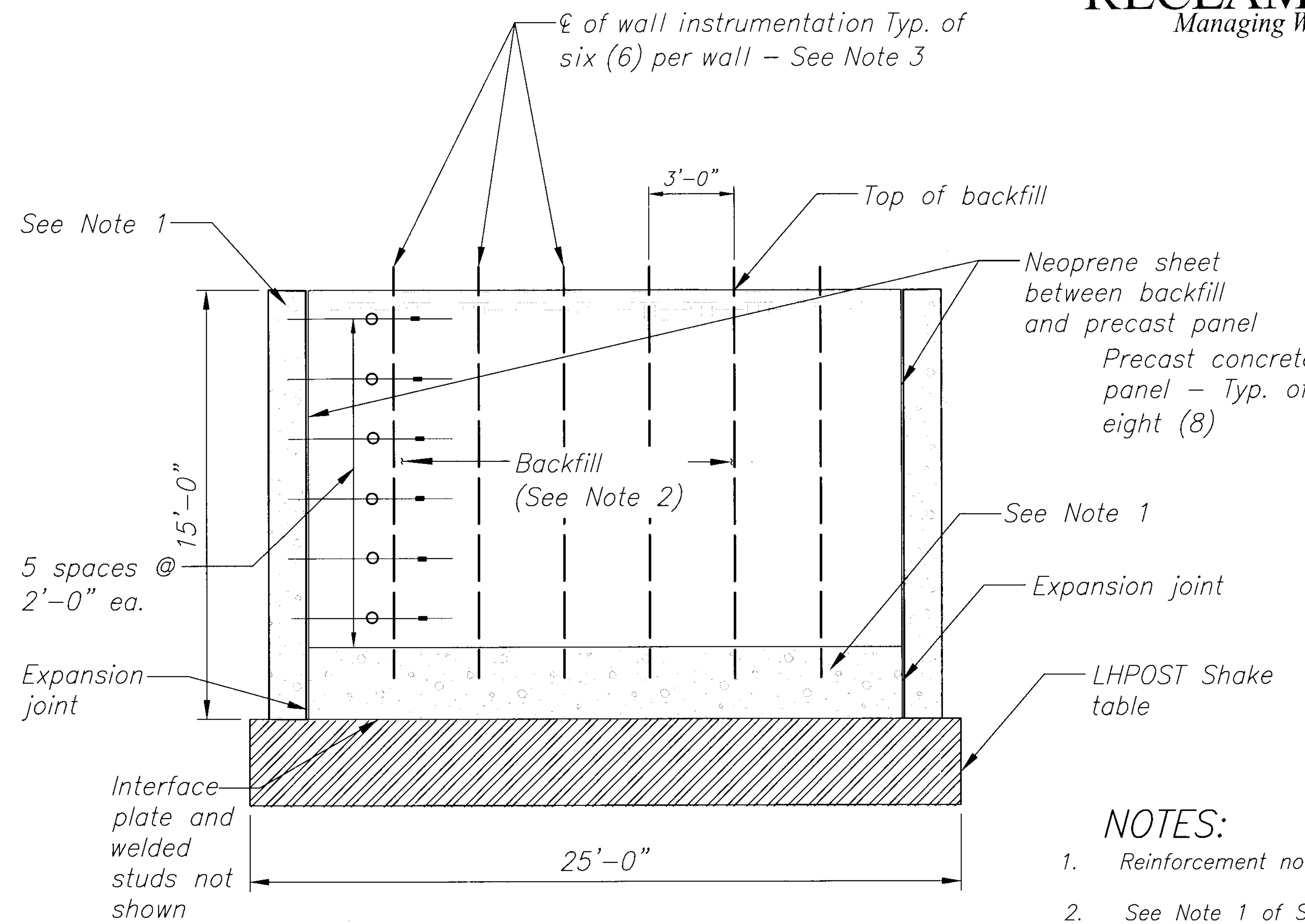

Not to scale

SECTION B–B

NOTES:

1. Reinforcement not shown.
2. See Note 1 of Sheet 1.
3. Provide six (6) pressure sensor and strain gage pairs at pairs at 2'–0" spacing along each instrumentation column line.
4. Provide displacement transducers on exterior walls at same locations as pressure sensors.

Instrument Symbols

- ○ Embedded Hydraulic Pressure Sensor
- ■ Strain Gage
- ⇨ Displacement Transducer
- ⟷ Accelerometers in Backfill

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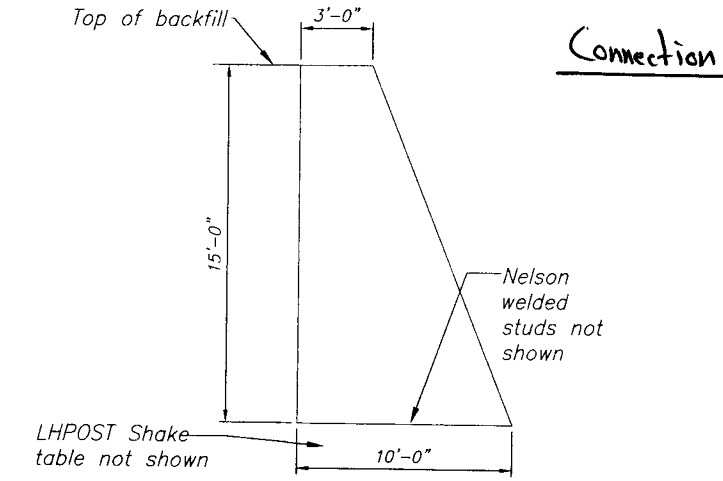
SSI RESEARCH PROJECT

APPRAISAL LEVEL DESIGN
FULL SCALE RETAINING WALL
Typical Sections

SHEET 2 OF 2

Connection Detail



SECTION C-C

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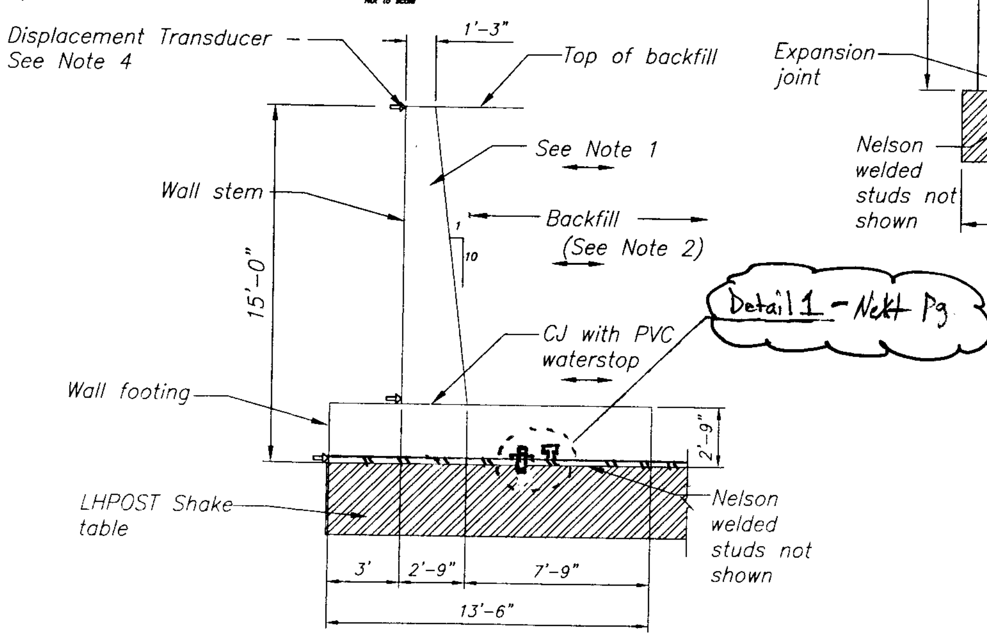

SECTION A-A

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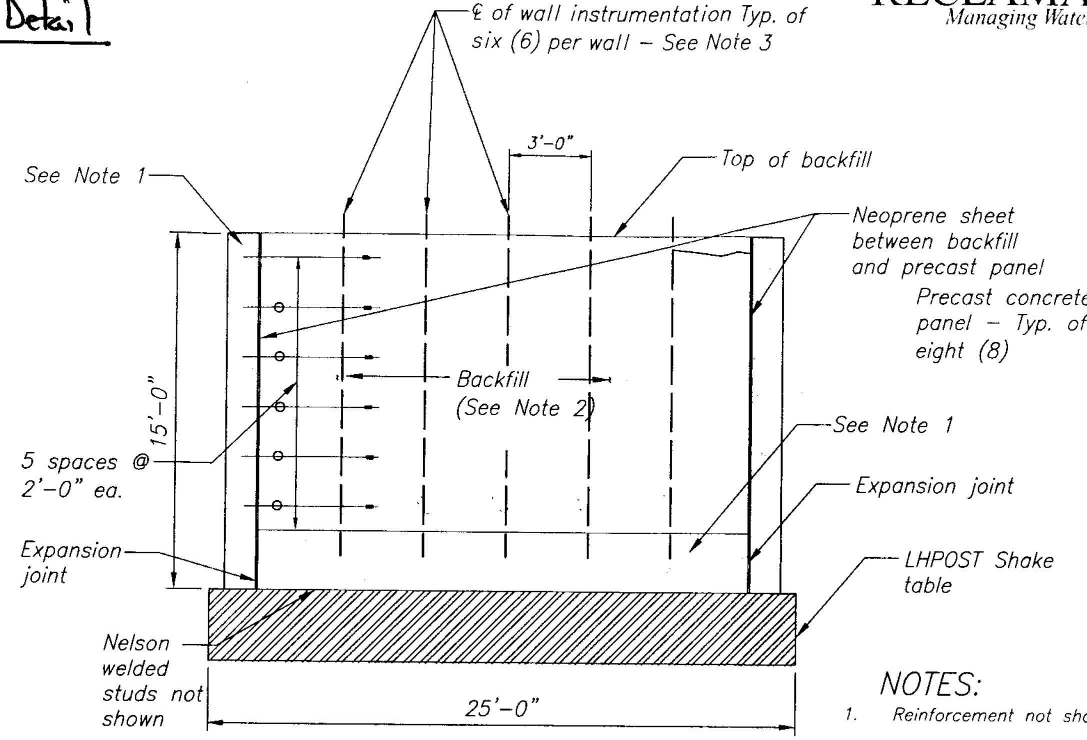


Not to scale

SECTION B-B

NOTES:

1. Reinforcement not shown.
2. See Note 1 of Sheet 1.
3. Provide six (6) pressure sensor and strain gage ~~pairs at~~ pairs at 2'-0" spacing along each instrumentation column line.
4. Provide displacement transducers on exterior walls at same locations as ~~pressor~~ pressure sensors.

Instrument Symbols

- ○ Embedded Hydraulic Pressure Sensor
- ■ Strain Gage
- ⇨ Displacement Transducer
- ⟷ Accelerometers in Backfill

10Hz e lg

ALWAYS THINK SAFETY

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SSI RESEARCH PROJECT

APPRAISAL LEVEL DESIGN
FULL SCALE RETAINING WALL

SJD

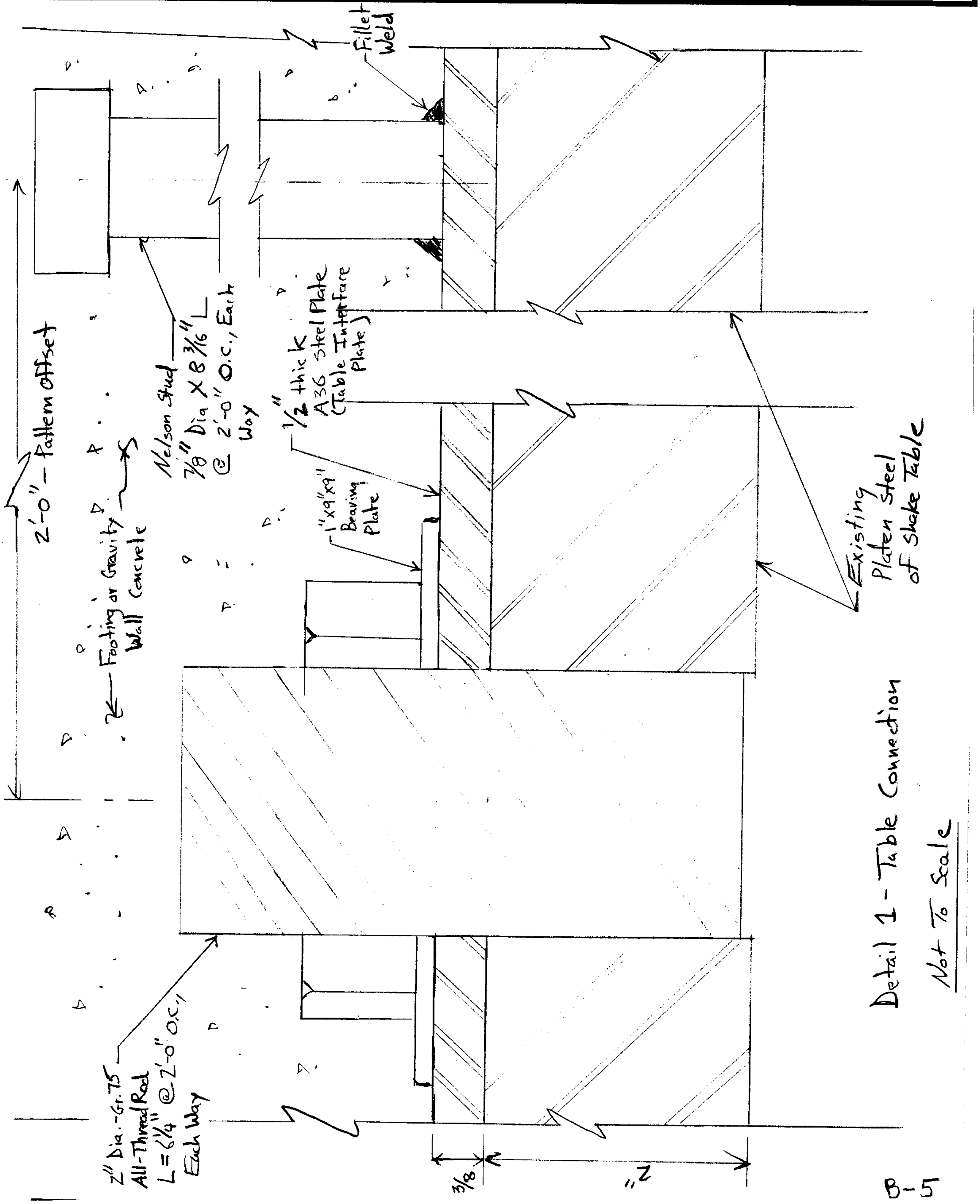
SHEET 1 OF 2

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
SJD	7/6/2009	SSINT- Phase 1	
CHKD BY	DATE	FEATURE	

DETAILS Interface Table Connection Detail

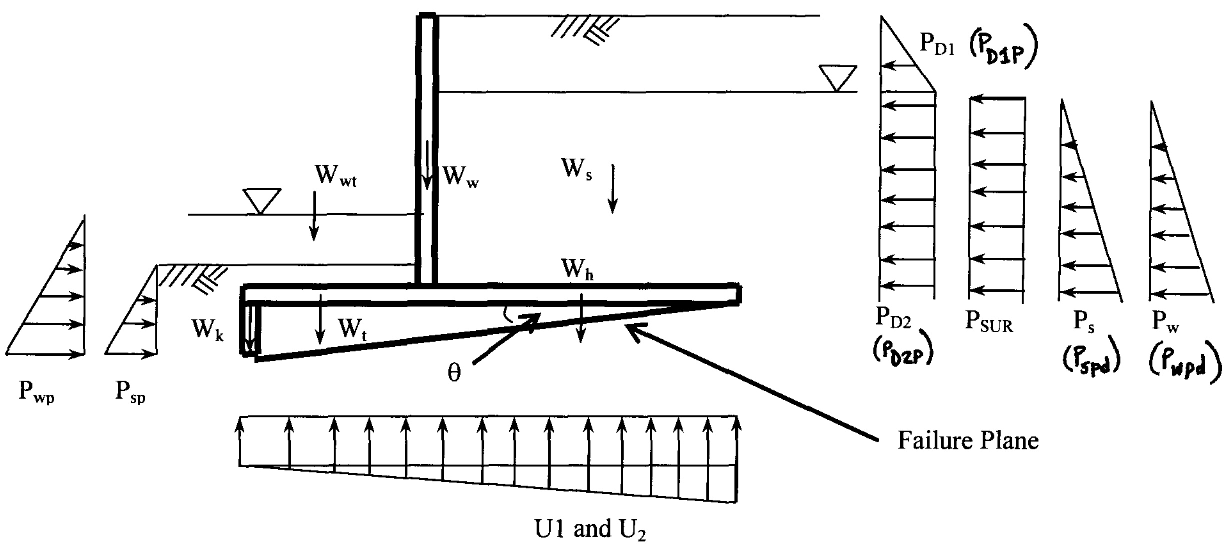


Detail 1 - Table Connection
Not To Scale

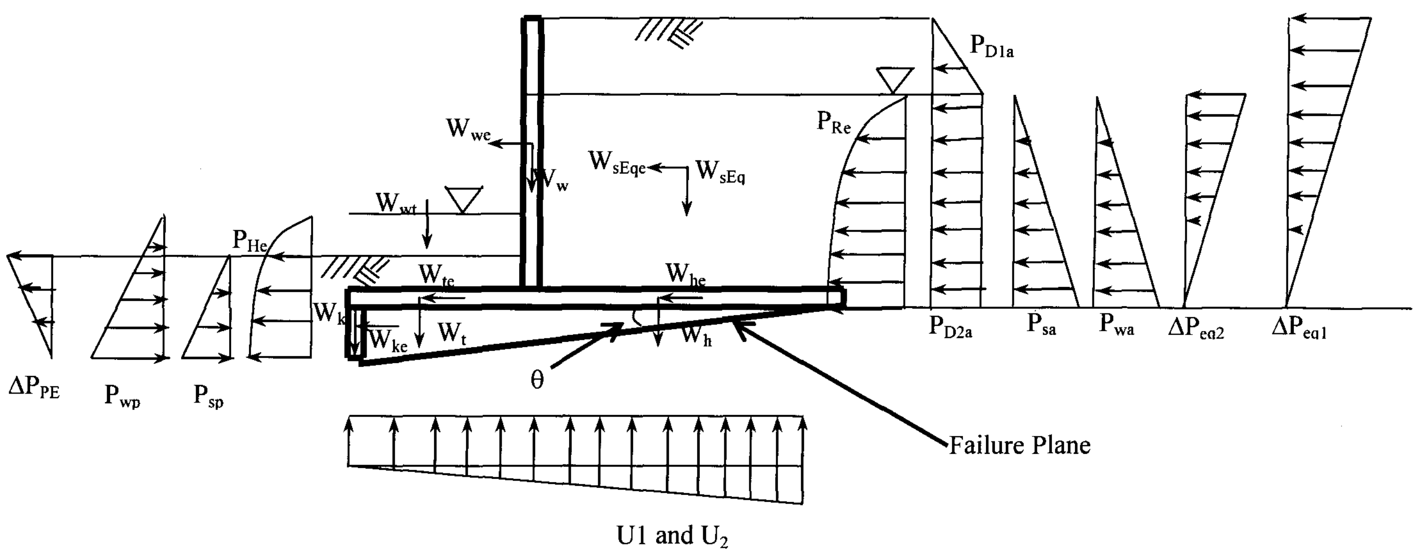
RETAINING WALL LOAD DIAGRAMS

*(Passive-Limit State)

Usual Load Case



Extreme Load Case (Seismic)



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USGS, Probabilistic Hazard Program, Version 3.10

Wall Dimensions

Wall height $H := 12.5ft$

Soil height (heel) $H_s := 12.5ft$

Soil height (toe) $H_{st} := 0.0ft$

Water height (heel) $H_{wh} := 5.0ft$

Water height (toe) $H_{wt} := 0.0ft$

Surcharge (construction) $S := 0ft$

Wall thickness $t := 2.75ft$

Footing thickness $d := 2.75ft$

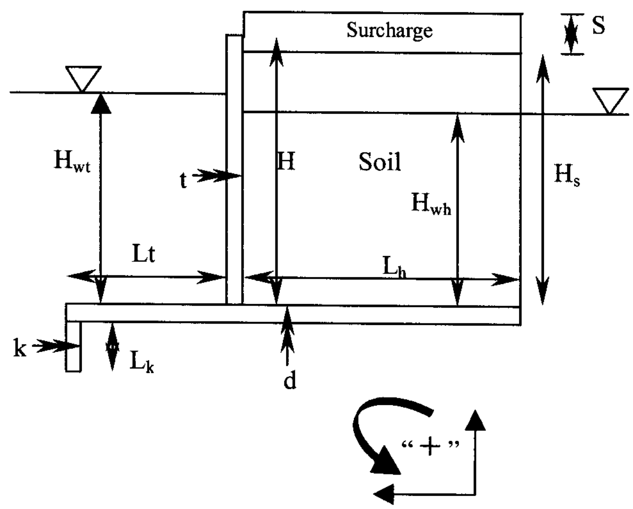
Key thickness $k := 0.0ft$

Heel length $L_h := 7.75ft$

Toe length $L_t := 3.0ft$

Key length $L_k := 0.0ft$

Footing Base Length $L_b := L_t + t + L_h$ $\quad L_b = 13.5\,ft$



Sheet Purpose:

The purpose of this calculation sheet is to complete an appraisel-level design for a full-scale cantilever retaining wall system for the purpose of potentially completing a dynamic shake table test on the LHPOST shake table located in San Diego, CA at the NEES Camp Elliott Facilities.

Assumptions:

These calculation were developed under the following assumptions based on an appraisel-level design effort:

1. *The proposed configuration based on the dimensions of the shake table is shown on Figures 1 and 2 - Attachment 1*
2. *The retaining wall and gravity boundary wall will be design using a worst case passive resistance earth pressure load. While this is a conservative approach, it is reasonable assumption for this appraisel level of development and allows for flexibility in terms of selecting earthquake magnitudes and corresponding time histories to be determined during later phases of the project.*
3. *The wall will be conservatively designed to structurally perform in the elastic range under the passive earth pressure conditions (see Assumption 2 above). When designing for passive earth pressure conditions, no load factors will be used as this is considered a limit state condition.*
4. *Several different soil types will be considered for conceptual-level and final design; however, for this level of study a internal friction angle of 30 degrees will be assumed with no cohesion will be assumed.*
5. *Wall friction will conservatively be neglected for this level of design.*
6. *A phreatic water surface five feet above the top of the footing will be assumed to address the possibility of performing a saturated backfill scenario.*
7. *The moist unit weight of the backfill will be 120 pcf and the saturated unit weight will be 130 pcf.*
8. *Incorporate precast panels lined with neoprene sheets to limit side friction in the non-shaking direction. The precast panels will allow for easier access to the backfill for removal and replacement to accomodate the potential for several test runs using different backfill configurations.*
9. *In the shaking direction, design one side as a flexible cantilever retaining wall and the other side as a stiffer gravity wall section for the purpose of limiting, to the extent possible, boundary effects. Target a relative natural period difference of an order of magnitude. The gravity wall could be replaced with a stiffer steel frame system during later design phases.*
10. *Anchor the concrete sections to the shake table using Nelson studs to simulate retaining systems founded on rock and to eliminate instability issues associated with sliding at the concrete steel table interface.*

Cantilever Retaining Wall Design:

Natural Period of Cantilever Wall

Use an allowable rational method in accordance with the requirements of ASCE 7, Section 9.14.5.4 to estimate period of walls. Based on *USBR Criteria for Concrete Retaining Walls*.

$T_{c1} := 0.000643 \cdot \left(\frac{H^2}{t}\right) \cdot \frac{1}{ft}$ $\qquad T_{c1} = 0.03653$

Compare with Eq. 9.5.5.3.2-1, ACSCE 7

$C_t := 0.02 \qquad x := 0.75 \qquad h_n := H \cdot \frac{1}{ft} \qquad h_n = 12.5$

$T_{c2} := C_t \cdot h_n^{\,x} \qquad T_{c2} = 0.13$

$T_{cant} := \min\left(T_{c1}, T_{c2}\right) \qquad T_{cant} = 0.0365$ sec

Uplift

Uplift at heel $\qquad U_h := \text{if}\left(H_{wh} > 0, H_{wh} + d, 0\right) \qquad U_h = 7.75\,ft$

Uplift at toe $\qquad U_t := \text{if}\left(H_{wt} > 0, H_{wt} + d, 0\right) \qquad U_t = 0\,ft$

Properties and Coefficients

Soil moist unit weight $\gamma := 120\frac{lb}{ft^2}$

Soil saturated unit weight $\gamma_s := 130\frac{lb}{ft^2}$

Concrete unit weight $\gamma_c := 150\frac{lb}{ft^2}$

Water unit weight $\gamma_w := 62.4\frac{lb}{ft^2}$

Slope of Backfill $\beta := 0deg$

Shear strength parameter $\phi := 30.0deg$

Frictional resistance $\delta := 24.2\cdot deg$ *Sliding resistance - friction resistance - see Attachment 2*

Cohesion $c := 0\frac{lb}{ft}$

Active pressure coefficient (Coulomb)

$$K_a := \frac{\cos(\phi)^2}{\left(1+\sqrt{\frac{\sin(\phi)\cdot\sin(\phi-\beta)}{\cos(\beta)}}\right)^2} \qquad K_a = 0.33$$

At-rest coefficient $K_o := 1 - \sin(\phi)$ $\qquad K_o = 0.5$

Passive pressure coefficient (Coulomb)

$$K_p := \frac{\cos(\phi)^2}{\left(1-\sqrt{\frac{\sin(\phi)\cdot\sin(\phi+\beta)}{\cos(\beta)}}\right)^2} \qquad K_p = 3$$

Seismic earth pressure coefficient $k_h := 0.576$ *Assume the maximum k_h value for M-O design (requires seismic intertia angle be less than or equal to phi angle) for load comparison to passive pressure design*

Seismic Inertia Angle $\psi := atan(k_h)$ $\qquad \psi = 29.94\cdot deg$

Angle of sliding failure plane

$$\theta := atan\left[\frac{L_k}{(L_t + t + L_h)}\right] \qquad \theta = 0\cdot deg$$

Allowable Bearing Pressure:

$$Q_{allowable} := 14400\frac{lb}{in^2} \qquad Q_{allowable} = 2073600\cdot\frac{lb}{ft^2} \qquad Q_{allowableEQ} := Q_{allowable} = 2073600\cdot\frac{lb}{ft^2}$$

Wall Forces

Wall section

$W_w := -\gamma_c \cdot H \cdot t$ $\quad x_w := L_t + \frac{t}{2}$ $\quad y_w := d + \frac{H}{2}$

$W_w = -5156.25\ lb$ $\quad x_w = 4.38\ ft$ $\quad y_w = 9\ ft$

Heel section

$W_h := -\gamma_c \cdot \left(L_h + \frac{t}{2}\right) \cdot d$ $\quad x_h := \frac{L_h + \frac{t}{2}}{2} + L_t + \frac{t}{2}$ $\quad y_h := \frac{d}{2}$

$W_h = -3764.063\ lb$ $\quad x_h = 8.94\ ft$ $\quad y_h = 1.38\ ft$

Toe section

$W_t := -\gamma_c \cdot \left(L_t + \frac{t}{2}\right) \cdot d$ $\quad x_t := \frac{L_t + \frac{t}{2}}{2}$ $\quad y_t := \frac{d}{2}$

$W_t = -1804.688\ lb$ $\quad x_t = 2.19\ ft$ $\quad y_t = 1.38\ ft$

Key section

$W_k := -\gamma_c \cdot L_k \cdot k$ $\quad x_k := \frac{k}{2}$ $\quad y_k := \frac{-L_k}{2}$

$W_k = 0$ $\quad x_k = 0$ $\quad y_k = 0$

Soil Static Forces (Coulomb Theory)

Vertical Forces

$W_s := -\gamma_s \cdot H_{wh} \cdot L_h + -\gamma \cdot (H_s - H_{wh}) \cdot L_h$ $\quad x_s := L_t + t + \frac{L_h}{2}$

$W_s = -12012.5\ lb$ $\quad x_s = 9.63\ ft$

$W_{wt} := -\gamma_w \cdot L_t \cdot H_{wt}$ $\quad y_{wt} := \frac{L_t}{2}$ $\quad y_{wt} = 1.5\ ft$

$W_{wt} = 0$

Surcharge Load for Construction Load Case

$W_{sSUR} := -\gamma \cdot (S) \cdot L_h$ $\quad x_{sSUR} := L_t + t + \frac{L_h}{2}$

$W_{sSUR} = 0$ $\quad x_{sSUR} = 9.63\ ft$

Horizontal Driving Forces - At-Rest Pressure

$$P_{D1o} := \text{if}\left[(H_{wh}) > 0, \frac{1}{2}\cdot K_o\cdot\gamma\cdot(H_s - H_{wh})^2, \frac{1}{2}\cdot K_o\cdot\gamma\cdot(H_s + d)^2\right] \qquad P_{D1o} = 1687.5\text{ lb}$$

$$y_{D1o} := \text{if}\left[H_{wh} > 0, H_{wh} + d + \frac{(H_s - H_{wh})}{3}, \frac{(H_s + d)}{3}\right] \qquad y_{D1o} = 10.25\text{ ft}$$

$$P_{D2o} := \text{if}\left[H_{wh} > 0, K_o\cdot\gamma\cdot(H_s - H_{wh})\cdot(H_{wh} + d), 0\right] \qquad P_{D2o} = 3487.5\text{ lb}$$

$$y_{D2o} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{2}, 0\right) \qquad y_{D2o} = 3.88\text{ ft}$$

$$P_{so} := \text{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_o\cdot(\gamma_s - \gamma_w)\cdot(H_{wh} + d)^2, 0\right] \qquad P_{so} = 1015.06\text{ lb}$$

$$y_{so} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{so} = 2.58\text{ ft}$$

$$P_{wo} := \text{if}\left[(H_{wh} > 0), \frac{1}{2}\cdot\gamma_w\cdot(H_{wh} + d)^2, 0\right] \qquad P_{wo} = 1873.95\text{ lb}$$

$$y_{wo} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{wo} = 2.58\text{ ft}$$

$$P_{SURo} := K_o\cdot\gamma\cdot S\cdot(H_s + d) \qquad P_{SURo} = 0$$

$$y_{SURo} := \frac{H_s + d}{2} \qquad y_{SURo} = 7.63\text{ ft}$$

Horizontal Resisting Forces - At-Rest Pressure

$$P_{spo} := \text{if}\left[H_{wt} > 0, \frac{-1}{2}\cdot K_o\cdot(\gamma_s - \gamma_w)\cdot(L_k + d + H_{st})^2, \left(\frac{-1}{2}\right)\cdot K_o\cdot\gamma\cdot(L_k + d + H_{st})^2\right] \qquad P_{spo} = -226.87\text{ lb}$$

$$P_{spo} := \text{if}(H_{st} > 0, P_{spo}, 0) \qquad P_{spo} = 0\text{ lb}$$

$$y_{spo} := \text{if}\left[H_{wt} > 0, (L_k + d + H_{st})\cdot\frac{1}{3}, 0\right] \qquad y_{spo} = 0\text{ ft}$$

$$P_{wpo} := \text{if}\left[H_{wt} > 0, \frac{-1}{2}\cdot\gamma_w\cdot(H_{wt} + d + L_k)^2, 0\right] \qquad P_{wpo} = 0\text{ lb}$$

$$y_{wpo} := \text{if}\left(H_{wt} > 0, \frac{H_{wt} + d + L_k}{3}, 0\right) \qquad y_{wpo} = 0\text{ ft}$$

Horizontal Driving Forces - Active Pressure

$$P_{D1a} := \text{if}\left[\left(H_{wh}\right) > 0, \frac{1}{2} \cdot K_a \cdot \gamma \cdot \left(H_s - H_{wh}\right)^2, \frac{1}{2} \cdot K_a \cdot \gamma \cdot \left(H_s + d\right)^2\right] \qquad P_{D1a} = 1125\,\text{lb}$$

$$y_{D1a} := \text{if}\left[H_{wh} > 0, H_{wh} + d + \frac{\left(H_s - H_{wh}\right)}{3}, \frac{\left(H_s + d\right)}{3}\right] \qquad y_{D1a} = 10.25\,\text{ft}$$

$$P_{D2a} := \text{if}\left[H_{wh} > 0, K_a \cdot \gamma \cdot \left(H_s - H_{wh}\right) \cdot \left(H_{wh} + d\right), 0\right] \qquad P_{D2a} = 2325\,\text{lb}$$

$$y_{D2a} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{2}, 0\right) \qquad y_{D2a} = 3.88\,\text{ft}$$

$$P_{sa} := \text{if}\left[H_{wh} > 0, \frac{1}{2} \cdot K_a \cdot \left(\gamma_s - \gamma_w\right) \cdot \left(H_{wh} + d\right)^2, 0\right] \qquad P_{sa} = 676.7\,\text{lb}$$

$$y_{sa} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{sa} = 2.58\,\text{ft}$$

$$P_{wa} := \text{if}\left[\left(H_{wh} > 0\right), \frac{1}{2} \cdot \gamma_w \cdot \left(H_{wh} + d\right)^2, 0\right] \qquad P_{wa} = 1873.95\,\text{lb}$$

$$y_{wa} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{wa} = 2.58\,\text{ft}$$

$P_{SURa} := K_a \cdot \gamma \cdot S \cdot \left(H_s + d\right)$ *construction surcharge* $\qquad P_{SURa} = 0$

$$y_{SURa} := \frac{H_s + d}{2} \qquad y_{SURa} = 7.63\,\text{ft}$$

Horizontal Resisting Forces - Passive Pressure

$$P_{sp} := \text{if}\left[H_{wt} > 0, \frac{-1}{2} \cdot K_p \cdot \left(\gamma_s - \gamma_w\right) \cdot \left(L_k + d + H_{st}\right)^2, \left(\frac{-1}{2}\right) \cdot K_p \cdot \gamma \cdot \left(L_k + d + H_{st}\right)^2\right] \qquad P_{sp} = -1361.25\,\text{lb}$$

$$P_{sp} := \text{if}\left(H_{st} > 0, P_{sp}, 0\right) \qquad P_{sp} = 0\,\text{lb}$$

$$y_{sp} := \text{if}\left[H_{wt} > 0, \left(L_k + d + H_{st}\right) \cdot \frac{1}{3}, 0\right] \qquad y_{sp} = 0\,\text{ft}$$

$$P_{wp} := \text{if}\left[H_{wt} > 0, \frac{-1}{2} \cdot \gamma_w \cdot \left(H_{wt} + d + L_k\right)^2, 0\right] \qquad P_{wp} = 0\,\text{lb}$$

$$y_{wp} := \text{if}\left(H_{wt} > 0, \frac{H_{wt} + d + L_k}{3}, 0\right) \qquad y_{wp} = 0\,\text{ft}$$

Earthquake Forces (Mononobe-Okabe Method)

K_{AE} is the active pressure coefficient for earthquake loading

$$K_{AE} := \frac{\cos(\phi - \psi)^2}{\cos(\psi)\cdot\cos(\psi)\left(1 + \sqrt{\frac{\sin(\phi)\cdot\sin(\phi - \beta - \psi)}{\cos(\psi)\cdot\cos(\beta)}}\right)^2} \qquad K_{AE} = 1.27$$

K_{PE} is the passive pressure coefficient for earthquake loading

$$K_{PE} := \frac{\cos(\phi - \psi)^2}{\cos(\psi)\cdot\cos(\psi)\cdot\left(1 - \sqrt{\frac{\sin(\phi)\cdot\sin(\phi - \beta - \psi)}{\cos(\psi)\cdot\cos(\beta)}}\right)^2} \qquad K_{PE} = 1.4$$

Total Horizontal Driving Seismic Forces - Active Pressure

(USACOE, EM 1110-2-2502, PAGE 3-64)

$$P_{D1AE} := \text{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_{AE}\cdot\gamma\cdot\left(H_s - H_{wh}\right)^2, \frac{1}{2}\cdot K_{AE}\cdot\gamma\cdot\left(H_s + d\right)^2\right] \qquad P_{D1AE} = 4285\,\text{lb}$$

$$P_{D2AE} := \text{if}\left[H_{wh} > 0, K_{AE}\cdot\gamma\cdot\left(H_s - H_{wh}\right)\cdot\left(H_{wh} + d\right), 0\right] \qquad P_{D2AE} = 8856\,\text{lb}$$

$$P_{sAE} := \text{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_{AE}\cdot\left(\gamma_s - \gamma_w\right)\cdot\left(H_{wh} + d\right)^2, 0\right] \qquad P_{sAE} = 2577\,\text{lb}$$

Dynamic Component of Driving Seismic Forces - Active Pressure (Kramer, Chapter 11)

$$\Delta P_{AE1} := \left(P_{D1AE} - P_{D1a}\right) + \left(P_{D2AE} - P_{D2a}\right) \qquad \Delta P_{AE1} = 9690.69\,\text{lb}$$

$$y_{AE1} := \frac{2\cdot\left(H_s + d\right)}{3} \qquad y_{AE1} = 10.17\,\text{ft}$$

$$\Delta P_{AE2} := P_{sAE} - P_{sa} \qquad \Delta P_{AE2} = 1900.79\,\text{lb}$$

$$y_{AE2} := \text{if}\left[H_{wh} > 0, \frac{2\cdot\left(H_{wh} + d\right)}{3}, 0\right] \qquad y_{AE2} = 5.17\,\text{ft}$$

Total Horizontal Seismic Resistng Forces - Passive Pressure (USACOE, EM 1110-2-2502, PAGE 3-64)

$$P_{sP} := \text{if}\left[H_{wt} > 0, \frac{-1}{2}\cdot K_{PE}\cdot(\gamma_s - \gamma_w)\cdot(L_k + d + H_{st})^2, \frac{-1}{2}\cdot K_{PE}\cdot\gamma\cdot(H_{st} + d)^2\right] \qquad P_{sP} = -634.61\ \text{lb}$$

$$P_{sP} := \text{if}(H_{st} > 0, P_{sP}, 0) \qquad P_{sP} = 0\ \text{lb}$$

$$y_{sP} := \text{if}\left(H_{st} > 0, \frac{L_k + d + H_{st}}{3}, 0\right) \qquad y_{sP} = 0\ \text{ft}$$

Dynamic Component of Resisting Seismic Forces - Passive Pressure

(Kramer, Chapter 11)

$$\Delta P_{PE} := P_{sP} - P_{sp} \qquad \Delta P_{PE} = 0\ \text{lb}$$

$$y_{PE} := \text{if}\left[H_{st} > 0, \frac{2\cdot(L_k + d + H_{st})}{3}, 0\right] \qquad y_{PE} = 0\ \text{ft}$$

Horizontal Driving Forces - Passive Pressure - Limit State Earthquake

$$P_{D1p} := \text{if}\left[(H_{wh}) > 0, \frac{1}{2}\cdot K_p\cdot\gamma\cdot(H_s - H_{wh})^2, \frac{1}{2}\cdot K_p\cdot\gamma\cdot(H_s + d)^2\right] \qquad P_{D1p} = 10125\ \text{lb}$$

$$y_{D1p} := \text{if}\left[H_{wh} > 0, H_{wh} + d + \frac{(H_s - H_{wh})}{3}, \frac{(H_s + d)}{3}\right] \qquad y_{D1p} = 10.25\ \text{ft}$$

$$P_{D2p} := \text{if}\left[H_{wh} > 0, K_p\cdot\gamma\cdot(H_s - H_{wh})\cdot(H_{wh} + d), 0\right] \qquad P_{D2p} = 20925\ \text{lb}$$

$$y_{D2p} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{2}, 0\right) \qquad y_{D2p} = 3.88\ \text{ft}$$

$$P_{spd} := \text{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_p\cdot(\gamma_s - \gamma_w)\cdot(H_{wh} + d)^2, 0\right] \qquad P_{spd} = 6090.34\ \text{lb}$$

$$y_{spd} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{spd} = 2.58\ \text{ft}$$

$$P_{wpd} := \text{if}\left[(H_{wh} > 0), \frac{1}{2}\cdot\gamma_w\cdot(H_{wh} + d)^2, 0\right] \qquad P_{wpd} = 1873.95\ \text{lb}$$

$$y_{wpd} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{wpd} = 2.58\ \text{ft}$$

$P_{SURp} := K_p\cdot\gamma\cdot S\cdot(H_s + d)$ *construction surcharge* $\qquad P_{SURp} = 0$

$$y_{SURp} := \frac{H_s + d}{2} \qquad y_{SURp} = 7.63\ \text{ft}$$

Horizontal Concrete Inertial Forces (ASCE 7, Section 9.14.5)

$k_h := 1.0$ *compute inertial forces based on test peak of 1.0g*

$W_{he} := -k_h \cdot W_h$ $\quad y_{he} := y_h$

$W_{he} = 3764.06\ \text{lb}$ $\quad y_{he} = 1.38\ \text{ft}$

$W_{we} := -k_h \cdot W_w$ $\quad y_{we} := y_w$

$W_{we} = 5156.25\ \text{lb}$ $\quad y_{we} = 9\ \text{ft}$

$W_{te} := -k_h \cdot W_t$ $\quad y_{te} := y_t$

$W_{te} = 1804.69\ \text{lb}$ $\quad y_{te} = 1.38\ \text{ft}$

$W_{ke} := -k_h \cdot W_k$ $\quad y_{ke} := y_k$

$W_{ke} = 0$ $\quad y_{ke} = 0$

Horizontal Soil Inertial Forces

$W_{sEq} := -k_h \cdot W_{sSUR}$ $\quad y_{sEq} := d + \frac{H_s}{2}$

$W_{sEq} = 0$ $\quad y_{sEq} = 9\ \text{ft}$

Hydrodynamic Force - Westergaard's Equation

Load for free pore water conditions in silty sand and gravel backfill or free standing water.
Reference USACE EM 1110-2-2502, page 3-78

$$C_e := 51.0 \cdot \frac{\text{lb}}{\text{ft}^2}$$

$P_{Re} := \frac{2}{3} \cdot C_e \cdot k_h \cdot H_{wh}^{\ 2}$ $\quad y_{Re} := .4 \cdot H_{wh} + d$

$P_{Re} = 850\ \text{lb}$ $\quad y_{Re} = 4.75\ \text{ft}$

$P_{He} := \frac{2}{3} \cdot C_e \cdot k_h \cdot H_{wt}^{\ 2}$ $\quad y_{He} := .4 \cdot H_{wt} + d$

$P_{He} = 0$ $\quad y_{He} = 2.75\ \text{ft}$

Uplift Forces

$U_1 := \gamma_w \cdot \frac{1}{2} \cdot (U_h - U_t) \cdot L_b$ $\quad x_{U1} := \frac{2}{3} \cdot L_b$

$U_1 = 3264.3\ \text{lb}$ $\quad x_{U1} = 9\ \text{ft}$

$U_2 := \gamma_w \cdot U_t \cdot L_b$ $\quad x_{U2} := \frac{1}{2} \cdot L_b$

$U_2 = 0\ \text{lb}$ $\quad x_{U2} = 6.75\ \text{ft}$

Moments About Toe

Conservatively check overturning neglecting cutoff wall key.

Resisting Moment

Usual Load Case (At-Rest Pressure)

$$M_{ro} := W_w \cdot x_w + W_h \cdot x_h + W_t \cdot x_t + W_s \cdot x_s + P_{wpo} \cdot y_{wpo} + W_{wt} \cdot y_{wt} + P_{spo} \cdot y_{spo} \qquad M_{ro} = -175768\ \text{lb}\cdot\text{ft}$$

Construction Load Case (Active Pressure)

$$M_{ra} := W_w \cdot x_w + W_h \cdot x_h + W_t \cdot x_t + W_s \cdot x_s + P_{wp} \cdot y_{wp} + W_{wt} \cdot y_{wt} + P_{sp} \cdot y_{sp} + W_{sSUR} \cdot x_{sSUR}$$

$$M_{ra} = -175768\ \text{lb}\cdot\text{ft}$$

Seismic Load Case (Active Pressure)

$$M_{Er} := W_w \cdot x_w + W_h \cdot x_h + W_t \cdot x_t + W_s \cdot x_s + P_{wp} \cdot y_{wp} + W_{wt} \cdot y_{wt} + P_{sp} \cdot y_{sp} + \Delta P_{PE} \cdot y_{PE}$$

$$M_{Er} = -175768\ \text{lb}\cdot\text{ft}$$

Overturning Moment

Usual Load Case (At-Rest Pressure)

$$M_{oo} := P_{D1o} \cdot y_{D1o} + P_{D2o} \cdot y_{D2o} + P_{so} \cdot y_{so} + P_{wo} \cdot y_{wo} + U_1 \cdot x_{U1} + U_2 \cdot x_{U2} \qquad M_{oo} = 67653\ \text{lb}\cdot\text{ft}$$

Construction Load Case (Active Pressure)

$$M_{oa} := P_{D1a} \cdot y_{D1a} + P_{D2a} \cdot y_{D2a} + P_{sa} \cdot y_{sa} + P_{wa} \cdot y_{wa} + P_{SURa} \cdot y_{SURa} + U_1 \cdot x_{U1} + U_2 \cdot x_{U2}$$

$$M_{oa} = 56509\ \text{lb}\cdot\text{ft}$$

Seismic Load Case (Active Pressure)

$$M_{Eo1} := \Delta P_{AE1} \cdot y_{AE1} + \Delta P_{AE2} \cdot y_{AE2} + P_{D1a} \cdot y_{D1a} + P_{D2a} \cdot y_{D2a} + P_{sa} \cdot y_{sa} + P_{wa} \cdot y_{wa}$$

$$M_{Eo2} := P_{Re} \cdot y_{Re} + U_1 \cdot x_{U1} + U_2 \cdot x_{U2} + W_{we} \cdot y_{we} + W_{he} \cdot y_{he} + W_{te} \cdot y_{te} + W_{sEq} \cdot y_{sEq} + P_{He} \cdot y_{He}$$

$$M_{Eo} := M_{Eo1} + M_{Eo2} \qquad M_{Eo} = 222952.11\ \text{lb}\cdot\text{ft}$$

Seismic Limit State Case (Passive Pressure)

$$M_{Ep1} := P_{D1p} \cdot y_{D1p} + P_{D2p} \cdot y_{D2p} + P_{spd} \cdot y_{spd} + P_{wpd} \cdot y_{wpd} + P_{SURp} \cdot y_{SURp}$$

$$M_{Ep2} := P_{Re} \cdot y_{Re} + U_1 \cdot x_{U1} + U_2 \cdot x_{U2} + W_{we} \cdot y_{we} + W_{he} \cdot y_{he} + W_{te} \cdot y_{te} + W_{sEq} \cdot y_{sEq} + P_{He} \cdot y_{He}$$

$$M_{Ep} := M_{Ep1} + M_{Ep2} \qquad M_{Ep} = 292919.52\ \text{lb}\cdot\text{ft}$$

ΣF_y Vertical Force

$$F_y := W_w + W_h + W_t + W_s + U_1 + U_2 + W_{wt} \qquad F_y = -19473\ \text{lb}$$ Usual & Seismic Load

$$F_{yC} := W_w + W_h + W_t + W_s + U_1 + U_2 + W_{wt} + W_{sSUR} \qquad F_{yC} = -19473\ \text{lb}$$ Construction Load

Location of Resultant - Usual Load Case (At-Rest Pressure)

$$\text{Resultant} := \frac{(M_{ro} + M_{oo})}{F_y} \qquad \text{Resultant} = 5.55\,\text{ft} \qquad OT_{NLC} := \text{if}\left(\text{Resultant} \geq \frac{L_b}{3}, \text{"YES"}, \text{"NO"}\right)$$

Resultant w/in middle third of base: $OT_{NLC} = \text{"YES"}$

$$\text{Eccentricity} := \frac{L_b}{2} - \text{Resultant} \qquad \text{Eccentricity} = 1.2\,\text{ft}$$

Stress Distribution Under the Footing Usual Load Case (At-Rest Pressure)

$$\sigma_1 := \frac{F_y}{L_b}\cdot\left(1 + \frac{6\cdot\text{Eccentricity}}{L_b}\right) \qquad \sigma_1 = -2210\,\frac{\text{lb}}{\text{ft}}$$

$$\sigma_2 := \frac{F_y}{L_b}\cdot\left(1 - \frac{6\cdot\text{Eccentricity}}{L_b}\right) \qquad \sigma_2 = -674\,\frac{\text{lb}}{\text{ft}}$$

$$b := \frac{3}{2}\cdot(L_b - 2\cdot\text{Eccentricity}) \qquad x_{tenso} := \text{if}(\sigma_2 > 0, b, 0) \qquad x_{tenso} = 0\,\text{ft}$$

$$\sigma_{toefaceo} := \text{if}\left(x_{tenso} > 0, \frac{4}{3}\,\frac{F_y}{L_b - 2\cdot\text{Eccentricity}}, \sigma_1\right) \qquad \sigma_{toefaceo} = -2210\,\frac{\text{lb}}{\text{ft}}$$

The stress at the toe

$$\sigma_{heelfaceo} := \text{if}(x_{tenso} > 0, 0, \sigma_2) \qquad \sigma_{heelfaceo} = -674\,\frac{\text{lb}}{\text{ft}}$$

The stress at the heel

$$bp_{oslope} := \text{if}\left(x_{tenso} > 0, \frac{\sigma_{toefaceo} - \sigma_{heelfaceo}}{b}, \frac{\sigma_{toefaceo} - \sigma_{heelfaceo}}{L_b}\right) \qquad bp_{oslope} = -113.78\,\text{lb}\cdot\text{ft}^{-2}$$

$$\sigma_{mido} := -bp_{oslope}\cdot\left(L_t + \frac{t}{2}\right) + \sigma_{toefaceo} \qquad \sigma_{mido} = -1713\,\frac{\text{lb}}{\text{ft}}$$

The stress at the centerline of the wall stem

Overturning Factor of Safety (Usual Load Case - At-Rest Pressure)

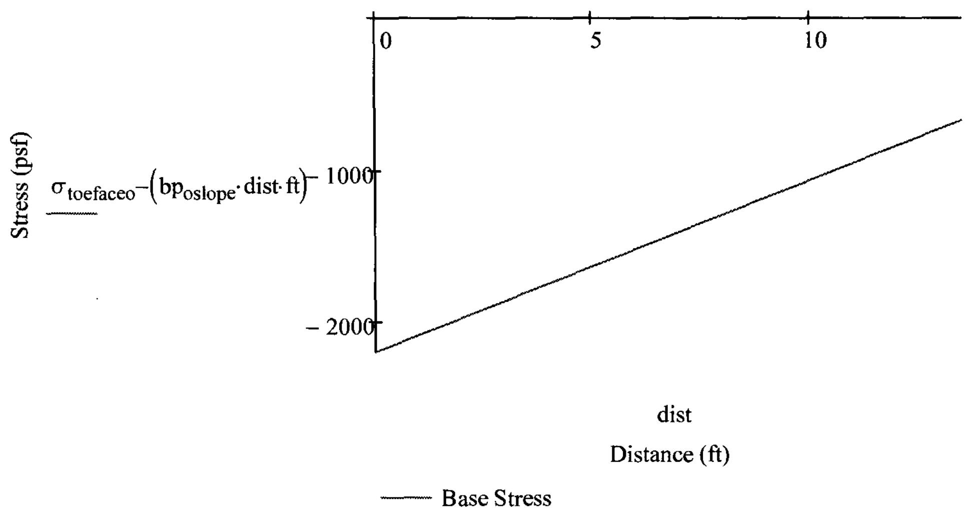
$$FS_{OT} := \frac{-M_{ro}}{M_{oo}} \qquad FS_{OT} = 2.6 \qquad OT_{result} := \text{if}(FS_{OT} < 1.5, \text{"NO GOOD"}, \text{"OKAY"})$$

$$OT_{result} = \text{"OKAY"}$$

Bearing Capacity Check

$$\text{Bearing}_{NLC} := \text{if}\left[(Q_{allowable}\cdot\text{ft}) \geq -\sigma_{toefaceo}, \text{"OKAY"}, \text{"NG"}\right] \qquad \text{Bearing}_{NLC} = \text{"OKAY"}$$

Usual Load Case -Base Stress Distribution (At-Rest Pressures)



Location of Resultant - Construction Load Case (Active Pressure)

$Resultant := \frac{(M_{ra} + M_{oa})}{F_{yC}}$ $\quad Resultant = 6.12\, ft \quad OT_{NLC} := if\left(Resultant \geq \frac{L_b}{3}, "YES", "NO"\right)$

Result w/in middle third of base: $\quad OT_{NLC} = "YES" \quad \frac{L_b}{3} = 4.5\, ft$

$Eccentricity := \frac{L_b}{2} - Resultant \qquad Eccentricity = 0.63\, ft$

Stress Distribution Under the Footing Construction Load Case (Active Pressure)

$$\sigma_1 := \frac{F_{yC}}{L_b}\cdot\left(1 + \frac{6\cdot Eccentricity}{L_b}\right) \qquad \sigma_1 = -1844\,\frac{lb}{ft}$$

$$\sigma_2 := \frac{F_{yC}}{L_b}\cdot\left(1 - \frac{6\cdot Eccentricity}{L_b}\right) \qquad \sigma_2 = -1041\,\frac{lb}{ft}$$

$b_a := \frac{3}{2}\cdot(L_b - 2\cdot Eccentricity) \qquad x_{tensa} := if(\sigma_2 > 0, b_a, 0) \qquad x_{tensa} = 0\, ft$

$\sigma_{toefacea} := if\left(x_{tenso} > 0, \frac{4}{3}\,\frac{F_y}{L_b - 2\cdot Eccentricity}, \sigma_1\right) \quad \sigma_{toefacea} = -1844\,\frac{lb}{ft}$ *The stress at the toe*

$\sigma_{heelfacea} := if(x_{tensa} > 0, 0, \sigma_2) \qquad \sigma_{heelfacea} = -1041\,\frac{lb}{ft}$ *The stress at the heel*

$$bp_{aslope} := \text{if}\left(x_{tensa} > 0, \frac{\sigma_{toefacea} - \sigma_{heelfacea}}{b_a}, \frac{\sigma_{toefacea} - \sigma_{heelfacea}}{L_b}\right) \qquad bp_{aslope} = -59.43\, lb \cdot ft^{-2}$$

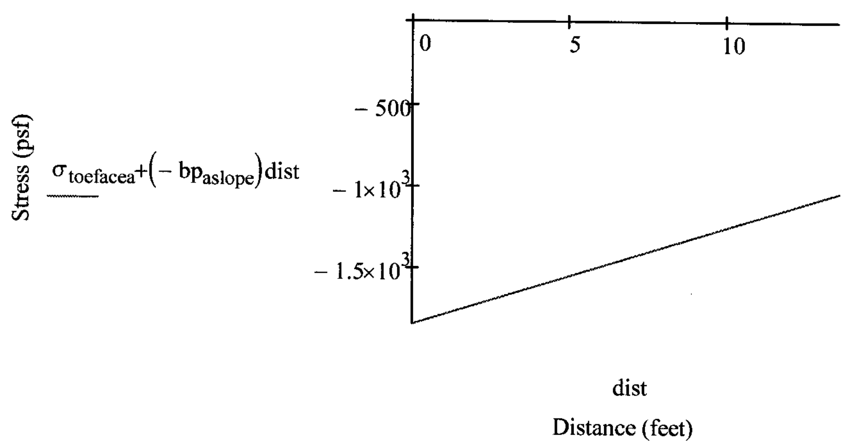
$$\sigma_{mida} := -bp_{aslope} \cdot \left(L_t + \frac{t}{2}\right) + \sigma_{toefacea} \qquad \sigma_{mida} = -1584\frac{lb}{ft}$$

The stress at the centerline of the wall stem

Bearing Capacity Check

$$Bearing_{NLC} := \text{if}\left[\left(Q_{allowable} \cdot 1ft\right) \ge -\sigma_{toefacea}, \text{"OKAY"}, \text{"NG"}\right] \qquad Bearing_{NLC} = \text{"OKAY"}$$

Construction Load Case -Base Stress Distribution (Active Pressures)



Location of Resultant - Seismic Load Case (Active Pressure)

$$Resultant_E := \frac{\left(M_{Er} + M_{Eo}\right)}{F_y} \qquad Resultant_E = -2.42\, ft \qquad OT_{ELC} := \text{if}\left(Resultant_E \ge 0, \text{"YES"}, \text{"NO"}\right)$$

Result w/in base: $OT_{ELC} = \text{"NO"}$

$$Eccentricity_E := \frac{L_b}{2} - Resultant_E \qquad Eccentricity_E = 9.17\, ft$$

Stress distribution Under the Footing (With Earthquake)

$$\sigma_{1E} := \frac{F_y}{L_b} \cdot \left(1 + \frac{6 \cdot Eccentricity_E}{L_b}\right) \qquad \sigma_{1E} = -7323\frac{lb}{ft}$$

$$\sigma_{2E} := \frac{F_y}{L_b} \cdot \left(1 - \frac{6 \cdot Eccentricity_E}{L_b}\right) \qquad \sigma_{2E} = 4438\frac{lb}{ft}$$

$b_E := \frac{3}{2}\cdot\left(L_b - 2\cdot Eccentricity_E\right)$ $\quad x_{tensE} := if\left(\sigma_{2E} > 0, b_E, 0\right)$ $\quad x_{tensE} = -7.27\,ft$ ***Resultant not within base - NG - Need Stud Anchors***

$\sigma_{toefaceE} := if\left(x_{tensE} > 0, \frac{4}{3}\frac{F_y}{L_b - 2\cdot Eccentricity}, \sigma_{1E}\right)$ $\quad \sigma_{toefaceE} = -7323\frac{lb}{ft}$ *The stress at the toe*

$\sigma_{heelfaceE} := if\left(x_{tensE} > 0, 0, \sigma_{2E}\right)$ $\quad \sigma_{heelfaceE} = 4438\frac{lb}{ft}$ *The stress at the heel*

$bp_{Eslope} := if\left(x_{tensE} > 0, \frac{\sigma_{toefaceE} - \sigma_{heelfaceE}}{b_E}, \frac{\sigma_{toefaceE} - \sigma_{heelfaceE}}{L_b}\right)$ $\quad bp_{Eslope} = -871.22\,lb\cdot ft^{-2}$

$\sigma_{midE} := -bp_{Eslope}\cdot\left(L_t + \frac{t}{2}\right) + \sigma_{toefaceE}$ $\quad \sigma_{midE} = -3512\frac{lb}{ft}$ *The stress at the centerline of the wall stem*

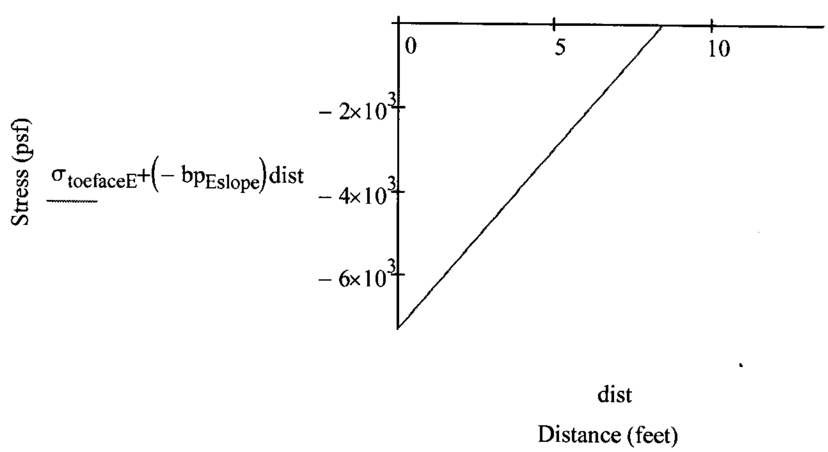
Bearing Factor of Safety

$FS_{bE} := \frac{-Q_{allowableEQ}\cdot 1\,ft}{\sigma_{toefaceE}}$ $\quad FS_{bE} = 283.15$ > 1.5 required **OK**

Bearing Capacity Check

$Bearing_{ELC} := if\left[\left(Q_{allowableEQ}\cdot 1\,ft\right) \geq -\sigma_{toefaceE}, \text{"OKAY"}, \text{"NG"}\right]$ $\quad Bearing_{ELC} = \text{"OKAY"}$

Extreme Load Case - Base Stress Distribution - Active Pressures



Location of Resultant - Seismic Limit State Case (Passive Pressure)

$$Resultant_{EL} := \frac{(M_{Er} + M_{Ep})}{F_y} \qquad Resultant_{EL} = -6.02\,ft \qquad OT_{EL} := if(Resultant_{EL} \geq 0, "YES", "NO")$$

Result w/in base: $OT_{EL} = "NO"$

$$Eccentricity_{EL} := \frac{L_b}{2} - Resultant_{EL} \qquad Eccentricity_{EL} = 12.77\,ft$$

Stress distribution Under the Footing (With Earthquake)

$$\sigma_{1EL} := \frac{F_y}{L_b} \cdot \left(1 + \frac{6 \cdot Eccentricity_{EL}}{L_b}\right) \qquad \sigma_{1EL} = -9627\,\frac{lb}{ft}$$

$$\sigma_{2EL} := \frac{F_y}{L_b} \cdot \left(1 - \frac{6 \cdot Eccentricity_{EL}}{L_b}\right) \qquad \sigma_{2EL} = 6742\,\frac{lb}{ft}$$

$$b_{EL} := \frac{3}{2} \cdot (L_b - 2 \cdot Eccentricity_{EL}) \quad x_{tensEL} := if(\sigma_{2EL} > 0, b_{EL}, 0) \quad x_{tensEL} = -18.05\,ft$$

Resultant not within base - NG - Need Stud Anchors

$$\sigma_{toefaceEL} := if\left(x_{tensEL} > 0, \frac{4}{3} \frac{F_y}{L_b - 2 \cdot Eccentricity}, \sigma_{1EL}\right) \quad \sigma_{toefaceEL} = -9627\,\frac{lb}{ft}$$

The stress at the toe

$$\sigma_{heelfaceEL} := if(x_{tensEL} > 0, 0, \sigma_{2EL}) \quad \sigma_{heelfaceEL} = 6742\,\frac{lb}{ft}$$

The stress at the heel

$$bp_{ELslope} := if\left(x_{tensEL} > 0, \frac{\sigma_{toefaceEL} - \sigma_{heelfaceEL}}{b_{EL}}, \frac{\sigma_{toefaceEL} - \sigma_{heelfaceEL}}{L_b}\right) \quad bp_{ELslope} = -1212.48\,lb \cdot ft^{-2}$$

$$\sigma_{midEL} := -bp_{ELslope} \cdot \left(L_t + \frac{t}{2}\right) + \sigma_{toefaceEL} \quad \sigma_{midEL} = -4322\,\frac{lb}{ft}$$

The stress at the centerline of the wall stem

Bearing Factor of Safety

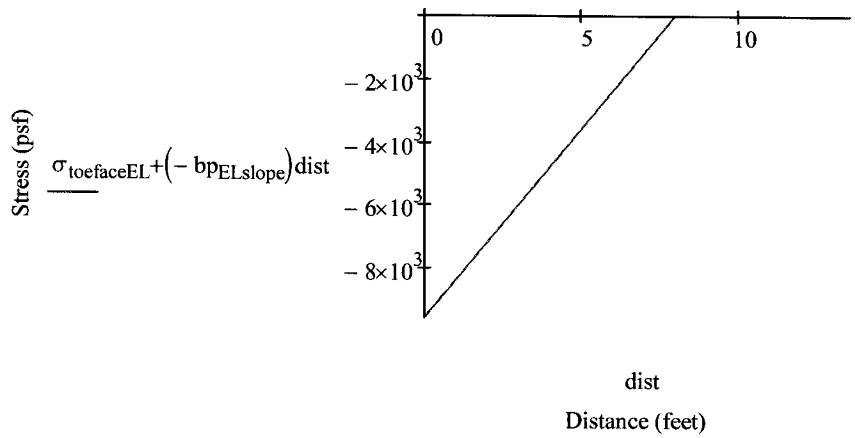
$$FS_{bEL} := \frac{-Q_{allowableEQ} \cdot 1\,ft}{\sigma_{toefaceEL}} \qquad FS_{bEL} = 215.4$$

> 1.5 required **OK**

Bearing Capacity Check

$$Bearing_{EL} := if[(Q_{allowableEQ} \cdot 1\,ft) \geq -\sigma_{toefaceEL}, "OKAY", "NG"] \qquad Bearing_{EL} = "OKAY"$$

Extreme Limit State Case - Base Stress Distribution - Passive Pressures



Summation of Forces (USACOE, EM 1110-2-2502, Page 4-26)

Check sliding including benefit of cutoff wall (if applicable).

$F_c := \cos(\theta)$ $\quad F_c = 1$ $\quad F_s := \sin(\theta)$ $\quad F_s = 0$

ΣF_y Vertical Force for Sliding - Usual Load Case (At-Rest Pressures)

$F_{y1o} := (W_w + W_h + W_t + W_s + W_k + U_1 + U_2 + W_{wt})\cdot F_c$ $\quad F_{y1o} = -19473\ lb$

$F_{y2o} := (P_{D1o} + P_{D2o} + P_{so} + P_{wo} + P_{so} + P_{wpo} + P_{spo})\cdot F_s$ $\quad F_{y2o} = 0\ lb$

$F_{yso} := F_{y1o} + F_{y2o}$ $\quad F_{yso} = -19473.2\ lb$

ΣF_y Vertical Force for Sliding - Construction Load Case (Actvie Pressures)

$F_{y1a} := (W_w + W_h + W_t + W_s + W_k + U_1 + U_2 + W_{wt} + W_{sSUR})\cdot F_c$ $\quad F_{y1a} = -19473\ lb$

$F_{y2a} := (P_{D1a} + P_{D2a} + P_{sa} + P_{wa} + P_{SURa} + P_{wa} + P_{sa})\cdot F_s$ $\quad F_{y2a} = 0\ lb$

$F_{ysa} := F_{y1a} + F_{y2a}$ $\quad F_{ysa} = -19473.2\ lb$

ΣF_y Vertical Force for Sliding - Seismic Load Case (Active Pressures)

$F_{y1E} := (W_w + W_h + W_t + W_s + W_k + U_1 + U_2 + W_{wt})\cdot F_c$ $\quad F_{y1E} = -19473\ lb$

$F_{y2E} := (P_{D1a} + P_{D2a} + P_{sa} + P_{wa} + P_{Re} + P_{He} + P_{wp} + P_{sp})\cdot F_s$ $\quad F_{y2E} = 0\ lb$

$F_{y3E} := (W_{we} + W_{he} + W_{we} + W_{te} + W_{ke} + W_s + \Delta P_{AE1} + \Delta P_{AE2})\cdot F_s$ $\quad F_{y3E} = 0\ lb$

$F_{ysE} := F_{y1E} + F_{y2E} + F_{y3E}$ $\quad F_{ysE} = -19473.2\ lb$

ΣF_y Vertical Force for Sliding - Seismic Limit State Case (Passive Pressures)

$F_{y1EL} := (W_w + W_h + W_t + W_s + W_k + U_1 + U_2 + W_{wt}) \cdot F_c$ $\quad F_{y1EL} = -19473\,lb$

$F_{y2EL} := (P_{D1a} + P_{D2a} + P_{sa} + P_{wa} + P_{Re} + P_{He} + P_{wp} + P_{sp}) \cdot F_s$ $\quad F_{y2EL} = 0\,lb$

$F_{y3EL} := (W_{we} + W_{he} + W_{we} + W_{te} + W_{ke} + W_s + \Delta P_{AE1} + \Delta P_{AE2}) \cdot F_s$ $\quad F_{y3EL} = 0\,lb$

$F_{ysEL} := F_{y1E} + F_{y2E} + F_{y3E}$ $\quad F_{ysEL} = -19473\,lb$

ΣF_x Horizontal Force - Usual Load Case (At-Rest Pressures)

$F_{x1o} := (P_{D1o} + P_{D2o} + P_{so} + P_{wo} + P_{so} + P_{wpo} + P_{spo}) \cdot F_c$ $\quad F_{x1o} = 9079\,lb$

$F_{x2o} := (-W_w - W_h - W_t - W_s - W_k - U_1 - U_2 - W_{wt}) \cdot F_s$ $\quad F_{x2o} = 0\,lb$

$F_{xo} := F_{x1o} + F_{x2o}$ $\quad F_{xo} = 9079\,lb$

ΣF_x Horizontal Force - Construction Load Case (Active Pressures)

$F_{x1a} := (P_{D1a} + P_{D2a} + P_{sa} + P_{wa} + P_{SURa} + P_{wp} + P_{sp}) \cdot F_c$ $\quad F_{x1a} = 6001\,lb$

$F_{x2a} := (-W_w - W_h - W_t - W_s - W_k - U_1 - U_2 - W_{wt} - W_{sSUR}) \cdot F_s$ $\quad F_{x2a} = 0\,lb$

$F_{xa} := F_{x1a} + F_{x2a}$ $\quad F_{xa} = 6001\,lb$

ΣF_x Horizontal Force - Seismic Load Case (Active Pressures)

$F_{Ex1} := (\Delta P_{AE1} + \Delta P_{AE2} + P_{wa} + P_{sa} + P_{D1a} + P_{D2a} + P_{Re} + P_{He} + P_{sp} + \Delta P_{PE} + P_{wp}) \cdot F_c$

$F_{Ex1} = 18442.14\,lb$

$F_{Ex2} := (W_{we} + W_{he} + W_{te} + W_{ke} + W_{sEq}) \cdot F_c$ $\quad F_{Ex2} = 10725\,lb$

$F_{Ex3} := (-W_w - W_h - W_t - W_s - W_k - U_1 - U_2 - W_{wt}) \cdot F_s$ $\quad F_{Ex3} = 0\,lb$

$F_{Ex} := F_{Ex1} + F_{Ex2} + F_{Ex3}$ $\quad F_{Ex} = 29167\,lb$

ΣF_x Horizontal Force - Seismic Limit State Case (Passive Pressures)

$F_{ExL1} := (P_{wpd} + P_{spd} + P_{D1p} + P_{D2p} + P_{Re} + P_{He} + P_{sp} + \Delta P_{PE} + P_{wp}) \cdot F_c$ $\quad F_{ExL1} = 39864\,lb$

$F_{ExL2} := (W_{we} + W_{he} + W_{te} + W_{ke} + W_{sEq}) \cdot F_c$ $\quad F_{ExL2} = 10725\,lb$

$F_{ExL3} := (-W_w - W_h - W_t - W_s - W_k - U_1 - U_2 - W_{wt}) \cdot F_s$ $\quad F_{ExL3} = 0\,lb$

$F_{ExL} := F_{ExL1} + F_{ExL2} + F_{ExL3}$ $\quad F_{ExL} = 50589\,lb$

Sliding Factor of Safety: Usual Load Case (At-Rest Pressures)

$$FS_s := \frac{c\cdot(L_b - x_{tenso}) + \tan(\delta)\cdot(-F_{yso})}{F_{xo}} \qquad FS_s := \mathrm{round}(FS_s, 1) \qquad FS_s = 1$$

Acceptable 1.0

$Slide_{NLC} := \mathrm{if}(FS_s \geq 1.0, \text{"OKAY"}, \text{"NG"}) \qquad Slide_{NLC} = \text{"OKAY"}$

Sliding Factor of Safety: Construction Load Case (Active Pressures)

$$FS_{sa} := \frac{c\cdot(L_b - x_{tensa}) + \tan(\delta)\cdot(-F_{ysa})}{F_{xa}} \qquad FS_{sa} := \mathrm{round}(FS_{sa}, 1) \qquad FS_{sa} = 1.5$$

Acceptable 1.5

$Slide_{NLCa} := \mathrm{if}(FS_{sa} \geq 1.5, \text{"OKAY"}, \text{"NG"}) \qquad Slide_{NLCa} = \text{"OKAY"}$

Sliding Factor of Safety: Seismic Active Pressure

$$FS_{sE} := \frac{c\cdot(L_b - x_{tensE}) + \tan(\delta)\cdot(-F_{ysE})}{F_{Ex}} \qquad FS_{sE} := \mathrm{round}(FS_{sE}, 1) \qquad FS_{sE} = 0.3$$

Acceptable 1.1

$Slide_{ELC} := \mathrm{if}(FS_{sE} \geq 1.0, \text{"OKAY"}, \text{"NG"}) \qquad Slide_{ELC} = \text{"NG"}$

$Resist_{studE} := F_{Ex} - \left[c\cdot(L_b - x_{tensE}) + \tan(\delta)\cdot(-F_{ysE})\right] \qquad Resist_{studE} = 20416\ \mathrm{lb}$

required shear anchorage force per foot of wall

Sliding Factor of Safety: Seismic Limit State Passive Pressure

$$FS_{sEL} := \frac{c\cdot(L_b - x_{tensEL}) + \tan(\delta)\cdot(-F_{ysEL})}{F_{ExL}} \qquad FS_{sEL} := \mathrm{round}(FS_{sEL}, 1) \qquad FS_{sEL} = 0.2$$

Acceptable 1.1

$Slide_{ELC} := \mathrm{if}(FS_{sEL} \geq 1.0, \text{"OKAY"}, \text{"NG"}) \qquad Slide_{ELC} = \text{"NG"}$

$Resist_{studEL} := F_{ExL} - \left[c\cdot(L_b - x_{tensEL}) + \tan(\delta)\cdot(-F_{ysEL})\right] \qquad Resist_{studEL} = 41838\ \mathrm{lb}$

required shear anchorage force per foot of wall

CONCRETE STRENGTH DESIGN

For limit state passive earth pressure for appraisel-level strength design use no load factors and include strength reduction factors. All other load cases use load factors

Concrete Strength Design Parameters

Concrete strength $f_c := 4000\frac{lb}{in^2}$ Steel strength $f_y := 60000\frac{lb}{in^2}$

Load Factors (Not Used for Passive Limit State) $LF_D := 1.2$ Dead $LF_H := 1.6$ Soil $LF_E := 1.0$ Earthquake

Strength Reduction Factors $\phi_m := 0.9$ Bending $\phi_v := 0.75$ Shear

Minimum Steel ratio

$\rho_{min} := 0.0025$ ACI 14.3.3.b $\rho_{min} = 0.0025$

Element Geometry

Footing thickness	$b_f := d$	Wall thickness	$b_w := t$
Key thickness	$b_k := k$	Unit width	$b := 12in$
Distance to bottom footing steel	$d_{fb} := b_f - 3.5in$	$d_{fb} = 29.5 \cdot in$	
Distance to top footing steel	$d_{ft} := b_f - 2.5in$	$d_{ft} = 30.5 \cdot in$	
Distance to steel in wall	$d_w := b_w - 2.5in$	$d_w = 30.5 \cdot in$	
Distance to steel in key	$d_k := b_k - 2.5in$	$d_k = -2.5 \cdot in$	

Wall Stem - Recompute Lateral Forces for Structural Design of Wall Stem

Horizontal Forces - At-Rest Earth Pressure for Usual Load Case

$P_{D1ows} := \frac{1}{2} \cdot K_o \cdot \gamma \cdot (H_s - H_{wh})^2$ $y_{D1ows} := H_{wh} + \frac{(H_s - H_{wh})}{3}$

$P_{D1ows} = 1687.5\,lb$ $y_{D1ows} = 7.5\,ft$

$P_{D2ows} := K_o \cdot \gamma \cdot (H_s - H_{wh}) \cdot (H_{wh})$ $y_{D2ows} := \frac{H_{wh}}{2}$

$P_{D2ows} = 2250\,lb$ $y_{D2ows} = 2.5\,ft$

$P_{sows} := \frac{1}{2} \cdot K_o \cdot (\gamma_s - \gamma_w) \cdot (H_{wh})^2$ $y_{sows} := \frac{H_{wh}}{3}$

$P_{sows} = 423\,lb$ $y_{sows} = 1.67\,ft$

$P_{wows} := \frac{1}{2} \cdot \gamma_w \cdot (H_{wh})^2$ $y_{wows} := \frac{H_{wh}}{3}$

$P_{wows} = 780\,lb$ $y_{wows} = 1.67\,ft$

Horizontal Forces - Active Earth Pressure for Construction Load Case

$$P_{D1aws} := \frac{1}{2} \cdot K_a \cdot \gamma \cdot \left(H_s - H_{wh}\right)^2 \qquad y_{D1aws} := H_{wh} + \frac{\left(H_s - H_{wh}\right)}{3}$$

$$P_{D1aws} = 1125\,\text{lb} \qquad y_{D1aws} = 7.5\,\text{ft}$$

$$P_{D2aws} := K_a \cdot \gamma \cdot \left(H_s - H_{wh}\right) \cdot \left(H_{wh}\right) \qquad y_{D2aws} := \frac{H_{wh}}{2}$$

$$P_{D2aws} = 1500\,\text{lb} \qquad y_{D2aws} = 2.5\,\text{ft}$$

$$P_{saws} := \frac{1}{2} \cdot K_a \cdot \left(\gamma_s - \gamma_w\right) \cdot \left(H_{wh}\right)^2 \qquad y_{saws} := \frac{H_{wh}}{3}$$

$$P_{saws} = 282\,\text{lb} \qquad y_{saws} = 1.67\,\text{ft}$$

$$P_{waws} := \frac{1}{2} \cdot \gamma_w \cdot \left(H_{wh}\right)^2 \qquad y_{waws} := \frac{H_{wh}}{3}$$

$$P_{waws} = 780\,\text{lb} \qquad y_{waws} = 1.67\,\text{ft}$$

$P_{SURaws} := K_a \cdot \gamma \cdot S \cdot \left(H_s\right)$ *Construction Surcharge* $\qquad y_{SURaws} := \frac{H_s}{2}$

$$P_{SURaws} = 0 \qquad y_{SURaws} = 6.25\,\text{ft}$$

Earthquake Forces - Active Pressures

Horizontal Forces - Active Earth Pressure for Extreme Load Case

$$P_{D1ews} := \frac{1}{2} \cdot K_{AE} \cdot \gamma \cdot \left(H_s - H_{wh}\right)^2 \qquad P_{D1ews} = 4285.01\,\text{lb} \qquad y_{wews} := \frac{H}{2}$$

$$P_{D2ews} := K_{AE} \cdot \gamma \cdot \left(H_s - H_{wh}\right) \cdot \left(H_{wh}\right) \qquad P_{D2ews} = 5713.35\,\text{lb} \qquad y_{sEqews} := \frac{H_s}{2}$$

$$P_{sews} := \frac{1}{2} \cdot K_{AE} \cdot \left(\gamma_s - \gamma_w\right) \cdot \left(H_{wh}\right)^2 \qquad P_{sews} = 1073\,\text{lb}$$

Horizontal Forces - Active Earth Dynamic Components

$$\Delta P_{AE1ews} := \left(P_{D1ews} - P_{D1aws}\right) + \left(P_{D2ews} - P_{D2aws}\right) \qquad y_{AE1ews} := \frac{2 \cdot \left(H_s\right)}{3}$$

$$\Delta P_{AE1ews} = 7373.35\,\text{lb} \qquad y_{AE1ews} = 8.33\,\text{ft}$$

$$\Delta P_{AE2ews} := P_{sews} - P_{saws} \qquad y_{AE2ews} := \frac{2 \cdot \left(H_{wh}\right)}{3}$$

$$\Delta P_{AE2ews} = 791.17\,\text{lb} \qquad y_{AE2ews} = 3.33\,\text{ft}$$

Horizontal Forces - Limit State Earthquake - Passive Earth Pressure

$P_{D1pws} := \frac{1}{2} \cdot K_p \cdot \gamma \cdot (H_s - H_{wh})^2$

$P_{D1pws} = 10125\ \text{lb}$

$y_{D1pws} := H_{wh} + \frac{(H_s - H_{wh})}{3}$

$y_{D1pws} = 7.5\ \text{ft}$

$P_{D2pws} := K_p \cdot \gamma \cdot (H_s - H_{wh}) \cdot (H_{wh})$

$P_{D2pws} = 13500\ \text{lb}$

$y_{D2pws} := \frac{H_{wh}}{2}$

$y_{D2pws} = 2.5\ \text{ft}$

$P_{spws} := \frac{1}{2} \cdot K_p \cdot (\gamma_s - \gamma_w) \cdot (H_{wh})^2$

$P_{spws} = 2535\ \text{lb}$

$y_{spws} := \frac{H_{wh}}{3}$

$y_{spws} = 1.67\ \text{ft}$

$P_{wpws} := \frac{1}{2} \cdot \gamma_w \cdot (H_{wh})^2$

$P_{wpws} = 780\ \text{lb}$

$y_{wpws} := \frac{H_{wh}}{3}$

$y_{wpws} = 1.67\ \text{ft}$

$P_{SURpws} := K_p \cdot \gamma \cdot S \cdot (H_s)$ *Construction Surcharge*

$P_{SURpws} = 0$

$y_{SURpws} := \frac{H_s}{2}$

$y_{SURpws} = 6.25\ \text{ft}$

Summation of Forces

ΣF_x Horizontal Force

Usual Load Case (At-Rest Pressures)

$F_{xows} := P_{D1ows} + P_{D2ows} + P_{sows} + P_{wows}$ $\qquad F_{xows} = 5140\ \text{lb}$

Construction Load Case (Active Pressures)

$F_{xaws} := P_{D1aws} + P_{D2aws} + P_{saws} + P_{waws} + P_{SURaws}$ $\qquad F_{xaws} = 3687\ \text{lb}$

Seismic Load Case (Active Pressures)

$F_{Ex1} := \Delta P_{AE1ews} + \Delta P_{AE2ews} + W_{we} + W_{sEq} + P_{Re} + P_{He}$ $\qquad F_{Ex1} = 14171\ \text{lb}$

$F_{Ex2} := P_{D1aws} + P_{D2aws} + P_{saws} + P_{waws}$ $\qquad F_{Ex2} = 3687\ \text{lb}$

$F_{Ex} := F_{Ex1} + F_{Ex2}$ $\qquad F_{Ex} = 17857.44\ \text{lb}$

Seismic Limit State Case (Passive Pressures)

$F_{ExLS1} := W_{we} + W_{sEq} + P_{Re} + P_{He}$ $\qquad F_{ExLS1} = 6006\ \text{lb}$

$F_{ExLS2} := P_{D1pws} + P_{D2pws} + P_{spws} + P_{wpws}$ $\qquad F_{ExLS2} = 26940\ \text{lb}$

$F_{ExLS} := F_{ExLS1} + F_{ExLS2}$ $\qquad F_{ExLS} = 32946.25\ \text{lb}$

Factored Design Shear - ACI 318 & ASCE 7

$V_{uLLo} := LF_H \cdot F_{xows}$ (Static - At-Rest) $V_{uLLo} = 8224\ lb$

$V_{uLLa} := LF_H \cdot F_{xaws}$ (Static - Active) $V_{uLLa} = 5899\ lb$

$V_{uE} := LF_E \cdot F_{Ex1} + LF_H \cdot F_{Ex2}$ (Seismic - Active) $V_{uE} = 20069\ lb$

$V_{uEL} := F_{ExLS1} + F_{ExLS2}$ (Seismic Limit State- Passive) $V_{uEL} = 32946\ lb$

$V_{uMax} := \max(V_{uLLo}, V_{uLLa}, V_{uE}, V_{uEL})$ $V_{uMax} = 32946\ lb$

Wall Stem Shear Check

$V_c := 2 \cdot \sqrt{f_c \cdot \frac{lb}{in^2}} \cdot b \cdot d_w$ $V_c = 46296\ lb$

$\phi V_n := \phi_v \cdot V_c$ $\phi V_n = 34722\ lb$

$Shear_{stem} := \text{if}(V_{uMax} \le \phi V_n, \text{"OKAY"}, \text{"NG"})$ $Shear_{stem} = \text{"OKAY"}$

Summation of Moments

Max Flexural Moment

Usual Load Case (At-Rest Pressures)

$M_{ows} := P_{D1ows} \cdot y_{D1ows} + P_{D2ows} \cdot y_{D2ows} + P_{sows} \cdot y_{sows} + P_{wows} \cdot y_{wows}$

$M_{ows} = 20285\ lb \cdot ft$

Max Flexural Moment

Construction Load Case (Active Pressures)

$M_{aws} := P_{D1aws} \cdot y_{D1aws} + P_{D2aws} \cdot y_{D2aws} + P_{saws} \cdot y_{saws} + P_{waws} \cdot y_{waws} + P_{SURaws} \cdot y_{SURaws}$

$M_{aws} = 13957\ lb \cdot ft$

Seismic Load Case (Active Pressures)

$M_{Ews1} := P_{D1aws} \cdot y_{D1aws} + P_{D2aws} \cdot y_{D2aws} + P_{saws} \cdot y_{saws} + P_{waws} \cdot y_{waws}$

$M_{Ews2} := \Delta P_{AE1ews} \cdot y_{AE2ews} + \Delta P_{AE2ews} \cdot y_{AE2ews} + P_{Re} \cdot y_{Re} + P_{He} \cdot y_{He} + W_{we} \cdot y_{wews} + W_{sEq} \cdot y_{sEqews}$

$M_{Ews1} = 13956.94\ lb \cdot ft$ $M_{Ews2} = 63479.15\ lb \cdot ft$ $M_{Ews} := M_{Ews1} + M_{Ews2} = 77436\ lb \cdot ft$

Seismic Limit State Case (Passive Pressures)

$M_{Ews1LS} := P_{D1pws} \cdot y_{D1pws} + P_{D2pws} \cdot y_{D2pws} + P_{spws} \cdot y_{spws} + P_{wpws} \cdot y_{wpws}$

$M_{Ews2LS} := P_{Re} \cdot y_{Re} + P_{He} \cdot y_{He} + W_{we} \cdot y_{wews} + W_{sEq} \cdot y_{sEqews}$

$M_{Ews1LS} = 115212\ lb \cdot ft$ $M_{Ews2LS} = 36264\ lb \cdot ft$ $M_{EwsLS} := M_{Ews1LS} + M_{Ews2LS} = 151477\ lb \cdot ft$

Factored Design Moment - ACI 318 & ASCE 7

$M_{uLLo} := LF_H \cdot M_{ows}$ (Static - At-Rest) $M_{uLLo} = 32457\ lb \cdot ft$

$M_{uLLa} := LF_H \cdot M_{aws}$ (Static - Active) $M_{uLLa} = 22331\ lb \cdot ft$

$M_{uE} := LF_H \cdot M_{Ews1} + LF_E \cdot M_{Ews2}$ (Seismic - Active) $M_{uE} = 85810\ lb \cdot ft$

$M_{uELS} := M_{EwsLS}$ (Seismic Limit State- Passive) $M_{uELS} = 151477\ lb \cdot ft$

$M_{uMaxws} := \max(M_{uLLo}, M_{uLLa}, M_{uE}, M_{uELS})$ $M_{uMaxws} = 151477\ lb \cdot ft$

$$k_u := 1 - \sqrt{1 - \frac{M_{uMaxws}}{0.425 \cdot \phi_m \cdot f_c \cdot b \cdot d_w^{\,2}}} \qquad k_u = 0.05$$

$$A_{sws} := \frac{0.85 \cdot f_c \cdot k_u \cdot b \cdot d_w}{f_y} \qquad A_{sws} = 1.13 \cdot in^2$$

$$A_{smin} := \rho_{min} \cdot b \cdot d_w \qquad A_{smin} = 0.91 \cdot in^2$$

#10 @ 12" Each Face Vertical Bars A_s = 1.27 in.^{2}

Conservatively use the same reinforcement on inside face of wall.

Temperature and Shrinkage Steel - USACE EM 1110 -2-2104

$$A_{ts} := \frac{.0028 \cdot b \cdot b_w}{2} \qquad A_{ts} = 0.55 \cdot in^2$$

#7 @ 12" Each Face A_s = 0.60 in.^{2}

Footing Design

For strength design of the footing, conservatively design the bottom reinforcement for the toe base pressure distribution from the edge of the toe to the centerline of the wall stem. Conservatively design the top reinforcement based on the weight of the fill above the heel.

Toe Shear Load

Static - At-Rest

$\sigma_{toefaceo} = -2210\,lb \cdot ft^{-1}$ $\quad \sigma_{mido} = -1713\,lb \cdot ft^{-1}$

$\sigma_{do} := \sigma_{mido} + bp_{oslope} \cdot \left(\frac{t}{2} + d_{fb}\right)$ $\quad \sigma_{do} = -2148.86\,lb \cdot ft^{-1}$

Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avgo} := \frac{\sigma_{toefaceo} + \sigma_{do}}{2}$ $\quad \sigma_{avgo} = -2179.68\,lb \cdot ft^{-1}$

$V_{toeo} := -\sigma_{avgo} \cdot (L_t - d_{fb})$ $\quad V_{toeo} = 1180.66\,lb$

Static - Active

$\sigma_{toefacea} = -1844\,lb \cdot ft^{-1}$ $\quad \sigma_{mida} = -1584\,lb \cdot ft^{-1}$

$\sigma_{da} := \sigma_{mida} + bp_{aslope} \cdot \left(\frac{t}{2} + d_{fb}\right)$ $\quad \sigma_{da} = -1811.41\,lb \cdot ft^{-1}$

Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avga} := \frac{\sigma_{toefacea} + \sigma_{da}}{2}$ $\quad \sigma_{avga} = -1827.5\,lb \cdot ft^{-1}$

$V_{toea} := -\sigma_{avga} \cdot (L_t - d_{fb})$ $\quad V_{toea} = 989.9\,lb$

Seismic - Active

$\sigma_{toefaceE} = -7323\,lb \cdot ft^{-1}$ $\quad \sigma_{midE} = -3512\,lb \cdot ft^{-1}$

$\sigma_{dE} := \sigma_{midE} + bp_{Eslope} \cdot \left(\frac{t}{2} + d_{fb}\right)$ $\quad \sigma_{dE} = -6851.31\,lb \cdot ft^{-1}$

Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avgE} := \frac{\sigma_{toefaceE} + \sigma_{dE}}{2}$ $\quad \sigma_{avgE} = -7087.27\,lb \cdot ft^{-1}$

$V_{toeE} := -\sigma_{avgE} \cdot (L_t - d_{fb})$ $\quad V_{toeE} = 3838.94\,lb$

Seismic Limit State - Passive

$\sigma_{toefaceEL} = -9627\,lb \cdot ft^{-1}$ $\sigma_{midEL} = -4322\,lb \cdot ft^{-1}$

$\sigma_{dEL} := \sigma_{midEL} + bp_{ELslope} \cdot \left(\frac{t}{2} + d_{fb}\right)$ $\sigma_{dEL} = -8969.92\,lb \cdot ft^{-1}$ Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avgEL} := \frac{\sigma_{toefaceEL} + \sigma_{dEL}}{2}$ $\sigma_{avgEL} = -9298.3\,lb \cdot ft^{-1}$

$V_{toeEL} := -\sigma_{avgEL} \cdot (L_t - d_{fb})$ $V_{toeEL} = 5036.58\,lb$

Factored Design Shear - ACI 318 & ASCE 7

$V_{utoeo} := LF_H \cdot V_{toeo}$ (Static - At-Rest) $V_{utoeo} = 1889\,lb$

$V_{utoea} := LF_H \cdot V_{toea}$ (Static - Active) $V_{utoea} = 1584\,lb$

$V_{uE} := LF_H \cdot V_{toea} + LF_E \cdot (V_{toeE} - V_{toea})$ (Seismic - Active) $V_{uE} = 4433\,lb$

$V_{uEL} := V_{toeEL}$ (Seismic Limit State- Passive) $V_{uEL} = 5037\,lb$

$V_{uMax} := \max(V_{utoeo}, V_{utoea}, V_{uE}, V_{uEL})$ $V_{uMax} = 5037\,lb$

Footing Toe Shear Check

$V_c := 2 \cdot \sqrt{f_c \cdot \frac{lb}{in^2}} \cdot b \cdot d_{fb}$ $V_c = 44778\,lb$

$\phi V_n := \phi_v \cdot V_c$ $\phi V_n = 33583\,lb$

$Shear_{foot} := if(V_{uMax} \le \phi V_n, "OKAY", "NG")$ $Shear_{foot} = "OKAY"$

Check the heel base pressure using the same approach that was used for the toe.

Heel Shear Load Due To Resultant Base Pressure

Static - At-Rest

$\sigma_{heelfaceo} = -674\,lb \cdot ft^{-1}$ $\sigma_{mido} = -1713\,lb \cdot ft^{-1}$

$\sigma_{dheelo} := \sigma_{mido} - bp_{oslope} \cdot \left(\frac{t}{2} + d_{fb}\right)$ $\sigma_{dheelo} = -1276.53\,lb \cdot ft^{-1}$ Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avgheelo} := \frac{\sigma_{heelfaceo} + \sigma_{dheelo}}{2}$ $\sigma_{avgheelo} = -975.48\,lb \cdot ft^{-1}$

$V_{heelo} := -\sigma_{avgheelo} \cdot (L_h - d_{fb})$ $V_{heelo} = 5162\,lb$

Static - Active

$\sigma_{heelfacea} = -1041\,lb\cdot ft^{-1}$ $\sigma_{mida} = -1584\,lb\cdot ft^{-1}$

$\sigma_{dheela} := \sigma_{mida} - bp_{aslope}\cdot\left(\frac{t}{2} + d_{fb}\right)$ $\sigma_{dheela} = -1355.79\,lb\cdot ft^{-1}$ Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avgheela} := \frac{\sigma_{heelfacea} + \sigma_{dheela}}{2}$ $\sigma_{avgheela} = -1198.56\,lb\cdot ft^{-1}$

$V_{heela} := -\sigma_{avgheela}\cdot\left(L_h - d_{fb}\right)$ $V_{heela} = 6342\,lb$

Seismic - Active

$\sigma_{heelfaceE} = 4438\,lb\cdot ft^{-1}$ $\sigma_{midE} = -3512\,lb\cdot ft^{-1}$

$\sigma_{dheelE} := \sigma_{midE} - bp_{Eslope}\cdot\left(\frac{t}{2} + d_{fb}\right)$ $\sigma_{dheelE} = -171.92\,lb\cdot ft^{-1}$ Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avgheelE} := \frac{\sigma_{heelfaceE} + \sigma_{dheelE}}{2}$ $\sigma_{avgheelE} = 2133.19\,lb\cdot ft^{-1}$

$\sigma_{avgheelE} := if\left(\sigma_{avgheelE} \geq 0, 0, \sigma_{avgheelE}\right)$

$V_{heelE} := -\sigma_{avgheelE}\cdot\left(L_h - d_{fb}\right)$ $V_{heelE} = 0\,lb$

Seismic Limit State - Passive

$\sigma_{heelfaceEL} = 6742\,lb\cdot ft^{-1}$ $\sigma_{midEL} = -4322\,lb\cdot ft^{-1}$

$\sigma_{dheelEL} := \sigma_{midEL} - bp_{ELslope}\cdot\left(\frac{t}{2} + d_{fb}\right)$ $\sigma_{dheelEL} = 325.74\,lb\cdot ft^{-1}$ Compute the shear a distance "d"away from the face of the wall.

$\sigma_{avgheelEL} := \frac{\sigma_{heelfaceEL} + \sigma_{dheelEL}}{2}$ $\sigma_{avgheelEL} = 3533.75\,lb\cdot ft^{-1}$

$\sigma_{avgheelEL} := if\left(\sigma_{avgheelEL} \geq 0, 0, \sigma_{avgheelEL}\right)$

$V_{heelEL} := -\sigma_{avgheelEL}\cdot\left(L_h - d_{fb}\right)$ $V_{heelEL} = 0\,lb$ Full tension - Therefore Not Applicable.

Factored Design Shear - ACI 318 & ASCE 7

$V_{uheelo} := LF_H \cdot V_{heelo}$ (Static - At-Rest) $V_{uheelo} = 8259\ lb$

$V_{uheela} := LF_H \cdot V_{heela}$ (Static - Active) $V_{uheela} = 10148\ lb$

$V_{uheelE} := LF_H \cdot V_{heela} + LF_E \cdot (V_{heelE} - V_{heela})$ (Seismic - Active) $V_{uheelE} = 3805\ lb$

$V_{uheelEL} := V_{heelEL}$ (Seismic Limit State- Passive) $V_{uheelEL} = 0\ lb$

$V_{uMax} := \max(V_{uheelo}, V_{uheela}, V_{uE}, V_{uEL})$ $V_{uMax} = 10148\ lb$

Footing Heel Shear Check

$V_c := 2 \cdot \sqrt{f_c \cdot \frac{lb}{in^2}} \cdot b \cdot d_{fb}$ $V_c = 44778\ lb$

$\phi V_n := \phi_v \cdot V_c$ $\phi V_n = 33583\ lb$

$Shear_{foot} := if(V_{uMax} \le \phi V_n, "OKAY", "NG")$ $Shear_{foot} = "OKAY"$

Shear Load Due To Dead Load Above Heel

$W_{dead} := F_y - U_1 - U_2$ $W_{dead} = -22737\ lb$ (Total dead load above heel)

$V_{uDead} := -LF_D \cdot W_{dead}$ $V_{uDead} = 27285\ lb$

Footing Heel Shear Check

$V_c := 2 \cdot \sqrt{f_c \cdot \frac{lb}{in^2}} \cdot b \cdot d_{ft}$ $V_c = 46296\ lb$

$\phi V_n := \phi_v \cdot V_c$ $\phi V_n = 34722\ lb$

$Shear_{foot} := if(V_{uDead} \le \phi V_n, "OKAY", "NG")$ $Shear_{foot} = "OKAY"$

Footing Toe - Bottom Reinforcement

Static - At-Rest

$$L_{toeo} := L_t + \frac{t}{2} \qquad L_{toeo} = 4.38\,\text{ft}$$

$$\sigma_{mido} = -1712.69\,\text{lb}\cdot\text{ft}^{-1}$$

$$\sigma_{toefaceo} = -2210.49\,\text{lb}\cdot\text{ft}^{-1}$$

$$x_{toeo} := \frac{1}{3}\cdot\left(\frac{\sigma_{mido} + 2\cdot\sigma_{toefaceo}}{\sigma_{mido} + \sigma_{toefaceo}}\right)\cdot L_{toeo} \qquad x_{toeo} = 2.28\,\text{ft}$$

$$M_{toeo} := -\left(\frac{\sigma_{toefaceo} + \sigma_{mido}}{2}\right)\cdot L_{toeo}\cdot x_{toeo} \qquad M_{toeo} = 19567\,\text{lb}\cdot\text{ft}$$

Static - Active

$$L_{toea} := L_t + \frac{t}{2} \qquad L_{toea} = 4.38\,\text{ft}$$

$$\sigma_{mida} = -1583.6\,\text{lb}\cdot\text{ft}^{-1}$$

$$\sigma_{toefacea} = -1843.6\,\text{lb}\cdot\text{ft}^{-1}$$

$$x_{toea} := \frac{1}{3}\cdot\left(\frac{\sigma_{mida} + 2\cdot\sigma_{toefacea}}{\sigma_{mida} + \sigma_{toefacea}}\right)\cdot L_{toea} \qquad x_{toea} = 2.24\,\text{ft}$$

$$M_{toea} := -\left(\frac{\sigma_{toefacea} + \sigma_{mida}}{2}\right)\cdot L_{toea}\cdot x_{toea} \qquad M_{toea} = 16814\,\text{lb}\cdot\text{ft}$$

Seismic - Active

$$L_{toeE} := L_t + \frac{t}{2} \qquad L_{toeE} = 4.38\,\text{ft}$$

$$\sigma_{midE} = -3511.62\,\text{lb}\cdot\text{ft}^{-1}$$

$$\sigma_{toefaceE} = -7323.22\,\text{lb}\cdot\text{ft}^{-1}$$

$$x_{toeE} := \frac{1}{3}\cdot\left(\frac{\sigma_{midE} + 2\cdot\sigma_{toefaceE}}{\sigma_{midE} + \sigma_{toefaceE}}\right)\cdot L_{toeE} \qquad x_{toeE} = 2.44\,\text{ft}$$

$$M_{toeE} := -\left(\frac{\sigma_{toefaceE} + \sigma_{midE}}{2}\right)\cdot L_{toeE}\cdot x_{toeE} \qquad M_{toeE} = 57926\,\text{lb}\cdot\text{ft}$$

Seismic Limit State- Passive

$$L_{toeEL} := L_t + \frac{t}{2} \qquad L_{toeEL} = 4.38\,\text{ft}$$

$$\sigma_{midEL} = -4322.09\,\text{lb}\cdot\text{ft}^{-1}$$

$$\sigma_{toefaceEL} = -9626.68\,\text{lb}\cdot\text{ft}^{-1}$$

$$x_{toeEL} := \frac{1}{3}\cdot\left(\frac{\sigma_{midEL} + 2\cdot\sigma_{toefaceEL}}{\sigma_{midEL} + \sigma_{toefaceEL}}\right)\cdot L_{toeEL} \qquad x_{toeEL} = 2.46\,\text{ft}$$

$$M_{toeEL} := -\left(\frac{\sigma_{toefaceEL} + \sigma_{midEL}}{2}\right)\cdot L_{toeEL}\cdot x_{toeEL} \qquad M_{toeEL} = 75208\,\text{lb}\cdot\text{ft}$$

Factored Design Moment - ACI 318 & ASCE 7

$M_{utoeo} := LF_H \cdot M_{toeo}$ (Static - At-Rest) $M_{utoeo} = 31307\ lb \cdot ft$

$M_{utoea} := LF_H \cdot M_{toea}$ (Static - Active) $M_{utoea} = 26903\ lb \cdot ft$

$M_{utoeE} := LF_H \cdot M_{toea} + LF_E \cdot (M_{toeE} - M_{toea})$ (Seismic - Active) $M_{utoeE} = 68015\ lb \cdot ft$

$M_{utoeEL} := M_{toeEL}$ (Seismic Limit State- Passive) $M_{utoeEL} = 75208\ lb \cdot ft$

$M_{uMax} := \max(M_{utoeo}, M_{utoea}, M_{utoeE}, M_{utoeEL})$ $M_{uMax} = 75208\ lb \cdot ft$

$$k_u := 1 - \sqrt{1 - \frac{M_{uMax}}{0.425 \cdot \phi_m \cdot f_c \cdot b \cdot d_{fb}^2}} \qquad k_u = 0.03$$

$$A_{stoe} := \frac{0.85 \cdot f_c \cdot k_u \cdot b \cdot d_{fb}}{f_y} \qquad A_{stoe} = 0.57 \cdot in^2$$

Footing Heel - Bottom Reinforcement

Static - At-Rest

$\sigma_{mido} = -1712.69\ lb \cdot ft^{-1}$

$\sigma_{heelfaceo} = -674.43\ lb \cdot ft^{-1}$

$$L_{heelo} := L_h + \frac{t}{2} - x_{tenso} \qquad L_{heelo} = 9.13\ ft$$

$$x_{heelo} := \frac{1}{3} \cdot \left(\frac{\sigma_{mido} + 2 \cdot \sigma_{heelfaceo}}{\sigma_{mido} + \sigma_{heelfaceo}} \right) \cdot L_{heelo} \qquad x_{heelo} = 3.9\ ft$$

$$M_{heelo} := -\left(\frac{\sigma_{heelfaceo} + \sigma_{mido}}{2} \right) \cdot L_{heelo} \cdot x_{heelo} \qquad M_{heelo} = 42487\ lb \cdot ft$$

Static - Active

$\sigma_{mida} = -1583.6\ lb \cdot ft^{-1}$

$\sigma_{heelfacea} = -1041.32\ lb \cdot ft^{-1}$

$$L_{heela} := L_h + \frac{t}{2} - x_{tensa} \qquad L_{heela} = 9.13\ ft$$

$$x_{heela} := \frac{1}{3} \cdot \left(\frac{\sigma_{mida} + 2 \cdot \sigma_{heelfacea}}{\sigma_{mida} + \sigma_{heelfacea}} \right) \cdot L_{heela} \qquad x_{heela} = 4.25\ ft$$

$$M_{heela} := -\left(\frac{\sigma_{heelfacea} + \sigma_{mida}}{2} \right) \cdot L_{heela} \cdot x_{heela} \qquad M_{heela} = 50879\ lb \cdot ft$$

Seismic - Active

$$L_{heelE} := L_h + \frac{t}{2} - x_{tensE} \quad L_{heelE} = 16.39\,ft$$

$$\sigma_{midE} = -3511.62\,lb \cdot ft^{-1}$$

$$\sigma_{heelfaceE} = 4438.31\,lb \cdot ft^{-1}$$

$$x_{heelE} := \frac{1}{3} \cdot \left(\frac{\sigma_{midE} + 2 \cdot \sigma_{heelfaceE}}{\sigma_{midE} + \sigma_{heelfaceE}} \right) \cdot L_{heelE} \qquad x_{heelE} = 31.64\,ft$$

$$M_{heelE} := -\left(\frac{\sigma_{heelfaceE} + \sigma_{midE}}{2} \right) \cdot L_{heelE} \cdot x_{heelE} \qquad M_{heelE} = -240322\,lb \cdot ft$$

Seismic Limit State- Passive

$$L_{heelEL} := L_h + \frac{t}{2} - x_{tensEL} \quad L_{heelEL} = 27.17\,ft$$

$$\sigma_{midEL} = -4322.09\,lb \cdot ft^{-1}$$

$$\sigma_{heelfaceEL} = 6741.76\,lb \cdot ft^{-1}$$

$$x_{heelEL} := \frac{1}{3} \cdot \left(\frac{\sigma_{midEL} + 2 \cdot \sigma_{heelfaceEL}}{\sigma_{midEL} + \sigma_{heelfaceEL}} \right) \cdot L_{heelEL} \quad x_{heelEL} = 34.29\,ft$$

$$M_{heelEL} := -\left(\frac{\sigma_{heelfaceEL} + \sigma_{midEL}}{2} \right) \cdot L_{heelEL} \cdot x_{heelEL} \qquad M_{heelEL} = -1 \times 10^6\,lb \cdot ft$$

<u>Factored Design Moment - ACI 318 & ASCE 7</u>

$M_{uheelo} := LF_H \cdot M_{heelo}$ (Static - At-Rest) $\qquad M_{uheelo} = 67979\,lb \cdot ft$

$M_{uheela} := LF_H \cdot M_{heela}$ (Static - Active) $\qquad M_{uheela} = 81406\,lb \cdot ft$

$M_{uheelE} := LF_H \cdot M_{heela} + LF_E \cdot (M_{heelE} - M_{heela})$ (Seismic - Active) $\qquad M_{uheelE} = -209794\,lb \cdot ft$

$M_{uheelEL} := M_{heelEL}$ (Seismic Limit State- Passive) $\qquad M_{uheelEL} = -1127433\,lb \cdot ft$

$$M_{uMax} := \max(M_{uheelo}, M_{uheela}, M_{uheelE}, M_{uheelEL}) \qquad M_{uMax} = 81406\,lb \cdot ft$$

$$k_u := 1 - \sqrt{1 - \frac{M_{uMax}}{0.425 \cdot \phi_m \cdot f_c \cdot b \cdot d_{fb}^2}} \qquad k_u = 0.03$$

$$A_{sheel} := \frac{0.85 \cdot f_c \cdot k_u \cdot b \cdot d_{fb}}{f_y} \qquad A_{sheel} = 0.62 \cdot in^2$$

$A_{smin} := \rho_{min} \cdot b \cdot d_{fb} \qquad A_{smin} = 0.89 \cdot in^2$ (Minimum for flexure)

$$A_{sBot} := \max(A_{stoe}, A_{sheel}, A_{smin}) \qquad A_{sBot} = 0.89 \cdot in^2$$

<u>#9 @ 12" Bottom Face A_s = 1.00 in.^{2}</u>

Moment Load Due To Dead Load Above Heel

$W_{dead} := F_y - U_l - U_2$ $\quad W_{dead} = -22737\,lb$ (Total dead load above heel)

$M_{uDead} := -LF_D \cdot W_{dead} \cdot \left(\frac{L_h}{2} + \frac{t}{2}\right)$ $\quad M_{uDead} = 143246\,lb \cdot ft$

$M_{uMax} := M_{uDead}$ $\quad M_{uMax} = 143246\,lb \cdot ft$

$k_u := 1 - \sqrt{1 - \frac{M_{uMax}}{0.425 \cdot \phi_m \cdot f_c \cdot b \cdot d_{ft}^2}}$ $\quad k_u = 0.05$

$A_{stop} := \frac{0.85 \cdot f_c \cdot k_u \cdot b \cdot d_{ft}}{f_y}$ $\quad A_{stop} = 1.07 \cdot in^2$

$A_{smin} := \rho_{min} \cdot b \cdot d_{ft}$ $\quad A_{smin} = 0.91 \cdot in^2$ (Minimum for flexure)

$A_{sTop} := \max(A_{stop}, A_{smin})$ $\quad A_{sTop} = 1.07 \cdot in^2$

#10 @ 12" Top Face A_s = 1.27 in.^{2}

Temperature and Shrinkage Steel

$A_{ts} := \frac{.0028 \cdot b \cdot b_f}{2}$ $\quad A_{ts} = 0.55 \cdot in^2$

#7 @ 12" Each Face A_s = 0.60 in.^{2}

Summary of Thickness and Reinforcement Requirements

Usual, Construction and Extreme Load Combinations

	Primary Reinforcement	Temperature and Shrink Reinforcement	Thickness
Wall Stem - Outside	#10 @ 12" Vertical	#7 @ 12" Horizontal	2'-9"
Wall Stem - Inside	#10 @ 12" Vertical	#7 @ 12" Horizontal	2'-9"
Footing - Bottom	#9 @ 12" Horizontal	#7 @ 12" Horizontal	2'-9"
Footing - Top	#10 @ 12" Horizontal	#7 @ 12" Horizontal	2'-9"

Determine Welded Stud Requirements for Shear During Shaking

$Resist_{studE} = 20416\ lb$ Required shear anchorage - seismic active pressure

$Resist_{UstudE} := LF_H \cdot F_{xa} + LF_E \cdot (Resist_{studE} - F_{xa})$ Factored shear anchorage - seismic active pressure

$Resist_{UstudE} = 24016\ lb$

$Resist_{studEL} = 41838\ lb$ Required shear anchorage - seismic limit state - passive pressure

$Resist_{UstudEL} := Resist_{studEL}$ Factored shear anchorage - seismic limit state - passive pressure

$Resist_{UstudEL} = 41838\ lb$

$Resist_{UMax} := \max(Resist_{UstudE}, Resist_{UstudEL})$

$Resist_{UMax} = 41837.68\ lb$ Maximum Factored shear anchorage

$Stud_{Vcap} := 21137 \cdot lb$ Shear capacity of 7/8" diameter S3L Nelson Stud - see attached

$No_{Stud} := \dfrac{Resist_{UMax}}{Stud_{Vcap}}$ Number of studs required per foot length of wall

$No_{Stud} = 1.98$ Say 2 studs required per foot length of wall for shear

Determine Welded Stud Requirements for Tension During Shaking

Seismic - Active

$\sigma_{heelfaceE} = 4438\ lb \cdot ft^{-1}$ $\sigma_{toefaceE} = -7323\ lb \cdot ft^{-1}$

$\sigma_{avgheelE} := \dfrac{\sigma_{heelfaceE} + \sigma_{toefaceE}}{2}$ $\sigma_{avgheelE} = -1442.46\ lb \cdot ft^{-1}$

$\sigma_{avgheelE} := if(\sigma_{avgheelE} \geq 0, 0, \sigma_{avgheelE})$

$P_{compE} := -\sigma_{avgheelE} \cdot (L_b - x_{tensE})$ $P_{compE} = 29959\ lb$

$P_{tensE} := P_{compE} - F_y$ $P_{tensE} = 49432\ lb$

Seismic Limit State - Passive

$\sigma_{heelfaceEL} = 6742\,lb\cdot ft^{-1}$ $\sigma_{toefaceEL} = -9627\,lb\cdot ft^{-1}$

$\sigma_{avgheelEL} := \dfrac{\sigma_{heelfaceEL} + \sigma_{toefaceEL}}{2}$ $\sigma_{avgheelEL} = -1442.46\,lb\cdot ft^{-1}$

$\sigma_{avgheelEL} := if(\sigma_{avgheelEL} \geq 0, 0, \sigma_{avgheelEL})$

$P_{compEL} := -\sigma_{avgheelEL}\cdot(L_b - x_{tensEL})$ $P_{compEL} = 45507\,lb$

$P_{tensEL} := P_{compEL} - F_y$ $P_{tensEL} = 64980\,lb$

$P_{UMax} := max(P_{tensE}, P_{tensEL})$

$P_{UMax} = 64980\,lb$

$Stud_{Pcap} := 0.6\cdot 23100\cdot lb$ Tensile capacity of 7/8" diameter S3L Nelson Stud - 0.6Fy - see attached

$Stud_{Pcap} = 13860\,lb$

$No_{PStud} := \dfrac{P_{UMax}}{Stud_{Pcap}}$ Number of studs required per foot length of wall

$No_{PStud} = 4.69$ Say 5 studs required per foot length of wall for tension

Re-check for net overturninig moment

$M_{OTMax} := M_{Er} + M_{Ep}$ $M_{OTMax} = 117152\,lb\cdot ft$

$k_{uMax} := 1 - \sqrt{1 - \dfrac{M_{OTMax}}{0.425\cdot\phi_m\cdot f_c\cdot b\cdot L_b^2}}$ $k_{uMax} = 0.0015$

$A_{stud} := \dfrac{0.85\cdot f_c\cdot k_u\cdot b\cdot L_b}{(80000)\cdot\dfrac{lb}{in^2}}$ $A_{stud} = 4.27\cdot in^2$ $N_{StudFlex} := \dfrac{A_{stud}}{\left[\dfrac{\pi\cdot\left(\dfrac{7}{8}\cdot in\right)^2}{4}\right]}$

$N_{StudFlex} = 7.1$

$No_{StudTot1} := No_{Stud} + No_{PStud}$ $No_{StudTot1} = 6.67$

$No_{StudTot2} := No_{Stud} + N_{StudFlex}$ $No_{StudTot2} = 9.08$

$No_{StudTot} := \max(No_{StudTot1}, No_{StudTot2})$ $\qquad$ $No_{StudTot} = 9.08$ Conservative value

Say 8 studs required per foot length of wall - Assume 2' x 2' Footing Grid Pattern to match table tie downs

Compute Wall Stiffness

$$E_c := 57000 \cdot \sqrt{f_c \cdot \frac{in^2}{lb}} \cdot \frac{lb}{in^2} \qquad E_c = 3604996.53 \cdot \frac{lb}{in^2}$$

$$I := \frac{1}{12} \cdot b \cdot t^3$$

$$I = 35937 \cdot in^4$$

$$K := \frac{15 \cdot E_c \cdot I}{H^3}$$ Cantilver beam with triangular shaped distributed load - Ref. AISC, Table 3-23, Case 18

$$K = 575790 \cdot \frac{lb}{in}$$

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COEFFICIENT OF FRICTION

- **The following friction coefficients shall be considered in calculating the sliding friction forces :**

Concrete to Soil / Rock	**0.30**
Concrete to Steel	**0.45**
Steel to Steel	**0.30**
Steel to Teflon Plate	**0.10**
Brick Masonry on moist clay	**0.33**
Brick Masonry on dry clay	**0.50**
Brick Masonry on sand	**0.40**
Brick Masonry on gravel	**0.60**
Brick Masonry to Brick	**0.70**
Brick Masonry on rock	**0.75**
Granite on Granite	**0.60**
Limestone on Limestone	**0.75**
Cement Blocks on Cement Blocks	**0.65**
Cement concrete on dry clay	**0.40**
Cement concrete on wet clay	**0.20**
Cement concrete on wet sand	**0.40**
Cement concrete on dry sand	**0.50 - 0.60**
Cement concrete on dry gravel	**0.50 - 0.60**
Cement concrete on dry rock	**0.60 - 0.70**
Cement concrete on wet rock	**0.50**
Brick on Brick	**0.65**
Wood on Wood	**0.48**
Note: Friction is more on dry surfaces of the same material compared to wet surface.	

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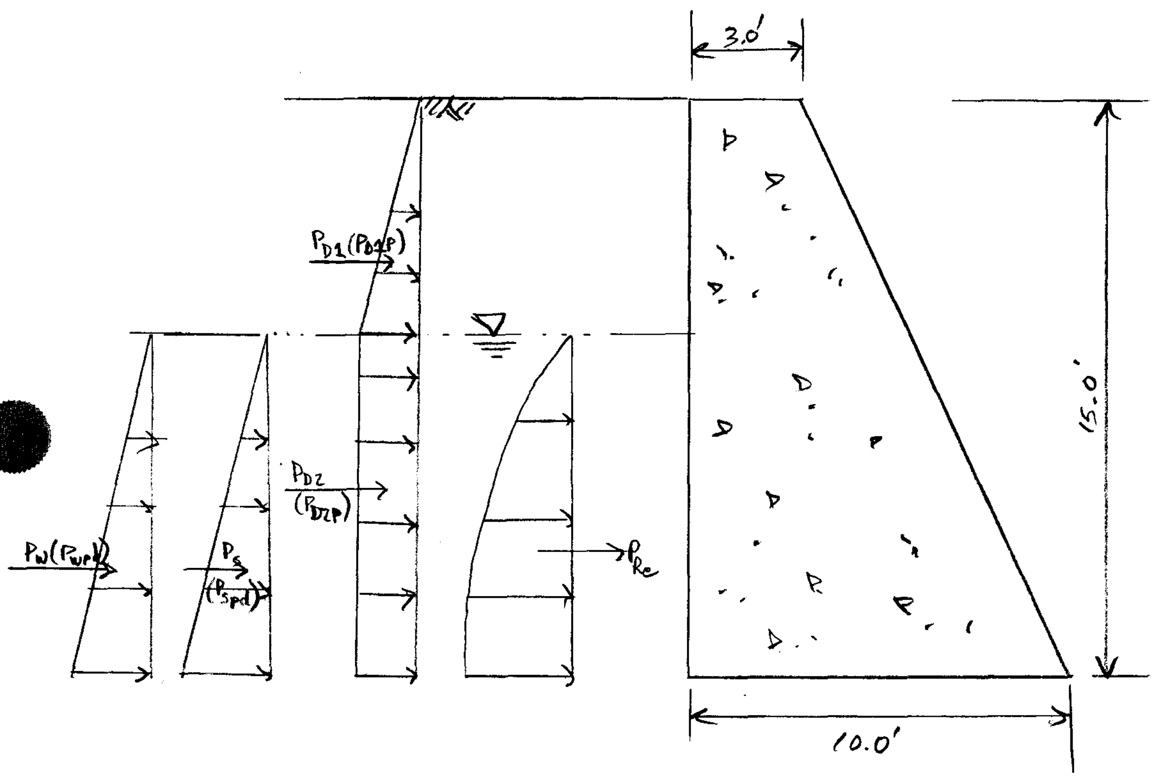
7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
SJD	6/3/2009	SSINT - Phase 1	
CHKD BY	DATE	FEATURE	
DETAILS Gravity Wall Design - Full-Scale Shake Table Test			

Gravity Wall Loading Diagram



References:

International Code Council, *International Building Code,* 2003
USACOE, *EM 1110-2-2502 - Retaining and Flood Walls*, September 29, 1989
Kramer, *Geotechnical Earthquake Engineering*, Chapter 11 - Seismic Design of Retaining Walls
NRCS, *TR-74 - Lateral Earth Pressures*, July 1989
ASCE, *SEI/ASCE 7-02 - Minimum Design Loads for Buildings and Other Structures,* 2002
USBR, *Design Criteria for Concrete Retaining Walls*, August 1971
NRCS, *TR-67 - Reinforced Concrete Strength Design*, August 1980
USGS, Probabilistic Hazard Program, Version 3.10

Gravity Wall Dimensions

Wall height $H := 15.0ft$

Soil height (heel) $H_s := 15.0ft$

Soil height (toe) $H_{st} := 0.0ft$

Water height (heel) $H_{wh} := 7.75ft$

Water height (toe) $H_{wt} := 0.0ft$

Surcharge (construction) $S := 0ft$

Avg. Wall thickness $t := 6.5ft$

Footing thickness $d := 0.0ft$

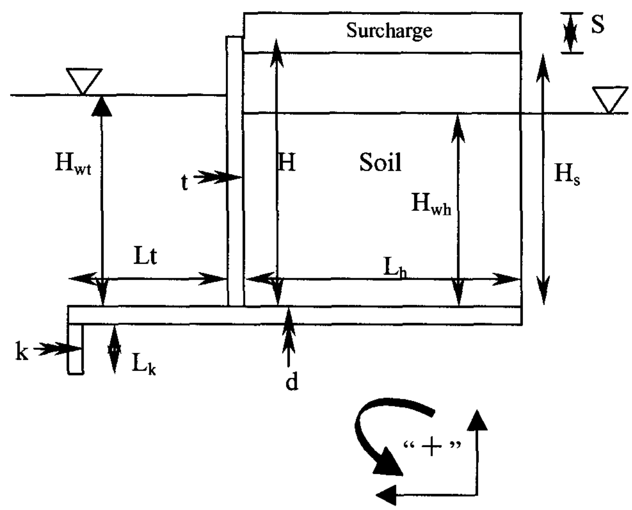
Key thickness $k := 0.0ft$

Heel length $L_h := 0.0ft$

Toe length $L_t := 0.0ft$

Key length $L_k := 0.0ft$

Footing Base Length $L_b := L_t + t + L_h$ $L_b = 6.5\,ft$



Sheet Purpose:

The purpose of this calculation sheet is to complete an appraisel-level design for a full-scale gravity retaining wall system for the purpose of potentially completing a dynamic shake table test on the LHPOST shake table located in San Diego, CA at the NEES Camp Elliott Facilities.

Assumptions:

These calculation were developed under the following assumptions based on an appraisel-level design effort:

1. *The proposed configuration based on the dimensions of the shake table is shown on Figures 1 and 2 - Attachment 1*
2. *The retaining wall and gravity boundary wall will be design using a worst case passive resistance earth pressure load. While this is a conservative approach, it is reasonable assumption for this appraisel level of development and allows for flexibility in terms of selecting earthquake magnitudes and corresponding time histories to be determined during later phases of the project.*
3. *The wall will be designed to structurally perform in the elastic range under the passive earth pressure conditions (see Assumption 2 above). For a limit state design using passive earth pressures, no load factors will be used for strength design.*
4. *Several different soil types will be considered for conceptual-level and final design; however, for this level of study a internal friction angle of 30 degrees will be assumed with no cohesion will be assumed.*
5. *Wall friction will conservatively be neglected for this level of design.*
6. *A phreatic water surface five feet above the top of the footing will be assumed to address the possibility of performing a saturated backfill scenario.*
7. *The moist unit weight of the backfill will be 120 pcf and the saturated unit weight will be 130 pcf.*
8. *Incorporate precast panels lined with neoprene sheets to limit side friction in the non-shaking direction. The precast panels will allow for easier access to the backfill for removal and replacement to accomodate the potential for several test runs using different backfill configurations.*
9. *In the shaking direction, design one side as a flexible cantilever retaining wall and the other side as a stiffer gravity wall section for the purpose of limiting, to the extent possible, boundary effects. Target a relative natural period difference of an order of magnitude. The gravity wall could be replaced with a stiffer steel frame system during later design phases.*
10. *Anchor the concrete sections to the shake table using Nelson studs to simulate retaining systems founded on rock and to eliminate instability issues associated with sliding at the concrete steel table interface.*

Cantilever Gravity Wall Design:

Natural Period of Cantilever Gravity Wall

Use an allowable rational method in accordance with the requirements of ASCE 7, Section 9.14.5.4 to estimate period of walls. Based on *USBR Criteria for Concrete Retaining Walls*.

$$T_{c1} := 0.000425 \cdot \left(\frac{H^2}{t}\right) \cdot \frac{1}{ft} \qquad T_{c1} = 0.01471$$

Compare with Eq. 9.5.5.3.2-1, ACSCE 7

$$C_t := 0.02 \qquad x := 0.75 \qquad h_n := H \cdot \frac{1}{ft} \qquad h_n = 15$$

$$T_{c2} := C_t \cdot h_n^{\,x} \qquad T_{c2} = 0.15$$

$$T_{cant} := \min(T_{c1}, T_{c2}) \qquad T_{cant} = 0.0147 \text{ sec}$$

Uplift

Uplift at heel $\qquad U_h := \text{if}(H_{wh} > 0, H_{wh} + d, 0) \qquad U_h = 7.75\,ft$

Uplift at toe $\qquad U_t := \text{if}(H_{wt} > 0, H_{wt} + d, 0) \qquad U_t = 0$

Properties and Coefficients

Soil moist unit weight $\gamma := 120 \frac{lb}{ft^2}$

Soil saturated unit weight $\gamma_s := 130 \frac{lb}{ft^2}$

Concrete unit weight $\gamma_c := 150 \frac{lb}{ft^2}$

Water unit weight $\gamma_w := 62.4 \frac{lb}{ft^2}$

Slope of Backfill $\beta := 0deg$

Shear strength parameter $\phi := 30.0deg$

Frictional resistance $\delta := 24.2 \cdot deg$ *Sliding resistance - friction resistance - see Attachment 2*

Cohesion $c := 0 \frac{lb}{ft}$

Active pressure coefficient (Coulomb) $K_a := \frac{\cos(\phi)^2}{\left(1 + \sqrt{\frac{\sin(\phi)\cdot\sin(\phi - \beta)}{\cos(\beta)}}\right)^2}$ $K_a = 0.33$

At-rest coefficient $K_o := 1 - \sin(\phi)$ $K_o = 0.5$

Passive pressure coefficient (Coulomb) $K_p := \frac{\cos(\phi)^2}{\left(1 - \sqrt{\frac{\sin(\phi)\cdot\sin(\phi + \beta)}{\cos(\beta)}}\right)^2}$ $K_p = 3$

Seismic earth pressure coefficient $k_h := 0.576$ *Assume the maximum k_h value for M-O design (requires seismic intertia angle be less than or equal to phi angle) for load comparison to passive pressure design*

Seismic Inertia Angle $\psi := \mathrm{atan}(k_h)$ $\psi = 29.94 \cdot deg$

Angle of sliding failure plane $\theta := \mathrm{atan}\left[\frac{L_k}{(L_t + t + L_h)}\right]$ $\theta = 0 \cdot deg$

Allwable Bearing Pressure: *Assume 0.4*F_y for steel shake table = 0.4*36ksi*

$Q_{allowable} := 14400 \frac{lb}{in^2}$ $Q_{allowable} = 2073600 \cdot \frac{lb}{ft^2}$ $Q_{allowableEQ} := Q_{allowable} = 2073600 \cdot \frac{lb}{ft^2}$

The gravity wall stability analyses are computed on a seperate Excel spreadsheet.

Wall Forces

Wall section

$W_w := -\gamma_c \cdot H \cdot t$ $\quad x_w := L_t + \frac{t}{2}$ $\quad y_w := d + \frac{H}{2}$

$W_w = -14625\,\text{lb}$ $\quad x_w = 3.25\,\text{ft}$ $\quad y_w = 7.5\,\text{ft}$

Heel section

$W_h := -\gamma_c \cdot \left(L_h + \frac{t}{2}\right) \cdot d$ $\quad x_h := \frac{L_h + \frac{t}{2}}{2} + L_t + \frac{t}{2}$ $\quad y_h := \frac{d}{2}$

$W_h = 0$ $\quad x_h = 4.88\,\text{ft}$ $\quad y_h = 0$

Toe section

$W_t := -\gamma_c \cdot \left(L_t + \frac{t}{2}\right) \cdot d$ $\quad x_t := \frac{L_t + \frac{t}{2}}{2}$ $\quad y_t := \frac{d}{2}$

$W_t = 0$ $\quad x_t = 1.63\,\text{ft}$ $\quad y_t = 0$

Key section

$W_k := -\gamma_c \cdot L_k \cdot k$ $\quad x_k := \frac{k}{2}$ $\quad y_k := \frac{-L_k}{2}$

$W_k = 0$ $\quad x_k = 0$ $\quad y_k = 0$

Soil Static Forces (Coulomb Theory)

Vertical Forces

$W_s := -\gamma_s \cdot H_{wh} \cdot L_h + -\gamma \cdot (H_s - H_{wh}) \cdot L_h$ $\quad x_s := L_t + t + \frac{L_h}{2}$

$W_s = 0$ $\quad x_s = 6.5\,\text{ft}$

$W_{wt} := -\gamma_w \cdot L_t \cdot H_{wt}$ $\quad y_{wt} := \frac{L_t}{2}$ $\quad y_{wt} = 0$

$W_{wt} = 0$

Surcharge Load for Construction Load Case

$W_{sSUR} := -\gamma \cdot (S) \cdot L_h$ $\quad x_{sSUR} := L_t + t + \frac{L_h}{2}$

$W_{sSUR} = 0$ $\quad x_{sSUR} = 6.5\,\text{ft}$

Horizontal Driving Forces - At-Rest Pressure

$$P_{D1o} := \text{if}\left[(H_{wh}) > 0, \frac{1}{2}\cdot K_o\cdot\gamma\cdot(H_s - H_{wh})^2, \frac{1}{2}\cdot K_o\cdot\gamma\cdot(H_s + d)^2\right] \qquad P_{D1o} = 1576.87\ \text{lb}$$

$$y_{D1o} := \text{if}\left[H_{wh} > 0, H_{wh} + d + \frac{(H_s - H_{wh})}{3}, \frac{(H_s + d)}{3}\right] \qquad y_{D1o} = 10.17\ \text{ft}$$

$$P_{D2o} := \text{if}\left[H_{wh} > 0, K_o\cdot\gamma\cdot(H_s - H_{wh})\cdot(H_{wh} + d), 0\right] \qquad P_{D2o} = 3371.25\ \text{lb}$$

$$y_{D2o} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{2}, 0\right) \qquad y_{D2o} = 3.88\ \text{ft}$$

$$P_{so} := \text{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_o\cdot(\gamma_s - \gamma_w)\cdot(H_{wh} + d)^2, 0\right] \qquad P_{so} = 1015.06\ \text{lb}$$

$$y_{so} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{so} = 2.58\ \text{ft}$$

$$P_{wo} := \text{if}\left[(H_{wh} > 0), \frac{1}{2}\cdot\gamma_w\cdot(H_{wh} + d)^2, 0\right] \qquad P_{wo} = 1873.95\ \text{lb}$$

$$y_{wo} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{wo} = 2.58\ \text{ft}$$

$$P_{SURo} := K_o\cdot\gamma\cdot S\cdot(H_s + d) \qquad P_{SURo} = 0$$

$$y_{SURo} := \frac{H_s + d}{2} \qquad y_{SURo} = 7.5\ \text{ft}$$

Horizontal Resisting Forces - At-Rest Pressure

$$P_{spo} := \text{if}\left[H_{wt} > 0, \frac{-1}{2}\cdot K_o\cdot(\gamma_s - \gamma_w)\cdot(L_k + d + H_{st})^2, \left(\frac{-1}{2}\right)\cdot K_o\cdot\gamma\cdot(L_k + d + H_{st})^2\right] \qquad P_{spo} = 0$$

$$P_{spo} := \text{if}(H_{st} > 0, P_{spo}, 0) \qquad P_{spo} = 0$$

$$y_{spo} := \text{if}\left[H_{wt} > 0, (L_k + d + H_{st})\cdot\frac{1}{3}, 0\right] \qquad y_{spo} = 0$$

$$P_{wpo} := \text{if}\left[H_{wt} > 0, \frac{-1}{2}\cdot\gamma_w\cdot(H_{wt} + d + L_k)^2, 0\right] \qquad P_{wpo} = 0$$

$$y_{wpo} := \text{if}\left(H_{wt} > 0, \frac{H_{wt} + d + L_k}{3}, 0\right) \qquad y_{wpo} = 0$$

Horizontal Driving Forces - Active Pressure

$$P_{D1a} := \text{if}\left[\left(H_{wh}\right) > 0, \frac{1}{2}\cdot K_a\cdot\gamma\cdot\left(H_s - H_{wh}\right)^2, \frac{1}{2}\cdot K_a\cdot\gamma\cdot\left(H_s + d\right)^2\right] \qquad P_{D1a} = 1051.25\,\text{lb}$$

$$y_{D1a} := \text{if}\left[H_{wh} > 0, H_{wh} + d + \frac{\left(H_s - H_{wh}\right)}{3}, \frac{\left(H_s + d\right)}{3}\right] \qquad y_{D1a} = 10.17\,\text{ft}$$

$$P_{D2a} := \text{if}\left[H_{wh} > 0, K_a\cdot\gamma\cdot\left(H_s - H_{wh}\right)\cdot\left(H_{wh} + d\right), 0\right] \qquad P_{D2a} = 2247.5\,\text{lb}$$

$$y_{D2a} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{2}, 0\right) \qquad y_{D2a} = 3.88\,\text{ft}$$

$$P_{sa} := \text{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_a\cdot\left(\gamma_s - \gamma_w\right)\cdot\left(H_{wh} + d\right)^2, 0\right] \qquad P_{sa} = 676.7\,\text{lb}$$

$$y_{sa} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{sa} = 2.58\,\text{ft}$$

$$P_{wa} := \text{if}\left[\left(H_{wh} > 0\right), \frac{1}{2}\cdot\gamma_w\cdot\left(H_{wh} + d\right)^2, 0\right] \qquad P_{wa} = 1873.95\,\text{lb}$$

$$y_{wa} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{wa} = 2.58\,\text{ft}$$

$$P_{SURa} := K_a\cdot\gamma\cdot S\cdot\left(H_s + d\right) \quad \textit{construction surcharge} \qquad P_{SURa} = 0$$

$$y_{SURa} := \frac{H_s + d}{2} \qquad y_{SURa} = 7.5\,\text{ft}$$

Horizontal Resisting Forces - Passive Pressure

$$P_{sp} := \text{if}\left[H_{wt} > 0, \frac{-1}{2}\cdot K_p\cdot\left(\gamma_s - \gamma_w\right)\cdot\left(L_k + d + H_{st}\right)^2, \left(\frac{-1}{2}\right)\cdot K_p\cdot\gamma\cdot\left(L_k + d + H_{st}\right)^2\right] \qquad P_{sp} = 0$$

$$P_{sp} := \text{if}\left(H_{st} > 0, P_{sp}, 0\right) \qquad P_{sp} = 0$$

$$y_{sp} := \text{if}\left[H_{wt} > 0, \left(L_k + d + H_{st}\right)\cdot\frac{1}{3}, 0\right] \qquad y_{sp} = 0$$

$$P_{wp} := \text{if}\left[H_{wt} > 0, \frac{-1}{2}\cdot\gamma_w\cdot\left(H_{wt} + d + L_k\right)^2, 0\right] \qquad P_{wp} = 0$$

$$y_{wp} := \text{if}\left(H_{wt} > 0, \frac{H_{wt} + d + L_k}{3}, 0\right) \qquad y_{wp} = 0$$

Earthquake Forces (Mononobe-Okabe Method)

K_{AE} is the active pressure coefficient for earthquake loading

$$K_{AE} := \frac{\cos(\phi - \psi)^2}{\cos(\psi)\cdot\cos(\psi)\left(1 + \sqrt{\frac{\sin(\phi)\cdot\sin(\phi - \beta - \psi)}{\cos(\psi)\cdot\cos(\beta)}}\right)^2} \qquad K_{AE} = 1.27$$

K_{PE} is the passive pressure coefficient for earthquake loading

$$K_{PE} := \frac{\cos(\phi - \psi)^2}{\cos(\psi)\cdot\cos(\psi)\cdot\left(1 - \sqrt{\frac{\sin(\phi)\cdot\sin(\phi - \beta - \psi)}{\cos(\psi)\cdot\cos(\beta)}}\right)^2} \qquad K_{PE} = 1.4$$

Total Horizontal Driving Seismic Forces - Active Pressure

(USACOE, EM 1110-2-2502, PAGE 3-64)

$$P_{D1AE} := \mathrm{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_{AE}\cdot\gamma\cdot\left(H_s - H_{wh}\right)^2, \frac{1}{2}\cdot K_{AE}\cdot\gamma\cdot\left(H_s + d\right)^2\right] \qquad P_{D1AE} = 4004\,\mathrm{lb}$$

$$P_{D2AE} := \mathrm{if}\left[H_{wh} > 0, K_{AE}\cdot\gamma\cdot\left(H_s - H_{wh}\right)\cdot\left(H_{wh} + d\right), 0\right] \qquad P_{D2AE} = 8560\,\mathrm{lb}$$

$$P_{sAE} := \mathrm{if}\left[H_{wh} > 0, \frac{1}{2}\cdot K_{AE}\cdot\left(\gamma_s - \gamma_w\right)\cdot\left(H_{wh} + d\right)^2, 0\right] \qquad P_{sAE} = 2577\,\mathrm{lb}$$

Dynamic Component of Driving Seismic Forces - Active Pressure (Kramer, Chapter 11)

$$\Delta P_{AE1} := \left(P_{D1AE} - P_{D1a}\right) + \left(P_{D2AE} - P_{D2a}\right) \qquad \Delta P_{AE1} = 9265.85\,\mathrm{lb}$$

$$y_{AE1} := \frac{2\cdot\left(H_s + d\right)}{3} \qquad y_{AE1} = 10\,\mathrm{ft}$$

$$\Delta P_{AE2} := P_{sAE} - P_{sa} \qquad \Delta P_{AE2} = 1900.79\,\mathrm{lb}$$

$$y_{AE2} := \mathrm{if}\left[H_{wh} > 0, \frac{2\cdot\left(H_{wh} + d\right)}{3}, 0\right] \qquad y_{AE2} = 5.17\,\mathrm{ft}$$

Total Horizontal Seismic Resistng Forces - Passive Pressure (USACOE, EM 1110-2-2502, PAGE 3-64)

$$P_{sP} := \text{if}\left[H_{wt} > 0, \frac{-1}{2} \cdot K_{PE} \cdot (\gamma_s - \gamma_w) \cdot (L_k + d + H_{st})^2, \frac{-1}{2} \cdot K_{PE} \cdot \gamma \cdot (H_{st} + d)^2\right] \qquad P_{sP} = 0$$

$$P_{sP} := \text{if}(H_{st} > 0, P_{sP}, 0) \qquad P_{sP} = 0$$

$$y_{sP} := \text{if}\left(H_{st} > 0, \frac{L_k + d + H_{st}}{3}, 0\right) \qquad y_{sP} = 0$$

Dynamic Component of Resisting Seismic Forces - Passive Pressure

(Kramer, Chapter 11)

$$\Delta P_{PE} := P_{sP} - P_{sp} \qquad \Delta P_{PE} = 0$$

$$y_{PE} := \text{if}\left[H_{st} > 0, \frac{2 \cdot (L_k + d + H_{st})}{3}, 0\right] \qquad y_{PE} = 0$$

Horizontal Driving Forces - Passive Pressure - Limit State Earthquake

$$P_{D1p} := \text{if}\left[(H_{wh}) > 0, \frac{1}{2} \cdot K_p \cdot \gamma \cdot (H_s - H_{wh})^2, \frac{1}{2} \cdot K_p \cdot \gamma \cdot (H_s + d)^2\right] \qquad P_{D1p} = 9461.25\,\text{lb}$$

$$y_{D1p} := \text{if}\left[H_{wh} > 0, H_{wh} + d + \frac{(H_s - H_{wh})}{3}, \frac{(H_s + d)}{3}\right] \qquad y_{D1p} = 10.17\,\text{ft}$$

$$P_{D2p} := \text{if}\left[H_{wh} > 0, K_p \cdot \gamma \cdot (H_s - H_{wh}) \cdot (H_{wh} + d), 0\right] \qquad P_{D2p} = 20227.5\,\text{lb}$$

$$y_{D2p} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{2}, 0\right) \qquad y_{D2p} = 3.88\,\text{ft}$$

$$P_{spd} := \text{if}\left[H_{wh} > 0, \frac{1}{2} \cdot K_p \cdot (\gamma_s - \gamma_w) \cdot (H_{wh} + d)^2, 0\right] \qquad P_{spd} = 6090.34\,\text{lb}$$

$$y_{spd} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{spd} = 2.58\,\text{ft}$$

$$P_{wpd} := \text{if}\left[(H_{wh} > 0), \frac{1}{2} \cdot \gamma_w \cdot (H_{wh} + d)^2, 0\right] \qquad P_{wpd} = 1873.95\,\text{lb}$$

$$y_{wpd} := \text{if}\left(H_{wh} > 0, \frac{H_{wh} + d}{3}, 0\right) \qquad y_{wpd} = 2.58\,\text{ft}$$

$P_{SURp} := K_p \cdot \gamma \cdot S \cdot (H_s + d)$ *construction surcharge* $\qquad P_{SURp} = 0$

$$y_{SURp} := \frac{H_s + d}{2} \qquad y_{SURp} = 7.5\,\text{ft}$$

Horizontal Concrete Inertial Forces (ASCE 7, Section 9.14.5)

$k_h := 1.0$ *compute inertial forces based on test peak of 1.0g*

$W_{he} := -k_h \cdot W_h$ $\qquad y_{he} := y_h$

$W_{he} = 0$ $\qquad y_{he} = 0$

$W_{we} := -k_h \cdot W_w$ $\qquad y_{we} := y_w$

$W_{we} = 14625\ \text{lb}$ $\qquad y_{we} = 7.5\ \text{ft}$

$W_{te} := -k_h \cdot W_t$ $\qquad y_{te} := y_t$

$W_{te} = 0$ $\qquad y_{te} = 0$

$W_{ke} := -k_h \cdot W_k$ $\qquad y_{ke} := y_k$

$W_{ke} = 0$ $\qquad y_{ke} = 0$

Horizontal Soil Inertial Forces

$W_{sEq} := -k_h \cdot W_{sSUR}$ $\qquad y_{sEq} := d + \frac{H_s}{2}$

$W_{sEq} = 0$ $\qquad y_{sEq} = 7.5\ \text{ft}$

Hydrodynamic Force - Westergaard's Equation

Load for free pore water conditions in silty sand and gravel backfill or free standing water.

$$C_e := 51.0 \cdot \frac{\text{lb}}{\text{ft}^2}$$

$P_{Re} := \frac{2}{3} \cdot C_e \cdot k_h \cdot H_{wh}^2$ $\qquad y_{Re} := .4 \cdot H_{wh} + d$

$P_{Re} = 2042.12\ \text{lb}$ $\qquad y_{Re} = 3.1\ \text{ft}$

$P_{He} := \frac{2}{3} \cdot C_e \cdot k_h \cdot H_{wt}^2$ $\qquad y_{He} := .4 \cdot H_{wt} + d$

$P_{He} = 0$ $\qquad y_{He} = 0$

Uplift Forces

$U_1 := \gamma_w \cdot \frac{1}{2} \cdot (U_h - U_t) \cdot L_b$ $\qquad x_{U1} := \frac{2}{3} \cdot L_b$

$U_1 = 1571.7\ \text{lb}$ $\qquad x_{U1} = 4.33\ \text{ft}$

$U_2 := \gamma_w \cdot U_t \cdot L_b$ $\qquad x_{U2} := \frac{1}{2} \cdot L_b$

$U_2 = 0$ $\qquad x_{U2} = 3.25\ \text{ft}$

The gravity wall stability analyses are computed on a seperate Excel spreadsheet.

CONCRETE STRENGTH DESIGN

Conservatively use limit state passive earth pressure for appraisel-level strength design with no load factors and include strength reduction factors

Concrete Strength Design Parameters

Concrete strength $f_c := 4000\dfrac{lb}{in^2}$ Steel strength $f_y := 60000\dfrac{lb}{in^2}$

Load Factors (excluded for passive case) $LF_D := 1.2$ Dead $LF_H := 1.6$ Soil $LF_E := 1.0$ Earhtquake

Strength Reduction Factors $\phi_m := 0.9$ Bending $\phi_v := 0.75$ Shear

Minimum Steel ratio

$\rho_{min} := 0.0025$ ACI 14.3.3.b $\rho_{min} = 0.0025$

Element Geometry

Footing thickness $b_f := d$ Wall thickness $b_w := t$

Key thickness $b_k := k$ Unit width $b := 12in$

Distance to bottom footing steel $d_{fb} := b_f - 3.5in$ $d_{fb} = -3.5 \cdot in$

Distance to top footing steel $d_{ft} := b_f - 2.5in$ $d_{ft} = -2.5 \cdot in$

Distance to steel in wall $d_w := b_w - 2.5in$ $d_w = 75.5 \cdot in$

Distance to steel in key $d_k := b_k - 2.5in$ $d_k = -2.5 \cdot in$

Wall Stem - Recompute Lateral Forces for Structural Design of Wall Stem

Horizontal Forces - At-Rest Earth Pressure for Usual Load Case

$$P_{D1ows} := \frac{1}{2} \cdot K_o \cdot \gamma \cdot (H_s - H_{wh})^2 \qquad y_{D1ows} := H_{wh} + \frac{(H_s - H_{wh})}{3}$$

$$P_{D1ows} = 1576.87\,lb \qquad y_{D1ows} = 10.17\,ft$$

$$P_{D2ows} := K_o \cdot \gamma \cdot (H_s - H_{wh}) \cdot (H_{wh}) \qquad y_{D2ows} := \frac{H_{wh}}{2}$$

$$P_{D2ows} = 3371.25\,lb \qquad y_{D2ows} = 3.88\,ft$$

$$P_{sows} := \frac{1}{2} \cdot K_o \cdot (\gamma_s - \gamma_w) \cdot (H_{wh})^2 \qquad y_{sows} := \frac{H_{wh}}{3}$$

$$P_{sows} = 1015\,lb \qquad y_{sows} = 2.58\,ft$$

$$P_{wows} := \frac{1}{2} \cdot \gamma_w \cdot (H_{wh})^2 \qquad y_{wows} := \frac{H_{wh}}{3}$$

$$P_{wows} = 1873.95\,lb \qquad y_{wows} = 2.58\,ft$$

Horizontal Forces - Active Earth Pressure for Construction Load Case

$$P_{D1aws} := \frac{1}{2} \cdot K_a \cdot \gamma \cdot (H_s - H_{wh})^2 \qquad y_{D1aws} := H_{wh} + \frac{(H_s - H_{wh})}{3}$$

$$P_{D1aws} = 1051.25\ \text{lb} \qquad y_{D1aws} = 10.17\ \text{ft}$$

$$P_{D2aws} := K_a \cdot \gamma \cdot (H_s - H_{wh}) \cdot (H_{wh}) \qquad y_{D2aws} := \frac{H_{wh}}{2}$$

$$P_{D2aws} = 2247.5\ \text{lb} \qquad y_{D2aws} = 3.88\ \text{ft}$$

$$P_{saws} := \frac{1}{2} \cdot K_a \cdot (\gamma_s - \gamma_w) \cdot (H_{wh})^2 \qquad y_{saws} := \frac{H_{wh}}{3}$$

$$P_{saws} = 677\ \text{lb} \qquad y_{saws} = 2.58\ \text{ft}$$

$$P_{waws} := \frac{1}{2} \cdot \gamma_w \cdot (H_{wh})^2 \qquad y_{waws} := \frac{H_{wh}}{3}$$

$$P_{waws} = 1873.95\ \text{lb} \qquad y_{waws} = 2.58\ \text{ft}$$

$P_{SURaws} := K_a \cdot \gamma \cdot S \cdot (H_s)$ *Construction Surcharge* $\qquad y_{SURaws} := \frac{H_s}{2}$

$$P_{SURaws} = 0 \qquad y_{SURaws} = 7.5\ \text{ft}$$

Earthquake Forces - Active Pressures

Horizontal Forces - Active Earth Pressure for Extreme Load Case

$$P_{D1ews} := \frac{1}{2} \cdot K_{AE} \cdot \gamma \cdot (H_s - H_{wh})^2 \qquad P_{D1ews} = 4004.1\ \text{lb} \qquad y_{wews} := \frac{H}{2}$$

$$P_{D2ews} := K_{AE} \cdot \gamma \cdot (H_s - H_{wh}) \cdot (H_{wh}) \qquad P_{D2ews} = 8560.5\ \text{lb} \qquad y_{sEqews} := \frac{H_s}{2}$$

$$P_{sews} := \frac{1}{2} \cdot K_{AE} \cdot (\gamma_s - \gamma_w) \cdot (H_{wh})^2 \qquad P_{sews} = 2577\ \text{lb}$$

Horizontal Forces - Active Earth Dynamic Components

$$\Delta P_{AE1ews} := (P_{D1ews} - P_{D1aws}) + (P_{D2ews} - P_{D2aws}) \qquad y_{AE1ews} := \frac{2 \cdot (H_s)}{3}$$

$$\Delta P_{AE1ews} = 9265.85\ \text{lb} \qquad y_{AE1ews} = 10\ \text{ft}$$

$$\Delta P_{AE2ews} := P_{sews} - P_{saws} \qquad y_{AE2ews} := \frac{2 \cdot (H_{wh})}{3}$$

$$\Delta P_{AE2ews} = 1900.79\ \text{lb} \qquad y_{AE2ews} = 5.17\ \text{ft}$$

Horizontal Forces - Limit State Earthquake - Passive Earth Pressure

$P_{D1pws} := \frac{1}{2} \cdot K_p \cdot \gamma \cdot \left(H_s - H_{wh}\right)^2$

$y_{D1pws} := H_{wh} + \frac{\left(H_s - H_{wh}\right)}{3}$

$P_{D1pws} = 9461.25\ \text{lb}$

$y_{D1pws} = 10.17\ \text{ft}$

$P_{D2pws} := K_p \cdot \gamma \cdot \left(H_s - H_{wh}\right) \cdot \left(H_{wh}\right)$

$y_{D2pws} := \frac{H_{wh}}{2}$

$P_{D2pws} = 20227.5\ \text{lb}$

$y_{D2pws} = 3.88\ \text{ft}$

$P_{spws} := \frac{1}{2} \cdot K_p \cdot \left(\gamma_s - \gamma_w\right) \cdot \left(H_{wh}\right)^2$

$y_{spws} := \frac{H_{wh}}{3}$

$P_{spws} = 6090\ \text{lb}$

$y_{spws} = 2.58\ \text{ft}$

$P_{wpws} := \frac{1}{2} \cdot \gamma_w \cdot \left(H_{wh}\right)^2$

$y_{wpws} := \frac{H_{wh}}{3}$

$P_{wpws} = 1873.95\ \text{lb}$

$y_{wpws} = 2.58\ \text{ft}$

$P_{SURpws} := K_p \cdot \gamma \cdot S \cdot \left(H_s\right)$ *Construction Surcharge*

$y_{SURpws} := \frac{H_s}{2}$

$P_{SURpws} = 0$

$y_{SURpws} = 7.5\ \text{ft}$

Summation of Forces

ΣF_x Horizontal Force

Usual Load Case (At-Rest Pressures)

$F_{xows} := P_{D1ows} + P_{D2ows} + P_{sows} + P_{wows}$ $\quad F_{xows} = 7837\ \text{lb}$

Construction Load Case (Active Pressures)

$F_{xaws} := P_{D1aws} + P_{D2aws} + P_{saws} + P_{waws} + P_{SURaws}$ $\quad F_{xaws} = 5849\ \text{lb}$

Seismic Load Case (Active Pressures)

$F_{Ex1} := \Delta P_{AE1ews} + \Delta P_{AE2ews} + W_{we} + W_{sEq} + P_{Re} + P_{He}$ $\quad F_{Ex1} = 27834\ \text{lb}$

$F_{Ex2} := P_{D1aws} + P_{D2aws} + P_{saws} + P_{waws}$ $\quad F_{Ex2} = 5849\ \text{lb}$

$F_{Ex} := F_{Ex1} + F_{Ex2}$ $\quad F_{Ex} = 33683.17\ \text{lb}$

Seismic Limit State Case (Passive Pressures)

$F_{ExLS1} := W_{we} + W_{sEq} + P_{Re} + P_{He}$ $\quad F_{ExLS1} = 16667\ \text{lb}$

$F_{ExLS2} := P_{D1pws} + P_{D2pws} + P_{spws} + P_{wpws}$ $\quad F_{ExLS2} = 37653\ \text{lb}$

$F_{ExLS} := F_{ExLS1} + F_{ExLS2}$ $\quad F_{ExLS} = 54320.16\ \text{lb}$

Factored Design Shear - ACI 318 & ASCE 7

$V_{uLLo} := LF_H \cdot F_{xows}$ (Static - At-Rest) $V_{uLLo} = 12539.41\ lb$

$V_{uLLa} := LF_H \cdot F_{xaws}$ (Static - Active) $V_{uLLa} = 9359\ lb$

$V_{uE} := LF_E \cdot F_{Ex1} + LF_H \cdot F_{Ex2}$ (Seismic - Active) $V_{uE} = 37193\ lb$

$V_{uEL} := F_{ExLS1} + F_{ExLS2}$ (Seismic Limit State- Passive) $V_{uEL} = 54320\ lb$

$V_{uMax} := \max\left(V_{uLLo}, V_{uLLa}, V_{uE}, V_{uEL}\right)$ $V_{uMax} = 54320\ lb$

Wall Stem Shear Check

$V_c := 2 \cdot \sqrt{f_c \cdot \frac{lb}{in^2}} \cdot b \cdot d_w$ $V_c = 114601\ lb$

$\phi V_n := \phi_v \cdot V_c$ $\phi V_n = 85951\ lb$

$Shear_{stem} := if\left(V_{uMax} \le \phi V_n, \text{"OKAY"}, \text{"NG"}\right)$ $Shear_{stem} = \text{"OKAY"}$

Summation of Moments

Max Flexural Moment

Usual Load Case (At-Rest Pressures)

$M_{ows} := P_{D1ows} \cdot y_{D1ows} + P_{D2ows} \cdot y_{D2ows} + P_{sows} \cdot y_{sows} + P_{wows} \cdot y_{wows}$

$M_{ows} = 36558\ lb \cdot ft$

Max Flexural Moment

Construction Load Case (Active Pressures)

$M_{aws} := P_{D1aws} \cdot y_{D1aws} + P_{D2aws} \cdot y_{D2aws} + P_{saws} \cdot y_{saws} + P_{waws} \cdot y_{waws} + P_{SURaws} \cdot y_{SURaws}$

$M_{aws} = 25986\ lb \cdot ft$

Seismic Load Case (Active Pressures)

$M_{Ews1} := P_{D1aws} \cdot y_{D1aws} + P_{D2aws} \cdot y_{D2aws} + P_{saws} \cdot y_{saws} + P_{waws} \cdot y_{waws}$

$M_{Ews2} := \Delta P_{AE1ews} \cdot y_{AE2ews} + \Delta P_{AE2ews} \cdot y_{AE2ews} + P_{Re} \cdot y_{Re} + P_{He} \cdot y_{He} + W_{we} \cdot y_{wews} + W_{sEq} \cdot y_{sEqews}$

$M_{Ews1} = 25985.96\ lb \cdot ft$ $M_{Ews2} = 173712.4\ lb \cdot ft$ $M_{Ews} := M_{Ews1} + M_{Ews2} = 199698\ lb \cdot ft$

Seismic Limit State Case (Passive Pressures)

$M_{Ews1LS} := P_{D1pws} \cdot y_{D1pws} + P_{D2pws} \cdot y_{D2pws} + P_{spws} \cdot y_{spws} + P_{wpws} \cdot y_{wpws}$

$M_{Ews2LS} := P_{Re} \cdot y_{Re} + P_{He} \cdot y_{He} + W_{we} \cdot y_{wews} + W_{sEq} \cdot y_{sEqews}$

$M_{Ews1LS} = 195145\ lb \cdot ft$ $M_{Ews2LS} = 116018\ lb \cdot ft$ $M_{EwsLS} := M_{Ews1LS} + M_{Ews2LS} = 311163\ lb \cdot ft$

Factored Design Moment - ACI 318 & ASCE 7

$M_{uLLo} := LF_H \cdot M_{ows}$ (Static - At-Rest) $M_{uLLo} = 58493\,lb \cdot ft$

$M_{uLLa} := LF_H \cdot M_{aws}$ (Static - Active) $M_{uLLa} = 41578\,lb \cdot ft$

$M_{uE} := LF_H \cdot M_{Ews1} + LF_E \cdot M_{Ews2}$ (Seismic - Active) $M_{uE} = 215290\,lb \cdot ft$

$M_{uELS} := M_{EwsLS}$ (Seismic Limit State- Passive) $M_{uELS} = 311163\,lb \cdot ft$

$M_{uMaxws} := \max(M_{uLLo}, M_{uLLa}, M_{uE}, M_{uELS})$ $M_{uMaxws} = 311163\,lb \cdot ft$

$$k_u := 1 - \sqrt{1 - \frac{M_{uMaxws}}{0.425 \cdot \phi_m \cdot f_c \cdot b \cdot d_w^2}}$$ $k_u = 0.02$

$$A_{sws} := \frac{0.85 \cdot f_c \cdot k_u \cdot b \cdot d_w}{f_y}$$ $A_{sws} = 0.92 \cdot in^2$

$A_{smin} := \rho_{min} \cdot b \cdot d_w$ $A_{smin} = 2.26 \cdot in^2$

$A_{sminalt} := A_{sws} \cdot 1.333$ (ACI 318, Section 10.5.3) $A_{sminalt} = 1.23 \cdot in^2$

#10 @ 12" Each Face Vertical Bars A_s = 1.27 in.^{2}

Conservatively use the same reinforcement on inside face of wall.

Temperature and Shrinkage Steel - USACE EM 1110 -2-2104

$$A_{ts} := \frac{.0028 \cdot b \cdot b_w}{2}$$ $A_{ts} = 1.31 \cdot in^2$ **#10 @ 12" Each Face A_s = 1.27 in.^{2}**

Summary of Thickness and Reinforcement Requirements

Usual, Construction and Extreme Load Combinations

	Primary Reinforcement	Temperature and Shrink Reinforcement	Thickness
Wall Stem - Outside	#10 @ 12" Vertical	#7 @ 12" Horizontal	3'-0" - Top of Wall
Wall Stem - Inside	#10 @ 12" Vertical	#7 @ 12" Horizontal	10'-0" - Bot of Wall
Footing - Bottom	N/A - Gravity Wall	#N/A - Gravity Wall	N/A - Gravity Wall
Footing - Top	N/A - Gravity Wall	#N/A - Gravity Wall	N/A - Gravity Wall

Determine Welded Stud Requirements for At-Rest Conditions

$Resist_{studO} := 10600 \cdot lb$ Required shear anchorage - at-rest earth pressure - Includes FS = 2.0 - See attached stability analysis

$Stud_{Vcap} := 21137 \cdot lb$ Shear capacity of 7/8" diameter S3L Nelson Stud - see attached

$$No_{Stud} := \frac{Resist_{studO}}{Stud_{Vcap}}$$ Number of studs required per foot length of wall

$No_{Stud} = 0.5$ Say 1 stud required per foot length of wall for shear

Determine Welded Stud Requirements for Passive Limit State Conditions

$Resist_{studP} := 34400 \cdot lb$ Required shear anchorage - passive earth pressure - Includes FS = 1.0 See attached stability analysis

$Stud_{Vcap} := 21137 \cdot lb$ Shear capacity of 7/8" diameter S3L Nelson Stud - see attached

$$No_{Stud} := \frac{Resist_{studP}}{Stud_{Vcap}}$$ Number of studs required per foot length of wall

$No_{Stud} = 1.63$ Say 2 stud required per foot length of wall for shear

Determine Welded Stud Requirements for Tension During Shaking

Seismic Limit State - Passive

$\sigma_{heelfaceP} := 107.2 \cdot \frac{lb}{in^2}$ Uplift pressure at heel - see attached stability analysis

$L_{tens} := 4.63 \cdot ft$ Base length in tension - see attached stability analysis

$\sigma_{avgheelP} := \frac{\sigma_{heelfaceP}}{2}$ $\sigma_{avgheelP} = 7718.4\, lb \cdot ft^{-2}$

$P_{tensEL} := \sigma_{avgheelP} \cdot L_{tens} \cdot 1 \cdot ft$ $P_{tensEL} = 35736\, lb$

$Stud_{Pcap} := 0.6 \cdot 23100 \cdot lb$ Tensile capacity of 7/8" diameter S3L Nelson Stud - 0.6Fy - see attached

$Stud_{Pcap} = 13860\, lb$

$No_{PStud} := \frac{P_{tensEL}}{Stud_{Pcap}}$ Number of studs required per foot length of wall

$No_{PStud} = 2.58$ Say 2 studs required per foot length of wall for tensioin

$No_{StudTot} := No_{Stud} + No_{PStud}$

$No_{StudTot} = 4.21$ Say 5 studs required per foot length of wall - Assume 2' x 2' Footing Grid Pattern

Compute Gravity Wall Stiffness

$E_c := 57000 \cdot \sqrt{f_c \cdot \frac{in^2}{lb}} \cdot \frac{lb}{in^2}$ $\qquad$ $E_c = 3604996.53 \cdot \frac{lb}{in^2}$

$I := \frac{1}{12} \cdot b \cdot L_b^{\,3}$

$I = 474552 \cdot in^4$

$K_{grav} := \frac{15 \cdot E_c \cdot I}{H^3}$ Cantilver beam with triangular shaped distributed load - Ref. AISC, Table 3-23, Case 18

$K_{grav} = 4400099 \cdot \frac{lb}{in}$

Compute Relative Stiffness Ratio

$K_{cant} := 575790 \cdot \frac{lb}{in}$ From cantilever wall design

$K_{relative} := \frac{K_{grav}}{K_{cant}}$ $\qquad$ $K_{relative} = 7.64$ $\qquad$ Almost one order of magnitude stiffer - say OKAY

SSINT - FULL-SCALE SHAKE TABLE TEST
STATIC STABILITY OF GRAVITY WALL SECTION
EXTREME CASE

T.O. GRAVITY WALL EL:	15.0		T.O. WATER EL:	7.75	
GRAVITY WALL BASE EL:	0.0		T.O. SOIL EL:	15.0	
			TAILWATER EL:	0.0	
DAM HEIGHT:	15.0	feet	U/S FACE SLOPE:	0.00	H : 1V
DRAINLINE:	10.0	feet	D/S FACE SLOPE:	0.47	H : 1V
CREST WIDTH:	3.00	feet	CONC. UNIT WGT.:	150	pcf
CHIMNEY HEIGHT:	0.0	feet	TAN PHI:	0.45	
BASE WIDTH:	10.0	feet	SOIL UNIT WGT.:	120	pcf (Moist)
			SOIL UNIT WGT.:	130	pcf (Sat)
			Ko:	0.5	at-rest coeff
			Kp:	3.0	passive coeff
			k_h:	1.0	g (peak test g)

LOADS AND MOMENTS:

APPLIED LOAD	FORCE (kips)	ARM LOCATION (heel @ 0,0) X	Y	ARM DIST. (feet)	MOMENT (kip-ft)	
V_1	6.8	1.50		3.50	24	
V_2	7.9	5.33		-0.33	-3	
Vw	0.0	10.00		-5.00	0	
V_{1E}	6.8		7.50	-7.50	-51	
V_{2E}	7.9		5.00	-5.00	-39	
P_{D1o}	1.6		10.17	-10.17	-16	
P_{D2o}	3.6		3.88	-3.88	-14	
Pspo	1.0		2.58	-2.58	-3	
Pw	1.9		2.58	-2.58	-5	
P_{D1p}	9.5		10.17	-10.17	-96	
P_{D2p}	21.6		3.88	-3.88	-84	
Pspd	6.1		2.58	-2.58	-16	
Pw	1.9		2.58	-2.58	-5	
P_{Re}	0.9		4.75	-4.75	-4	(From retaining wall analysis)

UPLIFT:

Uplift Pressure @ Heel:	0.48	ksf =	3.4	psi
Uplift Pressure @ Toe:	0.00	ksf =	0.0	psi
Uplift Pressure @ Drain:	0.00	ksf =	0.0	psi
Uplift Force on Dam:	2.4	kips		

BASE PRESSURES (At-Rest):

Σ Moments (ft.-kips):	-16	Base Area:	10.0 sq. ft.
Σ Vertical Loads (kips):	15	Length, l:	10.0 feet
Eccentricity, e (ft.):	-1.13		

	Applied Loads	Net
Stress @ u/s Face (psi):	3.30	-0.06
Stress @ d/s Face (psi):	17.01	17.01
Stress @ Drains (psi):	0.00	0.00

Tension on the upstream face.

Length in tension (ft.): 0.04 (Very small - Okay)
Length in compression (ft.): 9.96

SLIDING FACTOR OF SAFETY:

Sliding FOS: 0.68
Required Table Ancor Force (kips): 10.6 for F.S. 2.0

BASE PRESSURES (Passive -Limit State):

Σ Moments (ft.-kips):	-274	Base Area:	10.0 sq. ft.
Σ Vertical Loads (kips):	15	Length, l:	10.0 feet
Eccentricity, e (ft.):	-18.71		

	Applied Loads	Net
Stress @ u/s Face (psi):	-103.84	-107.20
Stress @ d/s Face (psi):	124.15	124.15
Stress @ Drains (psi):	0.00	0.00

Tension on the upstream face.

Length in tension (ft.): 4.63
Length in compression (ft.): 5.37

SLIDING FACTOR OF SAFETY:

Sliding FOS: 0.14
Required Table Ancor Force (kips): 34.4 for F.S. 1.0

Appendix C—Appraisal-Level Cost Estimate

BUREAU OF RECLAMATION **ESTIMATE WORKSHEET** SHEET _ 1_ OF _ 2_

FEATURE:	PROJECT:
Full Scale Retaining Wall - Dynamic Shake Table Test	Soil Structure Interaction - Phase 1, CA
	WOID: SSINT — ESTIMATE LEVEL: Appraisal
	REGION: TSC — UNIT PRICE LEVEL: Apr - 09
	FILE: U:\0001 ESTIMATES\Soil Structure Interaction\[SSI Research Project -Appraisal Estimate - June 2009.xls]SSI Shake Table - 2 of 2

PLANT ACCOUNT	PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
Civil Items							
	1	Mobilization		1	LS	See Sheet 2 of 2	
	2	Structural Concrete		130	CY	$1,120.00	$145,600.00
	3	Reinforcement		15,000	LBS	$2.00	$30,000.00
	4	PVC Waterstop - 9 inch - Center Bulb Type		21	LF	$18.00	$378.00
	5	Expansion Joint Filler - 3/8" Thick - Fiber		320	SF	$4.00	$1,280.00
	6	Expansive Waterstop		182	LF	$7.50	$1,365.00
	7	Construction Joint Preparation - Cant. Wall		60	SF	$5.00	$300.00
	8	Precast Concrete Panels - 15" Thick - 10' Wide x 15' Tall		8	EACH	$10,500.00	$84,000.00
	9	Structural Steel - W16x40		6,000	LBS	$4.50	$27,000.00
	10	Fillet Weld - 5/16"		50	LF	Included in Item 9	
	11	Neoprene Sheeting - 3/4" Thick		1,100	SF	$28.00	$30,800.00
	12	Place Backfill - Type I - 1' Lifts		350	CY	$375.00	$131,250.00
	13	Special Compaction Type I - 7' Adjacent to Walls		4,400	SF	$3.60	$15,840.00
	14	Excavate Backfill - Type I		350	CY	$76.00	$26,600.00
	15	Place Backfill - Type II - 1' Lifts		350	CY	$255.00	$89,250.00
	16	Special Compaction Type II - 7' Adjacent to Walls		4,400	SF	$3.60	$15,840.00
	17	Excavate Backfill - Type II		350	CY	$76.00	$26,600.00
	18	Place Backfill - Type III - 1' Lifts		350	CY	$305.00	$106,750.00
	19	Special Compaction Type III - 7' Adjacent to Walls		4,400	SF	$3.60	$15,840.00
	20	Excavate Backfill - Type III		350	CY	$76.00	$26,600.00
	21	Concrete Demolition		130	CY	$132.00	$17,160.00
	22	Table Interface Plate and Connections:		22,700	LBS	$7.25	$164,575.00
		A. Table Interface Plate		15,300	LBS	Included Above	
		B. Threaded Rods for Interface Plates		1,400	LBS	Included Above	
		C. Bearing Plates for Threaded Rods		5,800	LBS	Included Above	
		D. Nelson 7/8" Dia Welded Studs		200	LBS	Included Above	
	23	Instrument Frames HSS 8x8x5/16 Gr. 46 (7 Total Frames)		18,000	LBS	$6.75	$121,500.00
	24	Removals - Demobilization - Site Cleanup		1	LS	$64,000.00	$64,000.00
		SUBTOTAL CIVIL ITEMS					**$1,142,528.00**
Instrumentation Items							
	25	Strain Gages		320	EACH	$180.00	$57,600.00
	26	Displacement Transducer		66	EACH	$2,000.00	$132,000.00
	27	Accelerometers		228	EACH	$900.00	$205,200.00
	28	Shape-tape		90	LF	$280.00	$25,200.00
	29	Tactical Pressure Sensors (4' x 4' Grid)		32	EACH	$1,940.00	$62,080.00
		SUBTOTAL INSTRUMENTATION ITEMS					**$482,080.00**

QUANTITIES		PRICES	
BY Steve Dominic	CHECKED [signature] 9/23/2009	BY [signature] T. Hanke	CHECKED [signature] 6/16/09
DATE PREPARED May 17,2009	PEER REVIEW / DATE —	DATE PREPARED June 16, 2009	PEER REVIEW / DATE [initials] 6/16/09

BUREAU OF RECLAMATION **ESTIMATE WORKSHEET**

FEATURE: **Full Scale Retaining Wall** **- Dynamic Shake Table Test**	**PROJECT:** **Soil Structure Interaction - Phase 1, CA** **WOID:** SSINT **ESTIMATE LEVEL:** Appraisal **REGION:** TSC **UNIT PRICE LEVEL:** Apr - 09 **FILE:** U:\0001 ESTIMATES\Soil Structure Interaction\[SSI Research Project -Appraisal Estimate - June 2009.xls]SSI Shake Table - 2 of 2

PLANT ACCOUNT	PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
Engineer Labor During Testing							
	30	Estimate 2 Full-time Engineers for ~ 6 Months		220	Days	$1,390.00	$305,800.00
		SUBTOTAL ENGINEER LABOR ITEMS					$305,800.00
Summary :-							
		Subtotals:-					
		- Civil Items					$1,142,528.00
		- Instrumentation Items					$482,080.00
		- Engineer Labor					$305,800.00
		Subtotal 1					$1,930,408.00
		Mobilization	5%	+/-			$100,000.00
		Subtotal 1 with Mobilization					$2,030,408.00
		Design Contingencies	15%	+/-			$304,561.00
		Subtotal 2 = Subtotal 1 + Design Contingencies					$2,334,969.00
		Allowance for Procurement Strategies (APS)	5%	+/-			$116,749.00
		Type of solicitation assumed is: 8a RFP Negotiated					
		Subtotal 3 = Subtotal 2 + APS					$2,451,718.00
		CONTRACT COST					$2,500,000.00
		Construction Contingencies	15%	+/-			$300,000.00
		FIELD COST					$2,800,000.00
		Ref.: For appropriate use and terminology, see Reclamation Manual, Directives and Standards FAC; 09-01, 09-02 and 09-03.					

QUANTITIES		PRICES	
BY Steve Dominic	**CHECKED** 9/23/2009	**BY** T. Hanke	**CHECKED** 6/16/09
DATE PREPARED May 17,2009	**PEER REVIEW / DATE** –	**DATE PREPARED** June 16, 2009	**PEER REVIEW / DATE** HCN 6/16/09

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY SJD	DATE 05/19/09	PROJECT Soil-Structure Interaction	SHEET 1 OF 11
CHKD BY	DATE	FEATURE	
DETAILS Full-Scale Retaining Wall Shake Table Test			

Quantity Calculations

Pay Item 1 - Mobilization

Lump Sum Item

Pay Item 2 - Structural Concrete

→ Retaining Wall

Area → $A_{Ret} = \frac{1.25' + 2.75'}{2} \times 12.25' + 13.5' \times 2.75'$

$= 24.5\ ft^2 + 37.125\ ft^2$

$= 61.625\ ft^2$

Length → $L_{Ret} = 25' - 0.75' - 0.75' - 1.25' - 1.25'$

$= 21'$

Vol. → $V_{Ret} = A_{Ret} \times L_{Ret}$

$= 61.625\ ft^2 \times 21'$

$= 1294\ ft^3$

$= 48$ c.y. say 50 c.y.

→ Gravity Wall

Area → $A_{grav} = \frac{3' + 10'}{2} \times 15'$

$= 97.5\ ft^2$

Length → $L_{grav} = 21'$

Vol. → $V_{grav} = A_{grav} \times L_{grav}$

$= 97.5\ ft^2 \times 21'$

$= 2048\ ft^3$

$= 75.8$ c.y. say 80 c.y.

→ Total Struct Concrete

$V_{tot} = V_{Ret} + V_{grav} = 50$ c.y. $+ 80$ c.y. $= 130$ c.y.

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET
SJD	05/19/09	Soil-Structure Interaction	2 OF 11
CHKD BY	DATE	FEATURE	
DETAILS: Full-Scale Retaining Wall Shake Table Test			

Pay Item 3 – Reinforcement

→ Footing Bottom Face – #9 @ 12"

$$\ell_{FBF} = (13'\text{-}6") - 2" - 2" = 158" = 13.17' \text{ per ft. of footing}$$
$$W_{\#9} = 3.400 \text{ lbs/ft}$$
$$W_{FBF} = 13.17' \times 3.4 \text{ lbs/ft}$$
$$= 45 \text{ lbs}$$

→ Footing Top Face – #10 @ 12"

$$\ell_{FTF} = \ell_{FBF} = 13.17'$$
$$W_{\#10} = 4.303 \text{ lbs/ft}$$
$$W_{FTF} = 13.17' \times 4.303 \text{ lbs/ft}$$
$$= 57 \text{ lbs} \quad \text{per ft. of footing}$$

→ Vertical Dowels – #10 @ 12"

$$\ell_{dowel} = 79" + 33" - 3" + 22" \text{(hook)}$$
$$= 131"$$
$$= 10.92'$$
$$W_{\#10} = 4.303 \text{ lbs/ft}$$
$$W_{dowel} = 10.92' \times 2 \text{ dowels} \times 4.303 \text{ lbs/ft}$$
$$= 94 \text{ lbs}$$

→ Vert Bars – #10 @ 12" Each Face

$$\ell_{vert} = (15'\text{-}0") - (2'\text{-}9") - 2"$$
$$= 180" - 33" - 2"$$
$$= 145"$$
$$= 12.083'$$
$$W_{\#10} = 4.303 \text{ lbs/ft}$$
$$W_{vert} = 12.083' \times 2 \text{ verts} \times 4.303 \text{ lbs/ft}$$
$$= 104 \text{ lbs}$$

→ Temp & Shrinkage – #7 @ 12" EW, EF

$$\ell_{unit} = 1'$$
$$W_{\#7} = 2.044 \text{ lbs/ft}$$
$$L_{\#7} = (13'\text{-}6") - 6" \times 2 \text{ Faces} + (12'\text{-}3") - 3" \times 2 \text{ Face}$$
$$= 600"$$
$$= 50'$$
$$N_{\#7} = \frac{50'}{s} + 1 = \frac{50'}{1'} + 1 = 51 \text{ bars}$$

$$W_{T\&S} = 51 \times 1 \times 2.044$$
$$= 104 \text{ lbs per ft. of wall}$$

$$W_{tot_{RET}} = W_{FBF} + W_{FTF} + W_{dowel} + W_{vert} + W_{T\&S}$$
$$= 45 + 57 + 94 + 104 + 104$$
$$= 404 \text{ lbs per ft}$$

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET 3 OF 11
SJD	05/19/09	Soil-Structure Interaction	
CHKD BY	DATE	FEATURE	
DETAILS: Full-scale Retaining Wall Shake Table Test			

Pay Item 3 - Reinforcement (cont.)

→ Bottom face - #10@12"

$$\ell_{BF} = (10'\text{-}0") - 2" - 2" = 116" = 9'\text{-}8" = 9.667'$$

$$w_{\#10} = 4.303 \text{ lbs/ft}$$

$$w_{BF} = 9.667' \times 4.303 \text{ lbs/ft} = \underline{42 \text{ lbs}}$$

→ Top face #10@12"

$$\ell_{TF} = (3'\text{-}0") - 2" - 2" = 32" = 2.667'$$

$$w_{TF} = 2.667' \times 4.303 \text{ lbs/ft} = \underline{12 \text{ lbs}}$$

→ Vert face #10@12"

$$\ell_{VF} = (15'\text{-}0") - 2" - 3" = 175" = 14.583'$$

$$w_{VF} = 14.583' \times 4.303 \text{ lbs/ft} = \underline{63 \text{ lbs}}$$

→ sloped face #10@12"

$$\ell_{SF} = \sqrt{(15')^2 + (7')} - 3" - 2" = 193.6" = 16.14'$$

$$w_{SF} = 16.136' \times 4.303 \text{ lbs/ft} = \underline{69 \text{ lbs}}$$

→ Temp & shrinkage - #7@12"

$$\ell_{unit} = 1'$$

$$w_{\#7} = 2.044 \text{ lbs/ft}$$

$$L_{\#7} = (9.667' + 2.667' + 14.583' + 16.14') = 43.1'$$

$$N_{\#7} = \frac{43'}{s} + 2 = \frac{43'}{1'} + 2 = 45 \text{ bars}$$

$$w_{t\&s} = 45 \times 1 \times 2.044 = 92 \text{ lbs per ft. of wall}$$

$$w_{totGRAV} = w_{BF} + w_{TF} + w_{VF} + w_{SF} + w_{t\&s} = 42 + 12 + 63 + 69 + 92 = 278 \text{ lbs}$$

$$W_{Reinf} = (w_{totRet} + w_{totGRAV}) \times L = (404 \text{ lbs/ft} + 278 \text{ lb/ft}) \times 21'$$

$W_{Reinf} = 14{,}322$ lbs
Add 5% = $1.05 \times 14{,}322 = 15{,}038$
Say $\underline{15{,}000 \text{ lbs}}$

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET 4 OF 11
SJD	05/19/09	Soil-Structure Interaction	
CHKD BY	DATE	FEATURE	
DETAILS: Full-scale Retaining Wall Shake Table Test			

Pay Item 4 - 9" PVC Waterstop

→ Along retaining wall CJ

$\ell_{CJ} = 21'$

$\ell_{PVC} = 21'$

Pay Item 5 - Expansion Joint Filler

→ Between precast panels and retaining walls

Gravity wall section ⇒ $A_{grav} = \frac{3+10}{2} \times 15 \times 2\text{ sides} = 195\text{ sf}$

Cant. Wall ⇒ $A_{cant} = \left[\frac{1.25'+2.75'}{2} \times 12.25 + 2.75' \times 13.5'\right] \times 2\text{ sides} = 123\text{ sf}$

Total $= 195 + 123 = 318\text{ sf}$ say **320 sf**

Pay Item 6 — Expansive Waterstop

→ Along vertical expansion joints and around perimeter interface between shake table and concrete/precast panels

$\ell_{perim} = 21' \times 2\text{ sides} + 40' \times 2\text{ side}$
$= 122'$

$\ell_{vert} = 60'$

$\ell_{tot} = 122' + 60'$
$= 182'$

$\ell_{EWS} = 182'$

Pay Item 7- CJ Joint Prep

→ CJ length = 21'
Jt. width = 2'-9"
CJ Area = 21' × 2.75'
= 57.75 ft²
say 60 sf

Pay Item 8— Precast Panels-15" thick -10'×15'

From layout - 8 total required

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET
SJD	05/19/09	Soil-Structure Interaction	5 OF 11
CHKD BY	DATE	FEATURE	
DETAILS: Full-scale Retaining Wall Shake Table Test			

Pay Item 9 – W16X40 Steel Frame Supports for Precast Panels

$$W_{16X40} = 40 \text{ lbs/ft}$$
$$\ell = 15' \times 10$$
$$= 150'$$
$$W_{16x4} = 40 \text{ lbs/ft} \times 150$$
$$= 6{,}000 \text{ lbs}$$

Pay Item 10 – 5/16" Fillet Welds

Assume 5/16" all around perimeter of W16X40 at connection to shake table

Ref. AISC, Table 1-1

$$\ell_{perim} = 13\tfrac{5}{8}'' \times 2 \text{ sides} + 7'' \times 4 \text{ sides}$$

↑ web "T" ↑ b_f ↑ conserv. estimate

$$= 55.25 \text{ in}$$
$$\ell_{tot} = 55.25 \text{ in} \times 10 \text{ connections}$$
$$= 552.5''$$
$$= 46 \text{ ft} \quad \underline{\text{say } 50 \text{ ft}}$$

Pay Item 11 – Neoprene Sheeting – 3/4" Thick

→ Line backfill along non-shaking direction

$$\ell_{backfill} = (40'-0'') - 3' - 1' = 36'$$
$$h_{backfill} = 15'$$
$$A_{liner} = 36' \times 15' \times 2 \text{sides}$$
$$= 1080 \text{ sf}$$
$$\underline{\underline{\text{say } 1100 \text{ sf}}}$$

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET 6 OF 11
SJD	05/19/09	Soil-Structure Interaction	
CHKD BY	DATE	FEATURE	
DETAILS: Full-Scale Retaining Wall Shake Table Test			

Pay Item 12, 15, 18 - Place Test Run Backfill

→ Assume small roller - except adjacent to walls - 1' lifts

$l_{shake} = [(40'\text{-}0'') - (3'\text{-}0'') - (2'\text{-}0'') - (1'\text{-}0'') - (6'\text{-}6'')]$

Footing toe; Avg. wall t; Grav. Wall Edge Dist; Avg. Grav. wall "t"

$= 27.5'$

$l_{wall} = 21'$

$h = 15'$

Vol. $= 27.5' \times 21' \times 15'$

$= 8662.5 \text{ ft}^3$

$= 321$ c.y.

Add 10% compaction factor ⇒ $1.1 \times 321 = 353$ c.y.

Say 350 c.y. per test cycle

Pay Items 13, 16, 19 - Special Compaction

→ within 7' of wall every lift

$No_{Lifts} = 15$ (1' lifts)

Area $= 15 \times 7 \text{ feet} \times 21 \text{ feet} \times 2 \text{ sides}$

$= 4{,}410$ sf

Say 4,400 sf

Pay Items 14, 17, 20 - Excavation of Backfill per Test Run

per pay Items 12, 15 & 18 vol = 350 c.y.

Pay Item 21 - Concrete Demolition

Vol = 130 c.y. (see Pay Item 2)

Pay Item 24 - Demobilization & Site Cleanup

Lump sum item

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET 7 OF 11
SJD	05/19/09	Soil-Structure Interaction	
CHKD BY	DATE	FEATURE	
DETAILS Full-Scale Retaining Wall Shake Table Test			

Pay Item 22D - Welded studs

→ Per calculations require 2'x2' grid pattern
→ 7/8" Dia. studs x 8 3/16" long

$$A_{ret\,wall} = 21' \times 13.5' = 283.5\ sf$$

$$No.\ stud_{Ret} = \frac{283.5\,sf}{4\,sf/stud} = 71\ \text{studs}$$

$$A_{grav\ wall} = 21' \times 10' = 210\ sf$$

$$No.\ stud_{Grav} = \frac{210}{4\,sf/stud} = 52.5\ \text{studs}$$

Total studs = 71 + 53
= 124 studs

say $\frac{124}{35\ stud/box} = 3.5$ say 4 boxes

Weight per box = 50 lbs x 4 = 200 lbs

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET 8 OF 11
SJD	05/20/09	Soil-Structure Interaction	
CHKD BY	DATE	FEATURE	
DETAILS	Full-Scale Retaining Wall Shake Table Test		

Pay Item 24

→ Strain Gages

see revised instrument. calcs by Travers attached

• Assume 6 lines per wall at 2' spacing along height

• Also assume a replacement rate of 50% per test cycle

$SG_{1test} = 8$ gages per line × 6 lines per wall × 2 walls

$= 96$ gages

$SG_{tot} = \frac{96 \text{ gages}}{\text{test}} \times 3$ test × 0.5 replacement rate

$SG_{tot} = 144$ say 150 strain gages

Pay Item 25

see revised ~~calcs~~ instrum. calcs by Travers attached

→ Displacement Transducers

→ Assume 6 lines per wall w/ 3 transducers per line one at base of footing, one at base of stem, one at top of wall

→ Assume 5 at 5' spacing on top of backfill along 6 lines

$DT_{wall} = 3$ gages per line × 6 lines per wall × 2 walls

$= 36$

$DT_{Fill} = 5$ gages per line × 6 lines

$= 30$ gage

→ DT_s external to fill ∴ assume 0% replacement rate

$DT_{tot} = 36 + 30 = 66$ say 70 displacement transducers

7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY STD	DATE 05/20/09	PROJECT Soil-Structure Interaction	SHEET 9 OF 11
CHKD BY	DATE	FEATURE	
DETAILS Full-Scale Retaining Wall Shake Table Test			

Pay Item 26 – Accelerometers

see revised inst. calcs by Travers attached

→ Assume at same location as DTs

→ Provide additional accelerometers throughout backfill at 5' spacing depth – assume 100% replacement rate

$ACL = 70 \text{ (same as DT)} + \frac{15'}{5'_{spc}} \times 6 \text{ lines} \times 3 \text{ tests} \times 1.0$ Replacement Rate

$= 70 + 54$

$= 124$ say 125 Accelerometers

Pay Item 27 – Slope Acceleration Array (SAA) w/ triaxial MEMS ⇒ Micro-Electro-Mechanical System

see attached

→ Elgamal suggest 3 sensor tapes per wall measuring at 1' increments along wall height

→ Use 3 SAAs per wall.

SAA using MEMS say 6 total × 15' per array ⇐ see attached reference

= 90 ft of SAA tape

Pay Item 28 – Tactical Pressure Sensors (TPS) model 5315 15"×15"

Use lump sum estimate provided by Elgamal – see attached

LS = $60,000

→ Estimate a time requirement for installation of instrument every 2' along instrumentation vertical lines. Per FT, assume ½ day for each location × 12 location (6 per wall) = 6 days per 2 lifts of backfill → probably too conservative → 3 days per lift Assume that rate will be better, → 2 days per lift including instrumentation.

based on economies of scale

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET 10 OF 11
SJD	06/05/09	Soil-Structure Interaction	
CHKD BY	DATE	FEATURE	
DETAILS: Full-Scale Retaining Wall - Shake Table Test			

Pay Item 22A - Table Interface Plate

A36 - 1/2" thick plate

→ Assume coverage of entire table

$$W_{pl} = 25' \times 40' \times \frac{0.375}{12}' \times 490 \text{ pcf} = 15,313 \text{ lbs}$$

Say 15,300 lbs

Pay Item 22B - Threaded Rods for Interface Plates

Gr. 75 - 2" Dia., Length = 6 1/4" =

No. Req'd = 40' x 25' = 1,000 sf

Tie-down grid = 2' x 2' = 4 sf

$$\text{No. Req'd} = \frac{1000}{4} = 250 \text{ tie-downs}$$

$$W_{Rod} = 6\tfrac{1}{4}''/12 \times \frac{\pi (2'')^2}{4} / 144 \times 490 \text{ pcf}$$

$$= 5.57 \text{ lbs each}$$

$$W_{Rod_{Tot}} = 5.57 \times 250 = 1,392 \quad \text{say} \quad 1,400 \text{ lbs}$$

Pay Item 22C - Bearing Plates for Threaded Rods

Plate = 1" x 9" x 9" A36 steel

$$W_{Plate} = 1/12' \times 9/12' \times 9/12' \times 490 \text{ pcf}$$

$$= 22.97 \text{ lbs per plate}$$

$$W_{Pl_{tot}} = 250 \times 22.97$$

$$= 5,742 \quad \text{say} \quad 5,800 \text{ lbs}$$

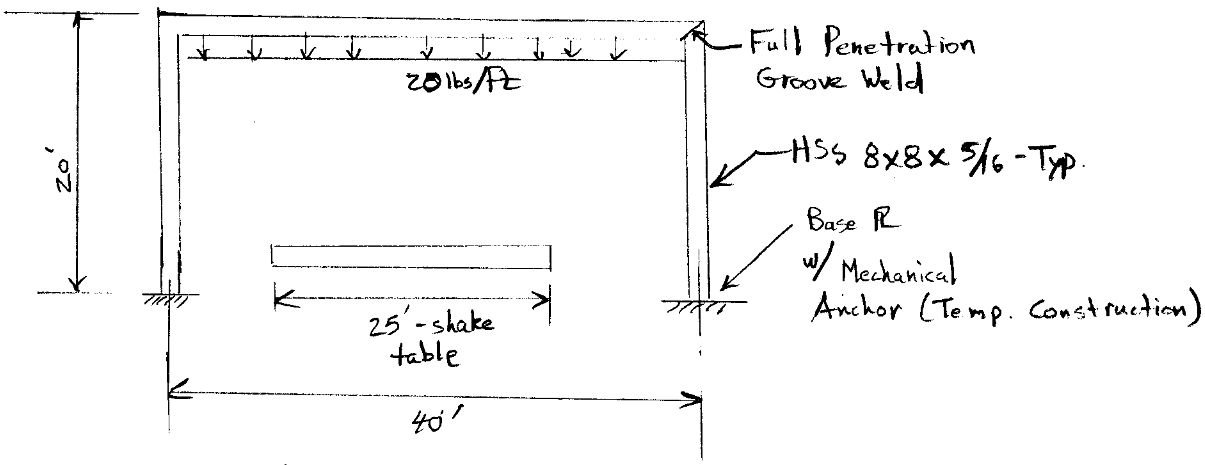
7-1654 (11-94)
Bureau of Reclamation

COMPUTATION SHEET

BY SJD	DATE 06/05/09	PROJECT	SHEET 11 OF 11
CHKD BY	DATE	FEATURE	
DETAILS Full-Scale Retaining Wall Shake Table Test			

Item 23 - Instrument Frame

- Assume simple moment frame designed for a uniform load of 20 lbs/ft for instrumentation mounts and bundled data wires



- size structural tube

$$\sigma = \frac{\frac{.020 \times (40' \times 12)^2}{8}}{S} = \frac{576 \text{ k-in}}{S}$$

$$S_{req} = \frac{576 \text{ k-in}}{0.6 \times 46 \text{ ksi}} = 20.9 \text{ in.}^3$$

HSS 8 x 8 x 5/16 31.79 lbs/ft
$S = 24.9 \text{ in.}^3 > 20.9$ ∴ ok

Use HSS 8x8x5/16 for beams & columns

$$W_{HSS} = [2 \times 20' + 40'] \times 32 \text{ lbs/ft}$$
$$= 2{,}560 \text{ lbs each}$$

No. Req'd = 5 above fill + 2 @ top of walls
= 7

$$W_{HSS_{Tot}} = 2{,}560 \times 7 = 17{,}920 \quad \text{say } \underline{18{,}000 \text{ lbs}}$$

Soil-Structure Interaction
Instrumentation Cost Estimate

Strain Gages	$ 40,933.33
Displacement Transducers	$ 95,610.00
Accelerometers	$ 148,380.00
Shape Tape	$ 18,087.00
Tactile Pressure Transducers	$ 44,593.33
Cost	$ 347,603.67
25% Contingency	$ 86,900.92
TOTAL Estimate	$ 434,504.58

Soil-Structure Interaction
Instrumentation Cost Estimate

Strain Gages

Assumptions

Each of 2 walls to have gages on 6 lines with 8 gages per line
1/3 of gage locations will require 3 gages (rosette)
1/2 of gages will requirement replacement after each test

2	walls
6	lines/wall
8	gages/line
33%	rosettes
50%	replacement rate/test
3	tests
96	gage locations
32	rosette locations
160	gages/test
80	replacments for 2nd and 3rd tests
320	total number of gages

Estimated Unit Costs

strain gages			
assume 4" gages	$	25.00	per gage
installation			
labor cost	$	100.00	per hour
includes			
gage layout		2	hr/wall
surface preparation		5	min/gage
gage installation		5	min/gage
cable connection		5	min/gage
routing to DAQ		20	min/gage
gage protection		20	min/gage
total installation time		55	minutes
supplies	$	10.00	per gage

Estimated Cost

First test			
160	gages		$ 4,000.00
4	layout		$ 400.00
160	installation		$ 14,666.67
160	supplies		$ 1,600.00
		Total	$ 20,666.67
Second and third tests			
80	gages		$ 2,000.00
0	layout		$ -
80	installation		$ 7,333.33
80	supplies		$ 800.00
		Total	$ 10,133.33
TOTAL for three tests			$ 40,933.33

Soil-Structure Interaction
Instrumentation Cost Estimate

Displacement transducers

Assumptions

Each of 2 walls to have gages on 6 lines with 3 transducers per line
Array of sensors on top of fill on a 5x6 grid
No transducers lost during test

2 walls
6 lines/wall
3 gages/line
36 wall transducers

5x6 top surface grid
30 surface transducers

Estimated Unit Costs

wall transducers			
assume ± 0.5" DC LVDT	$	650.00	per LVDT
fixture fabrication	$	100.00	per LVDT
wall transducer installation			
labor cost	$	100.00	per hour
includes			
gage layout		2	hr/wall
surface preparation		5	min/gage
gage installation		5	min/gage
cable fabrication		15	min/gage
routing to DAQ		20	min/gage
total installation time		45	minutes
supplies	$	10.00	per gage
surface transducers			
assume laser displacement	$	2,000.00	per sensor
fixture fabrication	$	100.00	per sensor
surface transducer installation			
labor cost	$	100.00	per hour
includes			
gage layout		4	hr
soil surface		5	min/sensor
gage installation		5	min/sensor
cable fabrication		15	min/sensor
routing to DAQ		20	min/sensor
total installation time		45	minutes
supplies	$	10.00	per gage

Estimated Cost

wall transducers		
36 LVDTs	$	23,400.00
36 fixture fabrication	$	3,600.00
36 installation	$	2,700.00
36 supplies	$	360.00
Total	$	30,060.00
surface transducers		
30 transducers	$	60,000.00
30 fixture fabrication	$	3,000.00
30 installation	$	2,250.00
30 supplies	$	300.00
Total	$	65,550.00
TOTAL for all tests	$	95,610.00

Soil-Structure Interaction
Instrumentation Cost Estimate

Accelerometers

Assumptions

Maximum acceleration of 1 g
Frequency range of 1 to 10 Hz
Each of 2 walls to have gages on 6 lines with 3 transducers per line
Array of accelerometers through fill
1/3 of accelerometers triaxial
Assume 100% replacement of accelerometers in fill for each test

wall accelerometers

2	walls
6	lines/wall
3	gages/line
36	single axis wall accelerometers
33%	triaxial accelerometers
12	triaxial accelerometers
48	wall accelerometers

fill accelerometers

3	levels
6	lines
3	per line
54	fill accelerometers
33%	triaxial accelerometers
18	triaxial accelerometers
72	fill accelerometers

Estimated Unit Costs

accelerometers			
assume PCB 393A03	$	500.00	per gage
installation			
labor cost	$	100.00	per hour
wall accelerometers			
gage layout		2	hr/wall
wall mounting		15	min/gage
installation		5	min/gage
cable fabrication		30	min/gage
routing to DAQ		20	min/gage
wall installation time		70	minutes
supplies	$	10.00	per gage
fill accelerometers			
gage layout		1	hr/level
installation		5	min/gage
cable fabrication		30	min/gage
routing to DAQ		20	min/gage
gage protection		20	min/gage
fill installation time		75	minutes
supplies, fixture	$	25.00	per gage

Estimated Cost

First test			
48	wall accelerometers	$	24,000.00
4	layout	$	400.00
48	wall installation	$	5,600.00
48	supplies	$	480.00
	Total	$	30,480.00
72	fill accelerometers	$	36,000.00
3	layout	$	300.00
72	fill installation	$	9,000.00
72	supplies	$	1,800.00
	Total	$	47,100.00
Total for first test		$	77,580.00
Second and third tests			
54	fill accelerometers	$	27,000.00
3	layout	$	300.00
54	fill installation	$	6,750.00
54	supplies	$	1,350.00
	Total	$	35,400.00
TOTAL for three tests		$	148,380.00

Soil-Structure Interaction
Instrumentation Cost Estimate

Shape Tape

Assumptions

3 sensor tapes per wall oriented top to bottom
Sensors at 1' spacing
No transducers lost during test

2 walls
3 lines/wall
6 sensor tapes
15 gages/line
90 sensors

Estimated Unit Costs

sensor tapes

6 tapes, 15', 15 sensors/tape	$ 10,000.00	per gage

installation

labor cost	$ 100.00	per hour
includes		
gage layout	1	hr/wall
gage installation	30	min/gage
cable connection	5	min/gage
routing to DAQ	30	min/gage
gage protection	20	min/gage
total installation time	85	minutes
supplies	$ 100.00	per gage

Estimated Cost

First test

6	gages	$ 60,000.00
2	layout	$ 200.00
6	installation	$ 850.00
6	supplies	$ 600.00
	Total	$ 61,650.00

TOTAL for three tests $ 61,650.00

Talked to Lee Danish of Measureand on 6/11/09
506-462-9119

Recommended ShapeAccelArray rather than ShapeTape

Recommended SAAR line - research line
gives higher data rate for dynamic measurements

6 tapes, 16' long, 16 sensors/tape	$ 145.00	per sensor
	6	tapes
	16	sensors/tape
	$ 13,920.00	
interface box and 8-channel RS-485	$ 3,000.00	
software - free but many use other for	$ 1,000.00	

$ 18,087.00

Soil-Structure Interaction
Instrumentation Cost Estimate

Tactile Pressure Sensors

Assumptions

16 sensors per wall
Sensors on 4x4 grid
No transducers lost during test

2	walls
16	sensors/wall
32	sensors

Estimated Unit Costs

sensors

2 sensors with system	$ 4,995.00	
additional sensors	$ 795.00	per sensor
8 sensor system	$ 9,765.00	per 8 sensors
16 sensor system	$ 19,530.00	per 16 sensors

installation

labor cost	$ 100.00	per hour
includes		
gage layout	1	hr/wall
gage installation	30	min/gage
cable connection	5	min/gage
routing to DAQ	30	min/gage
gage protection	20	min/gage
total installation time	85	minutes
supplies	$ 25.00	per gage

Estimated Cost

First test

32	sensors	$ 39,060.00
2	layout	$ 200.00
32	installation	$ 4,533.33
32	supplies	$ 800.00
	Total	$ 44,593.33

TOTAL for three tests $ 44,593.33

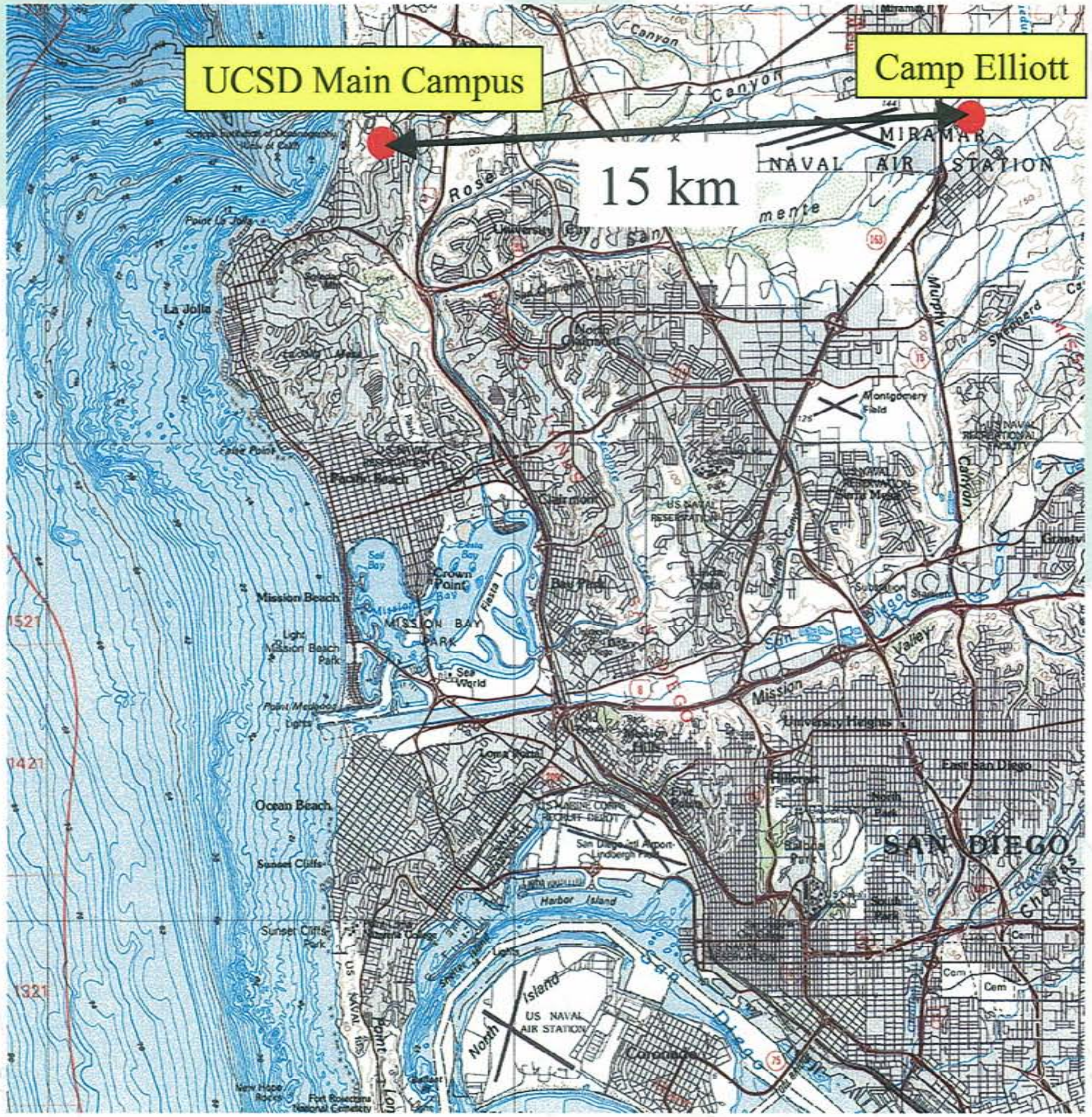
Soil-Structure Interaction
Instrumentation Cost Estimate

- Support Structure
  - Requirements
    - Support displacement transducers at each end
    - Support displacement transducers over top of fill
    - Rigid enough to remain stationary during test

    - Brackets for LVDTs in 3x6 grid at each wall
    - Brackets for laser displacement transducers in 5x6 grid over fill

    - Removable during test reconfiguration

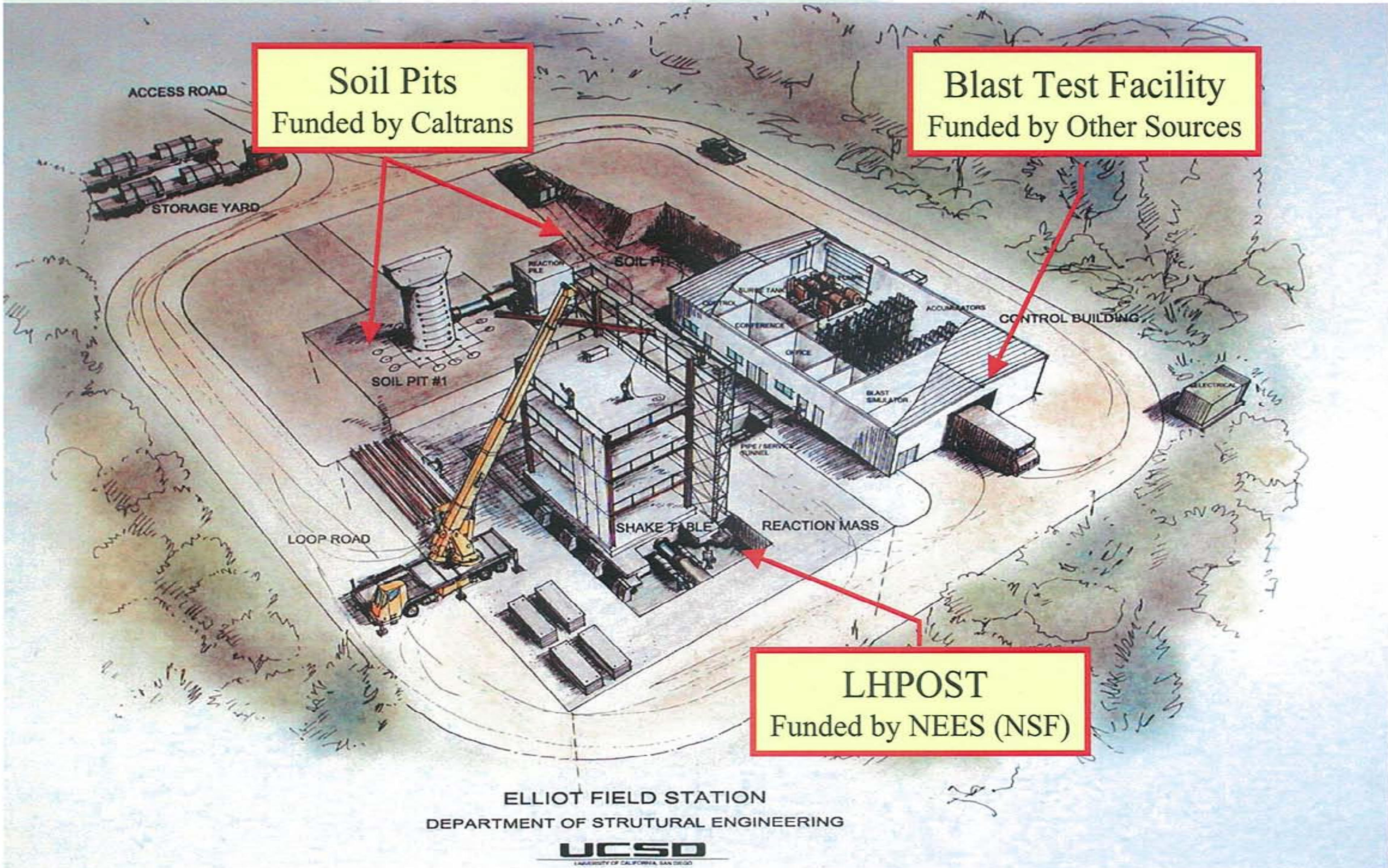
UCSD Large High Performance Outdoor Shake Table (LHPOST)

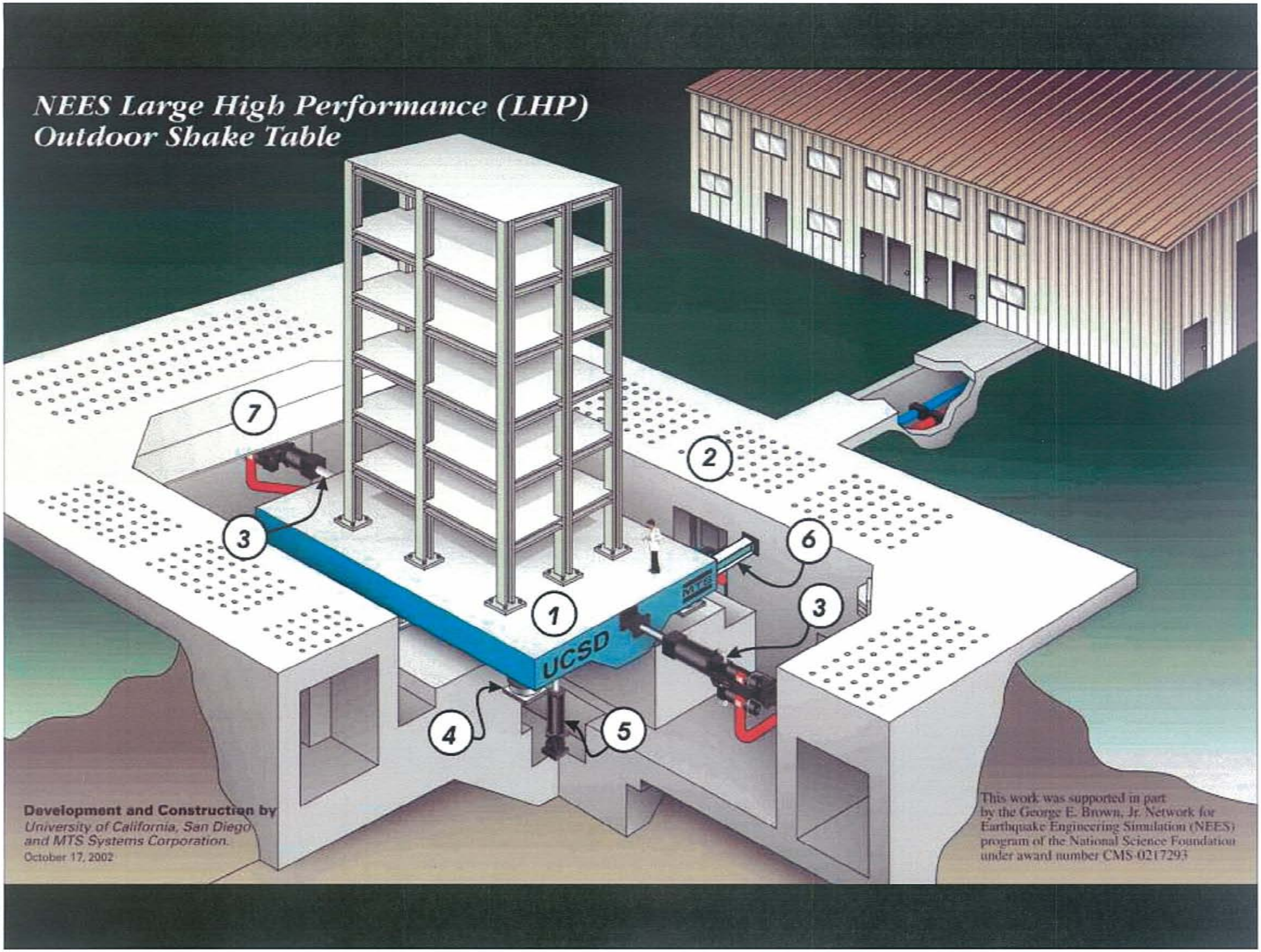


UCSD

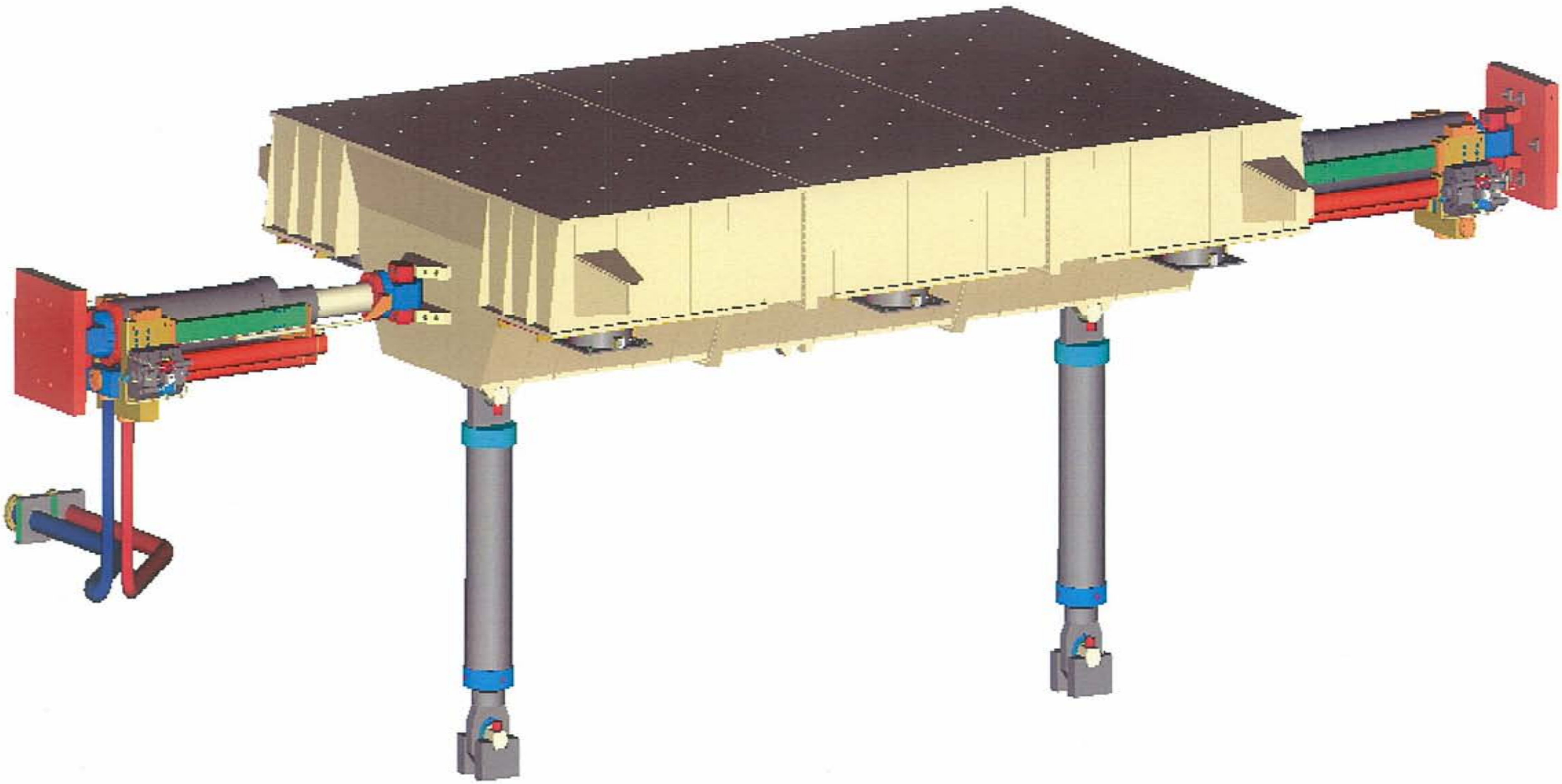
UCSD Large High Performance Outdoor Shake Table (LHPOST)

Camp Elliott Development Project




NEES Large High Performance (LHP)
Outdoor Shake Table
1
2
3
3
4
5
6
7
UCSD
Development and Construction by
University of California, San Diego
and MTS Systems Corporation.
October 17, 2002
This work was supported in part
by the George E. Brown, Jr. Network for
Earthquake Engineering Simulation (NEES)
program of the National Science Foundation
under award number CMS-0217293

Hydrostatic Bearings with Discrete Tension Cylinder Prestressing System



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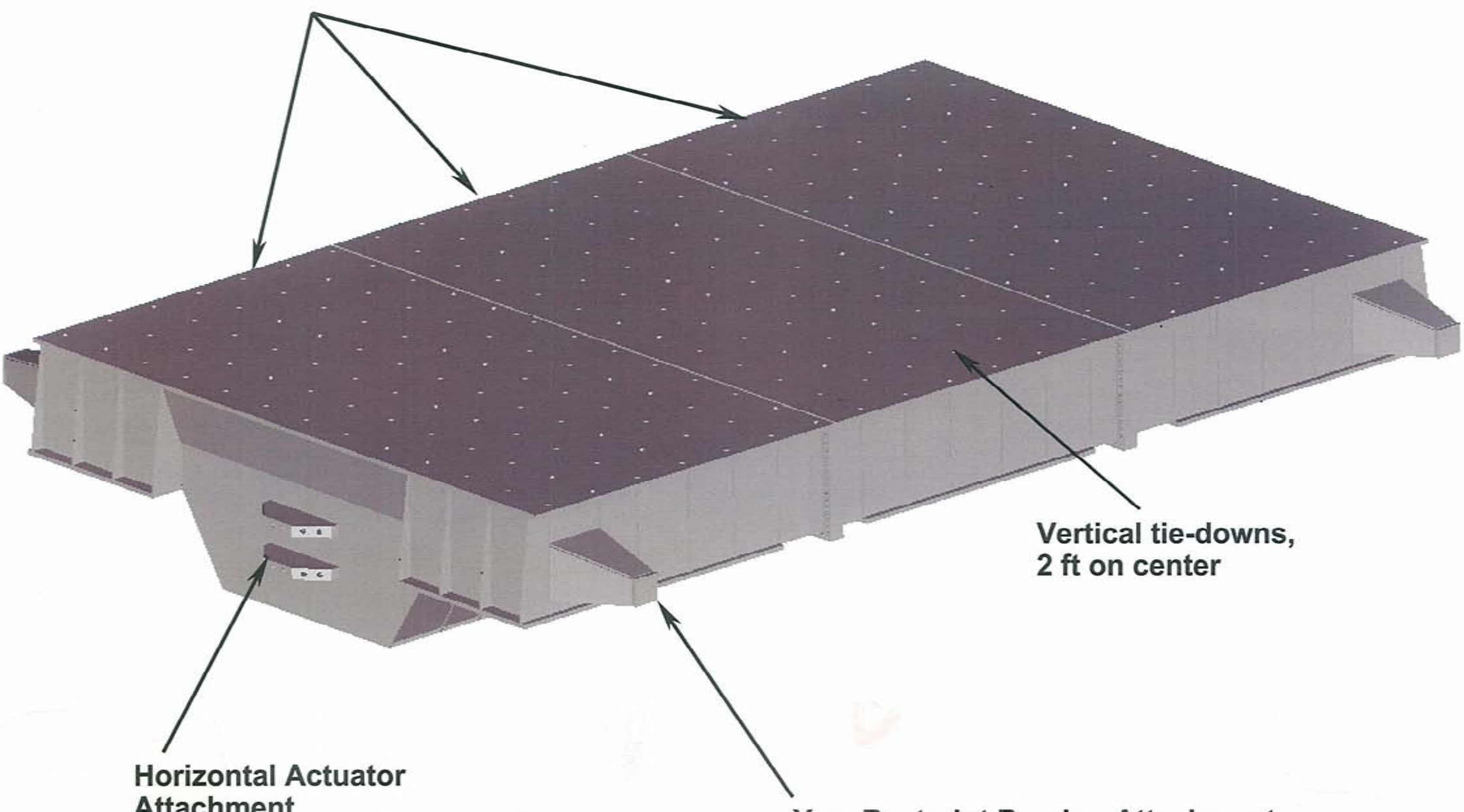
Platen Design & Analysis



Approx. Weight = 150 tons

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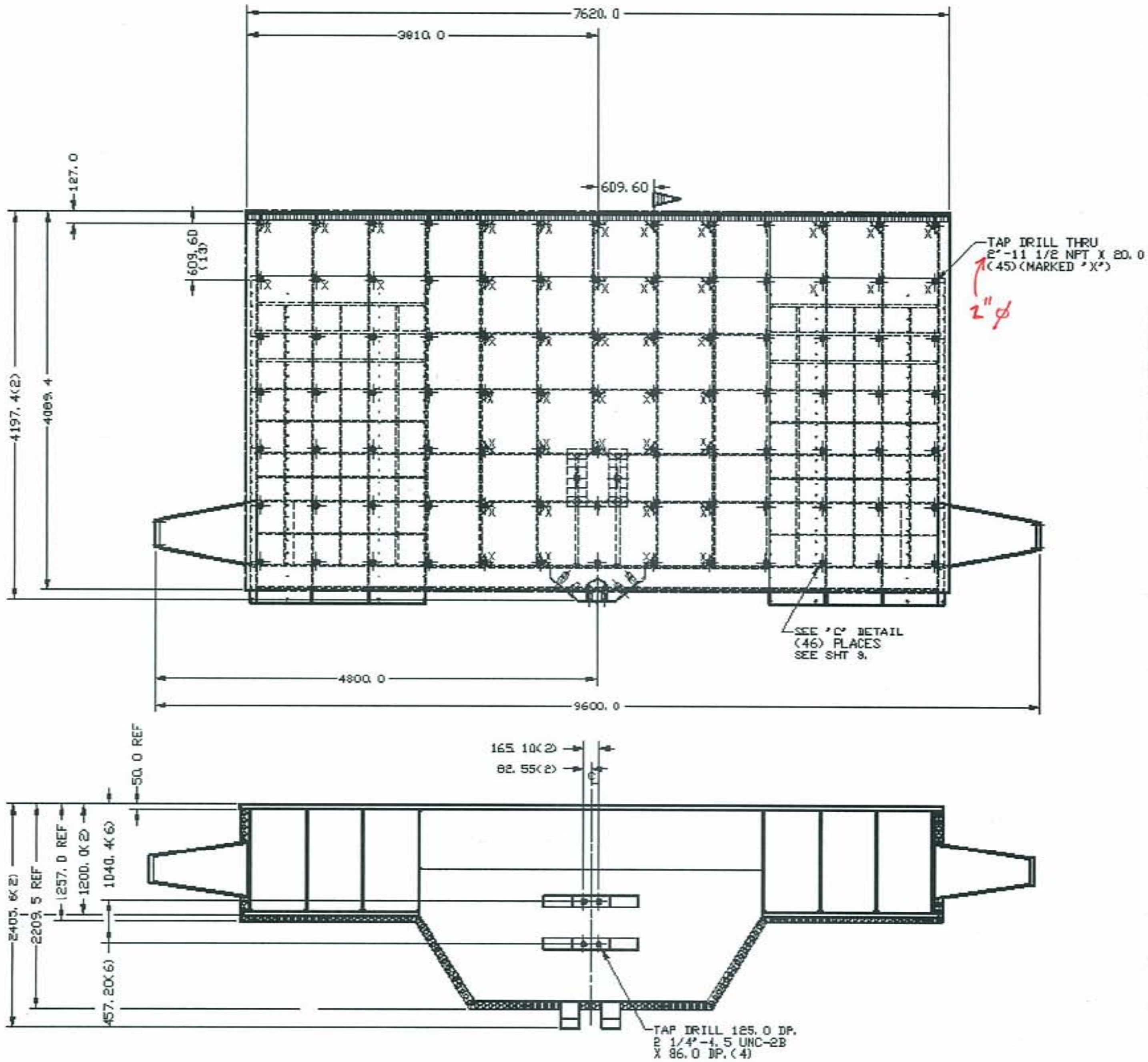
UCSD Large High Performance Outdoor Shake Table (LHPOST)



UCSD

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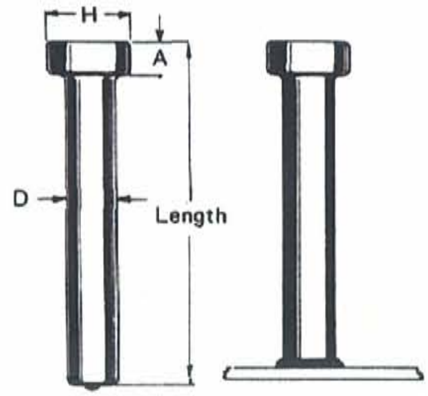
NELSON STUD WELDING

NELSON STUD WELDING

SPECIFICATION: S3L Shear Connectors (SC)

✔ Check Standard Stock

S3L



Nelson headed shear connectors deliver code specified shear strength values as used in composite construction, securing concrete to steel structural components. Nelson shear connectors meet requirements of the following codes and are also USNRC approved:

AWS D1.1 Structural Welding Code – Steel
AWS D1.6 Structural Welding Code – Stainless Steel
AWS D1.5 Bridge Welding Code /
    AASHTO Standard Specification for Highway Bridges
ISO-13918 welding – Studs for arc stud welding
Canadian Standards Association W59 – Welded Steel Construction
International Building Code Section 19
AISC Manual of Steel Construction – Allowable Stress Design
AISC Manual of Steel Construction – Load & Resistance Factor Design

See also: ICBO Evaluation Report ER-2614 Nelson Shear Connectors

Shear connectors are typically used in composite steel construction for holding concrete slabs to steel members to resist shear forces and increase shear loading capacity in steel buildings, bridges, columns caissons, containment liners, etc. They also act as embedment anchors on miscellaneous embedded plates, frames, angles, strip plates, attachments and connections. Options for Welding Through Metal Deck are available for this stud.

For similar function studs, see Nelson H4L Headed Concrete Anchors and D2L Deformed Bar Anchors.

When ordering, specify Type, Diameter, Length, Material, Quantity, and Part Number
***Example:* S3L 3/4 x 6-3/16"; Mild Steel; 10,000 pieces; #101098015**

Stud Diameter	Burn Off	A	H	Chuck	Recommended Standard Accessories: Foot	Ferrule Holder	Ferrule for Flat
3/4" 19 mm	0.187 4mm	0.375	1.250	500001088	502002042	501006027	100101152
7/8" 22mm	0.187 4mm	0.375	1.375	500001091	502002042	501006028	100101140
1" 25mm	0.250 6mm	0.500	1.625	500001424	502002042	501006046	100101045

* Burn Off: Burn off lengths shown are for welding to bare steel. For burn off values and other details when studs are welded through metal deck to steel see **WELD THROUGH DECK SPECIFICATION SHEET**

The **Nelson Ferrule Shooter** is available for semi-automatic dispensing of ceramic ferrules along with standard ferrules assembled 50 pieces per "string" for easy and fast loading onto the Ferrule Shooter dispenser. Following are the current ferrule assemblies available:

Stud diameter	Ferrule
3/4" downhand	100101260
3/4" weld through deck	100101249
7/8" downhand	100101261
7/8" weld through deck	100101262

MATERIALS: Studs are available in Low Carbon Mild Steel and 316L Stainless Steel. For specific grade information and physical and chemical properties, conforming standards, and information on stud plating and heat treating, please see General Material Specifications. Certified Material Test Reports (CMTR) and Certificates of Compliance (COC) are available and must be requested at time of order.
For ferrules and grips used in welding at an angle to plate, welding to angles, and welding to a vertical base plate, see the Special Applications section of the Ferrule Specifications.

FLUX: All Nelson concrete anchors have a solid flux load.

Visit our website **www.NelsonStudWelding.com** for a list of our standard stock products.

Nelson Stud Welding, Inc.
7900 West Ridge Road
Elyria, Ohio 44035

PH 1-800-NEL-WELD
1-800-635-9353

E-mail: Nelson.Sales@NelsonStud.com

NELSON STUD WELDING

General Information for Stud Welding Studs

Material Specifications

Nelson's studs may be made of one of the following materials, as specified on individual specification sheets. Certificates of chemical analysis and physical properties are available, upon request. All physical and chemical properties are independent of stud size or shape.

Mild Steel

Standard mild steel studs manufactured by Nelson conform to ASTM A –108 specifications for 1010 through 1020 mild steels. Physical properties and chemical composition of mild steel Nelson studs are in accordance with AWS D1.1. Special studs can also be manufactured of other weldable mild steels. Heat treatments and plating can be applied to mild steel studs, upon request.

Mild Steel Chemical Composition

Element	Minimum wt%	Maximum wt%
C	0.08	0.23
Mn	0.30	0.90
P	--	0.04
S	--	0.05

Stainless Steel

Standard Nelson studs manufactured of stainless steel conform to ASTM A –276 or A –493 specifications. Studs can be manufactured from other weldable stainless steel alloys. Mechanical properties of Nelson stainless steel studs depend on the cold working or heat treatment applied to the studs after forming. Stainless steel studs can be annealed, upon request.

Stainless Steel Chemical Composition

Element	302HQ (30430) All other studs		316L (31603) Shear and Concrete Anchors	
	Minimum wt%	Maximum wt%	Minimum wt%	Maximum wt%
C	--	0.12	--	0.03
Cr	17.00	20.00	16.00	18.00
Ni	8.00	13.00	10.00	14.00
Mn	--	2.00	--	2.00
Cu	3.00	4.00	--	--

Mechanical Properties - Standard

Minimum Values	Mild Steel Shear and Concrete Anchors	Standard Mild Steel Studs	Mild Steel Deformed Bar Anchors	Stainless Steel Studs, as formed	Stainless Steel Studs, as formed, post-annealed
Yield, 0.2% offset (psi), R_e	51,000	49,000	70,000	50,000	30,000
Ultimate Tensile (psi), R_m	65,000	61,000	80,000	75,000	70,000
% Elongation, A_5, in 2" gage length	20	17	N/A	40	40
% Area Reduction	N/A	N/A	N/A	50	50

Mechanical Properties - Metric

Minimum Values	Mild Steel Shear and Concrete Anchors	Standard Mild Steel Studs	Mild Steel Deformed Bar Anchors	Stainless Steel Studs, as formed	Stainless Steel Studs, as formed, post-annealed
Yield, 0.2% offset (N/MM^2), R_e	350	340	485	345	206
Ultimate Tensile (N/MM^2), R_m	450	420	552	517	483
% Elongation, A_5, in 5 x diameter	20	17	N/A	40	40
% Area Reduction	N/A	N/A	N/A	50	50

NELSON STUD WELDING

General Information for Stud Welding Studs

Standard Arc Welding Studs – Tensile and Torque Strengths

Mild Steel – 61,000psi Minimum Ultimate, 50,000 psi Minimum Yield

Thread Diameter	META[1] (sq. in.)	Yield Load (lbs.) at 50,000 psi	Ultimate Tensile Load (lbs) at 61,000 psi	Yield Torque[2] (ft-lbs) at 50,000 psi	Ultimate Torque (ft-lbs) at 61,000 psi	Shear Strength[3] (75% of Tensile Strength)
10-24 UNC	0.0174	870	1,061	2.7	3.3	796
10-32 UNF	0.0199	1,000	1,220	3.1	3.8	915
1/4-20 UNC	0.0317	1,590	1,940	6.6	8.1	1,455
1/4-28 UNF	0.0362	1,810	2,208	7.5	9.2	1,656
5/16-18 UNC	0.0522	2,620	3,196	13.6	16.6	2,397
5/16-24 UNF	0.0579	2,895	3,532	15.1	18.4	2,649
3/8-16 INC	0.0773	3,875	4,728	24.2	29.5	3,546
3/8-24 UNF	0.0876	4,380	5,344	27.4	33.4	4,008
7/16- 14 UNC	0.1060	5,315	6,484	38.7	47.2	4,863
7/16-20 UNF	0.1185	5,900	7,198	43.0	52.4	5,399
1/2-13 UNC	0.1416	7,095	8,656	59.1	72.1	6,492
1/2-20 UNF	0.1597	8,000	9,760	66.7	81.3	7,320
5/8-11 UNC	0.2256	11,300	13,786	117.7	143.6	10,340
5/8-18 UNF	0.2555	12,750	15,555	132.8	162.0	11,666
3/4-10 INC	0.3340	16,700	20,374	208.8	254.7	15,281
3/4-16 UNF	0.3724	18,600	22,692	232.5	283.7	17,019
7/8-9 UNC	0.4612	23,100	28,182	336.9	411.0	21,137
7/8-14 UNF	0.5088	25,450	31,049	371.1	452.8	23,287
1-8 UNC	0.6051	30,300	36,966	505.0	616.1	27,725
1-14 UNF	0.6791	33,900	41,358	565.0	689.3	31,019

* Torque figures based on assumption that excessive deformation of thread has not taken relationship between torque/tension out of its proportional range.

In actual practice, stud should not be used at its yield load. A factor of safety must be applied. It is generally recommended that studs not be used at more than 60% of yield strength, however, the factor of safety may vary up or down according to the particular application in which the studs are being used.

The user of these studs will make this determination

Formulae used to make the above calculations are as follows:

Ultimate Tensile	$L = SA$	Ultimate Torque	$T = 0.2 \times D \times L$
Yield	$Z = YA$	Yield Torque	$T = 0.2 \times D \times Z$

Where
- $D =$ Nominal Thread Diameter
- $S =$ Tensile Stress (psi)
- $L =$ Tensile Load (lbs)
- $A =$ Mean Effective Thread Area (META)
- $Y =$ Yield Stress (psi)
- $Z =$ Yield Load
- $T =$ Torque (in-lbs)

1 META is used instead of root area in calculating screw lengths because of closer correlation with actual tensile strength. META is based on mean diameter, which is the diameter of an imaginary co-axial cylinder whose surface would pass through the thread profile approximately midway between the minor and pitch diameters.

2 In actual practice, stud should not be used at its yield load. A factor of safety must be applied. It is generally recommended that studs not be used at more than 60% of yield strength, however, the factor of safety may vary up or down according to the particular application in which the studs are being used.

The user will make this safety factor determination

3 Shear values are based on Tensile Strength of the stud.

NELSON
STUD WELDING

General Information for Stud Welding Studs

Approximate Weight of Unthreaded Studs per 1000
(length before welding is used to determine weight)
Weights are in pounds. To convert to kilograms, multiply values below by 0.4536

Length	Diameter								
	3/16	1/4	5/16	3/8	7/16	1/2	5/8	3/4	7/8
3/4	6.0	10.5	16.4	23.5	31.9	41.7	--	--	--
1	8.0	14.0	21.8	31.3	42.5	55.6	86.6	--	--
1-1/4	10.0	17.5	27.3	39.1	53.1	69.5	108.3	156.0	--
1-1/2	12.0	21.0	32.7	47.0	63.8	83.4	129.9	187.2	255.0
1-3/4	14.0	24.5	38.2	54.8	74.4	97.3	151.6	218.4	297.5
2	16.0	28.0	43.6	62.6	85.0	111.2	173.2	249.6	340.0
2-1/4	18.0	31.5	49.1	70.4	95.6	125.1	194.9	280.8	382.5
2-1/2	20.0	35.0	54.5	78.3	106.3	139.0	216.5	312.0	425.0
2-3/4	22.0	38.5	60.0	86.1	116.9	152.9	238.2	343.2	467.5
3	24.0	42.0	65.4	93.9	127.5	166.8	259.8	374.4	510.0
3-1/4	26.0	45.5	70.9	101.7	138.1	180.7	281.5	405.6	552.5
3-1/2	28.0	49.0	76.3	117.4	148.8	194.6	303.1	436.8	595.0
3-3/4	30.0	52.5	81.8	125.2	159.4	208.5	324.8	468.0	637.5
4	32.0	56.0	87.2	125.2	170.0	222.4	346.4	499.2	680.0
4-1/4	34.0	59.5	92.7	133.0	180.6	236.3	368.1	530.4	722.5
4-1/2	36.0	63.0	98.1	140.9	191.3	250.2	389.7	561.6	765.0
4-3/4	38.0	66.5	103.6	148.7	210.9	264.1	411.4	592.8	807.5
5	40.0	70.0	109.0	156.5	212.5	278.0	433.0	624.0	850.0
Each Additional Inch	8.0	14.0	21.8	31.3	42.5	55.6	86.6	124.8	170.0
Ferrule	3.0	3.5	4.0	5.0	6.0	7.5	9.0	27.0	37.0

Approximate Weight of Shear Connectors
(length before welding is used to determine weight)
Weights are in pounds. To convert to kilograms, multiply values below by 0.4536

S3L Shear Connector Description	Small Shear Cartons				
	Weight Per Box, w/o Box	Quantity Per Box	Quantity Per Pallet	Weight Per 1000 Pieces	Net Weight of Pallet
3/4 x 3-3/16	60.9	130	3,510	468	1,643
3/4 x 3-3/8	58.9	120	3,240	488	1,589
3/4 x 3-7/8	60.2	110	2,970	548	1,625
3/4 x 4-3/16	55.5	95	2,565	585	1,499
3/4 x 4-7/8	54.3	80	2,160	678	1,466
3/4 x 5-3/16	56.6	80	2,160	708	1,529
3/4 x 5-3/8	56.3	75	2,025	750	1,519
3/4 x 5-7/8	56.6	70	1,890	794	1,529
3/4 x 6-3/16	49.8	60	1,620	825	1,345
3/4 x 7-3/16	51.9	55	1,485	946	1,403
3/4 x 8-3/16	42.9	40	1,080	1067	1,158
7/8 x 3-11/16	61.3	85	2,295	726	1,656
7/8 x 4-3/16	60.0	75	2,025	811	1,642
7/8 x 5-3/16	58.2	60	1,620	980	1,584
7/8 x 6-3/16	56.6	50	1,350	1153	1,528
7/8 x 7-3/16	52.0	40	1,080	1320	1,426
7/8 x 8-3/16	49.9	35	945	1473	1,391

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Hard Neoprene

Hard Neoprene is a blend of SBR (Styrene Butadiene Rubber), CR (Neoprene) and NBR (Nitrile) rubbers. This sheet product has a durometer range of 65-75 which is considered a harder and denser rubber sheet.
It is a moderately oil and ozone resistant rubber sheet, for use as sealing and in general gasket applications.
This sheet has less elasticity and flexibility than other rubber sheet goods. Hard Neoprene is a good choice for applications which require a resilient and abrasion resistant elastomer. Some recommended applications include bumper pads, hanger seals, and industrial surface protection. Hard Neoprene is available in lineal footage and in sizes as small as thirty-six inch x five feet.

Fabricated Parts

Pricing: All prices below are per linear feet & in USD$. Minimum order on any stocked width is 5 feet in length.

Thickness Inches	Width Inches	5ft to 24ft	25ft to 99ft	100ft to 499ft	500ft or more	Item In Stock
1/16	36	$3.85	$3.21	$2.14	$1.80	Yes
1/8	36	$7.71	$6.42	$4.28	$3.60	Yes
3/16	36	$11.56	$9.63	$6.42	$5.39	Yes
1/4	36	$15.41	$12.84	$8.56	$7.19	Yes
3/8	36	$23.10	$19.25	$12.83	$10.78	Yes
1/2	36	$30.56	$25.47	$16.98	$14.26	Yes
3/4	36	$45.72	$38.10	$25.40	$21.34	Yes
1	36	$61.13	$50.94	$33.96	$28.53	Yes
1/16	48	$5.14	$4.28	$2.85	$2.40	Yes
1/8	48	$10.27	$8.56	$5.71	$4.79	Yes
3/16	48	N/A	N/A	N/A	$7.19	No
1/4	48	$20.55	$17.12	$11.42	$9.59	Yes
3/8	48	$30.80	$25.67	$17.11	$14.37	Yes
1/2	48	$40.75	$33.96	$22.64	$19.02	Yes
3/4	48	$60.96	$50.80	$33.87	$28.45	Yes
1	48	$81.50	$67.92	$45.28	$38.03	Yes

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Conveyor Belt Heavy Duty
Corrugated Rubber
E.P.D.M. Blend
Gray Sheet Rubber
Hard Neoprene Blend
Neoprene Blend
Nitrile Blend
Pure Gum
Red Sheet Rubber
Santoprene New
Silicone Commercial Grade
Silicone Premium Grade
Silicone FDA
Silicone Translucent
Skirtboard
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Description: Hard Neoprene sheet is a black colored rubber which is used where moderate oil, petroleum, ozone and weathering-resistance is needed. It is very popular due to the broad range of applications in which it may be used.

Compound: Blend of SBR (Styrene Butadiene Rubber), CR (Neoprene) and NBR (Nitrile)

C-33

[illegible]	[illegible] rubbers
Color:	Black
Weight:	Approximate weight per square foot: 1/8" weighs 1-1/4 lbs.
Durometer:	65-75
Temperature Range:	-20 F to 170 F
Minimum Tensile:	725 PSI or 5 MPA
Finish:	Smooth
Minimum Elongation:	300%
Gauges:	1/16", 1/8", 3/16", 1/4", 3/8", 1/2", 3/4", 1" (custom gauges up to 2" thick are available upon request)
Widths:	36" (custom widths up to 78" are available upon request)
PSA:	CCT TF-1574 series pressure sensitive adhesive
Roll Length:	25ft. or 50ft. (depending on thickness).
Chemical Resistance:	Excellent resistance to Hydrogen Gas, Natural Gas, Salt/Sea Water, Butanol (primary), Acetic Acids (up to 20%), Ammonium Salts, Mineral Oils, Silicone Oils and Greases, and many more. Moderate oil, petroleum-based solvents and ozone resistance. For Neoprene's compatibility with your specific medium please consult a Rubber-Cal representative.
Applications:	Sound Studios, Construction Sites, Industrial Gasketing, Underlayment, Chemical Resistant Applications, Laboratory Equipment Protection, Marine, Pet Care Flooring.
Flexibility:	This hard durometer (65-75) sheet rubber offers limited pliability and low elasticity.
Custom Cuts:	In addition to hand fabrication, this product can be fabricated using laser, die, and water-jet cut. Please submit your drawings for a price quote.
Availability:	Popular gauges are always in stock.

Rubber-Cal, Inc.
3012 South Croddy Way, Santa Ana, CA 92704.
Tel 800.370.9152 714.772.3000
Fax 714-772-3088

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