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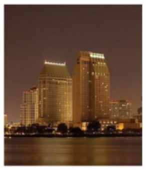

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A REVIEW OF SEISMIC LRFD (LOAD-AND-RESISTANCE FACTOR DESIGN) METHOD FOR MSE (MECHANICALLY STABILIZED EARTH) WALLS

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ABSTRACT

The introduction of AASHTO's LRFD (load-and-resistance factor design) method for the design of MSE (mechanically stabilized earth) walls in 2004 has gradually replaced conventional state-of-the-practice seismic ASD (allowable stress design) method in some states, and by FHWA mandate should completely replace the ASD method by 2010. Limit equilibrium analyses based on Mononabe-Okabe (M-O) pseudo-static method had been the standard method of estimating the seismic external thrust and inertia force for MSE walls. Considering the flexible nature of MSE walls that allow deformation without compromising structural integrity, in the LRFD method, the displacement based pseudo-static method that was developed from Newmark sliding block analyses is used. In this paper, parametric studies are used to highlight the variations of soil reinforcement length/wall height ratios and internal lateral stresses between the LRFD and the current state-of-the-practice ASD methods. The results are compared with referenced past experimental studies and recorded seismic field performance of MSE walls. In addition, results from preliminary dynamic constitutive models are provided for comparison with displacements based on M-O pseudo static method. This paper shows that, by selecting an appropriate amount of tolerable wall deformation (i.e. between 25 and 200 mm as specified in AASHTO and FHWA), the seismic LRFD method for MSE walls is conservative and in general is in agreement with the conventional ASD method that has been widely used in the design of the MSE walls that have performed well during past major seismic events.

INTRODUCTION

Since the invention of modern mechanically stabilized earth (MSE) by Henry Vidal in the 1960s, the concept has been widely used and adapted, with more than 30,000 Reinforced Earth structures having been built worldwide. Although many of these are in high seismic areas, most of the early walls were not designed for seismic conditions. In general MSE walls have performed well during seismic events (Frankenberger et al, 1996, Sankey and Segrestin, 2001).

Seismic design has been previously ignored probably because static design was considered conservative and adequate for most seismic conditions. However, experience in major earthquakes in the last 20 years (Loma Prieta 1989, Northridge 1994, Kobe 1995, and Turkey 1999) has led to additional seismic design considerations for MSE walls. The most widely used seismic design methodology is based on the pseudo-static Mononobe-Okabe method which has gone through many modifications leading to that incorporated in the 2004 AASHTO LRFD Bridge Design Specifications.

Seismic design methods for MSE walls were initially adapted from that for rigid gravity structures. The method was originally introduced in the 1920s by Mononobe and Okabe (Okabe, 1926, Mononobe, 1929, Mononobe and Matsuo, 1929) and is based on Coulomb-Rankine sliding wedge theory; the total seismic active thrust P_{AE} is calculated using force equilibrium of the retained wedge. The original Mononobe-Okabe analysis did not consider inertial forces due to the mass of the retaining wall itself.

Seed and Whitman's (1970) experimental studies showed that the lateral pressure coefficients for cohesionless backfills computed using the Mononobe-Okabe method are in reasonably good agreement with values measured in small-scale model walls. The studies indicated that the dynamic pressure acted at a height varying from 0.5H to 0.67H above the base of the wall. The maximum dynamic active earth pressure, P_{AE}, consists of the initial static pressure, P_A, and the dynamic lateral force component, ΔP_{AE}.

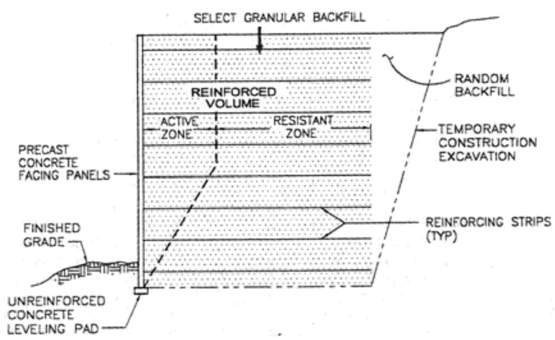


Figure 1. Typical Section of a MSE Structure with Steel Strip Reinforcements.

A concept for seismic analysis of MSE walls was first introduced by Richardson (1978), based on data from a full-scale field test, the stiffness relationships observed in shake-table tests, and the statistical response spectra concept. Richardson found that reinforced earth walls rotated about the base and that the total dynamic lateral forces were inversely proportional to the wall stiffness, reaching a maximum near resonant frequency of the wall.

Prendergast and Ramsey (1980) modified Richardson's (1978) method for design of reinforced earth walls in Wellington, New Zealand and found reasonable agreement with the Mononabe-Okabe approach for a seismic coefficient, k_h of 0.24.

Seed and Mitchell (1981) developed a simplified method for reinforced earth walls that also accounts for wall inertia. The seismic load consists of two components: external dynamic active pressure, P_{AE}, caused by the sliding wedge behind the wall, and the inertia of the wall (EI). Inertial forces are assumed to be horizontal, evenly distributed over the height of the wall, and act on a width of one-half of the wall height.

The displacement based approach used for gravity retaining walls was proposed by Elms and Richards (1990). The model incorporated the Mononobe-Okabe method, the effect of wall inertia, Newmark's sliding block method, and earthquake field records (Franklin and Chang, 1977). The study concluded that the displacement-controlled design approach can be used for design of steel-reinforced MSE walls. It should be noted that displacements in this case primarily occur at the base of the wall due to sliding of MSE mass. The deformation within the MSE mass itself (primarily the upper portion of the wall) is not clearly understood.

OVERVIEW OF AASHTO METHOD

The method described in the AASHTO Standard Specifications for Highway Bridges (1996, 2002) is based on Mononobe-Okabe's pseudo-static approach, modified to account for inertial forces. In this method the following assumptions are made:

1. The wall is free to yield sufficiently to enable full strength or active earth pressure to be mobilized.
2. The backfill in the MSE wall reinforced mass is cohesionless.
3. The backfill is unsaturated, so that liquefaction is not possible.

Seismic events affect both external and internal stability of the MSE walls. The dynamic forces are a function of maximum horizontal acceleration (A_m) as shown in Figure 2. The value of A_m is related to peak horizontal ground acceleration (A) as follows:

$$A_m = (1.45 - A)\,A \quad \text{For the range of } A \leq 0.45 \qquad (1)$$

Values of peak horizontal ground acceleration (A) corresponding with appropriate return periods and geographical region can be found in AASHTO and USGS publications. For external stability, one-half of dynamic horizontal thrust (P_{AE}) is combined with the full inertial force (P_{IR}) in addition to the static thrust, as shown in Figure 2. The values of P_{IR} and P_{AE} for structures with horizontal and sloping backfill are functions of A_m and ΔK_{AE}; where ΔK_{AE} is the dynamic increment of the active earth pressure coefficient.

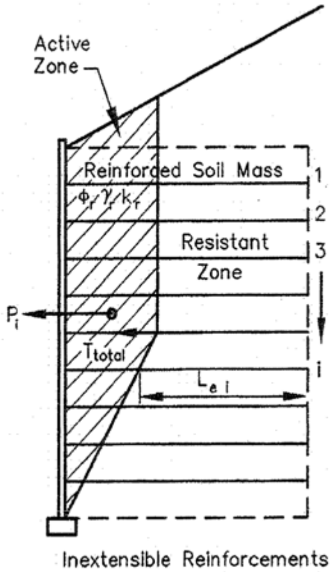
ΔK_{AE} is determined from:

$$\Delta K_{AE} = K_{AE} - K_A \qquad (2)$$

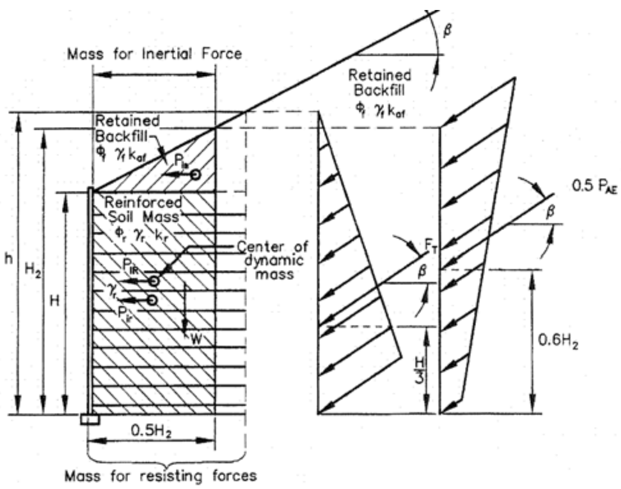
K_A is coefficient of active earth pressure for static conditions and K_{AE} is the total seismic active earth pressure coefficient from the Mononobe-Okabe equation:

$$K_{AE} = \frac{\cos^2(\varphi - \xi - 90 + \theta)}{\cos\xi \cos^2(90-\theta)\cos(I + 90 - \theta + \xi} \left[1 + \sqrt{\frac{\sin(\varphi + I)\sin(\varphi - \xi - I)}{\cos(I + 90 - \theta + \xi)\cos(I - 90 + \theta)}}\right]^2 \qquad (3)$$

I = the backfill slope angle; ξ = arc tan (k_h /1 – k_v); φ = the soil friction angle; and θ = the slope angle of the face.



(a)



(b)

Figure 2. Seismic Stability of MSE Wall; (a) Internal and (b) External.

As mentioned in Richards and Elms (1979), Equation (3) contains a limitation that the term (φ-ξ-I) should be positive for a real mathematical solution. Therefore the friction angle of the soil should not be less than:

$$\varphi \geq I + \xi$$

To reach equilibrium, the maximum horizontal acceleration that can be sustained by the wall structure is limited to:

$$k_h \leq (1 - k_v) \tan(\varphi - I)$$

For these reasons, use of the full value of seismic acceleration in the Mononobe-Okabe method will often produce mathematically impossible solutions for sloping backfill conditions. Because of this condition, displacement based design, in which k_h is reduced, is often preferable.

DISPLACEMENT BASED DESIGN

The use of the full value of A_m in Mononobe-Okabe stability evaluations assumes zero wall displacement, resulting in an overly conservative wall design. MSE walls are known to perform well in large seismic events, with horizontal deformations of more than 100 mm at the top of walls with heights of 10 meters or more without undue stability problems (Sankey and Segrestin, 2001).

To provide a more economical structure, design for a small tolerable displacement is preferable (FHWA 2001; AASHTO 1998; AASHTO 2007). To calculate k_h based on an allowable permanent displacement AASHTO adopted the Newmark sliding block method as modified by Franklin and Chang (1977). The study by Elms and Martin (1979) also suggested that a simplified approach for free standing gravity walls may be based on k_h = 0.5A, provided that displacements up to 250A (in mm) are acceptable.

AASHTO recommends that allowable wall deformations should be considered where A > 0.29 g. In the 2007 LRFD Bridge Design Specifications, AASHTO adopted a simplified version of the Newmark sliding block analysis to modify the horizontal seismic coefficient (k_h) based on inputs of A_m and acceptable horizontal displacement (Kavazanjian, 1997).

The reduced horizontal acceleration coefficient can be computed as follows:

$$K_h = 1.66 A_m \left(\frac{A_m}{d} \right)^{0.25} \qquad (4)$$

where d = lateral wall displacement in mm, for displacement ranging from 25 to 200 mm.

Reducing k_h allows a corresponding reduction in P_{AE} and the resulting driving force for seismic design. Though AASHTO refers to the use of numerical modeling as a supplement to Newmark's sliding analysis, there is no clear recommendation regarding where such modeling may be needed.

OVERVIEW OF ASD AND LRFD METHODS

Current state-of-the-practice for external and internal stability calculations for seismic evaluation of MSE walls uses the Allowable Strength Design (ASD) method described in the

1996 and 2002 edition of AASHTO Standard Specifications, with factors of safety (FS) for external stability against overturning ($FS_{OT} \geq 1.5$) and sliding ($FS_{SL} \geq 1.125$). Internal stability is checked separately by calculating seismic resistance to pullout of reinforcements ($FS_{PO} \geq 1.125$) and tensile strength ($FS_T \geq 1.36$).

The development of the LRFD method in AASHTO (2007) was calibrated against the ASD design method. The fundamental difference between the two methods is that the ASD method simply evaluates the external factors of safety resulting from the computations of driving and resisting forces for the different physical conditions of internal and external stability, while LRFD attempts to apply discrete load and resistance conditions in different combinations to simulate the state of limit equilibrium. The various load and resistance factors in LRFD method act as embedded factors of safety for the structure. The LRFD method applies different factors of safety to various loads based on the level of uncertainty of each individual load or resisting force.

The load and resistance factors for the seismic design of MSE walls given in 2007 AASHTO LRFD are summarized as follows:

Reinforcement Tension & Bearing Pressure:

Vertical Dead Load (MSE Fill) – 1.35
Horizontal Earth Pressure (Backfill) – 1.50
Dynamic Earth Pressure & Force Due to Inertia of MSE Mass (Extreme I) – 1.00
Tensile Resistance (Steel Strips) – 1.00 F_y

Sliding, Overturning & Pullout

Vertical Dead Load Sliding & Overturning – 1.00 Vertical Dead Load for Pullout – 1.00
Horizontal Earth Pressure (Backfill) – 1.50
Dynamic Earth Pressure & Force Due to Inertia of MSE Mass (Extreme I) – 1.00
Resistance Factor for Pullout of Strips (Internal Stability) – 1.20
Base Sliding Resistance (Mass Stability) – 1.00

SEISMIC FIELD PERFORMANCE

Field measurement during earthquakes to validate the current design model is difficult due to the very short duration of such events. There is also very little information regarding soil reinforcement tensions and wall displacements during seismic loading. However, although generally older steel-reinforced MSE walls have not been designed for earthquake loading, they have generally performed well during earthquakes.
For example, the Loma Prieta earthquake in California (1989), with horizontal ground accelerations ranging from 0.5g to 0.6g, affected 20 Reinforced Earth walls ranging from 5 to 10 meters in height, all within 11 to 100 kilometers of the epicenter. Observation by RECo (1990) after the earthquake found no evidence of damage to any of the walls.

The Northridge earthquake in California (1994) affected 23 Reinforced Earth structures within 13 to 83 km from the epicenter; horizontal ground accelerations ranged from 0.46 to 0.66g. Frankenberger, Bloomfield and Anderson (1996) reported that all structures performed well, with only minor damage such as spalling of some panels.

It should be noted that these walls surveyed after the Loma Prieta and Northridge earthquakes were designed using Caltrans method that does not explicitly consider seismic forces. Caltrans method is essentially the same as AASHTO ASD method for static condition, although the Caltrans method uses lower anchorage factor for internal pullout analysis of the soil reinforcement presumably to limit the wall deformation.

The Great Hanshin (Kobe) earthquake in 1995 (ground accelerations > 0.8g) caused widespread damage. 812 reinforced earth structures were affected, although only 124 structures were inspected after the earthquake. The structures ranged from 1.5 to 16.5 m in height with a majority (70%) greater than 5 m in height. Kobayashi, Tabata and Boyd (1996) reported that 114 structures (92%) were undamaged. The remaining 8% showed some panel cracking, opening of the vertical joints, deformation up to 94mm, and tilting movement of less than 2% of the wall height. These structures were located in areas where other types of structures suffered relatively heavy damage. Despite the damage, the structures remained functional and structurally intact. Kobayashi, Tabata and Boyd (1996) also concluded that the conventional pseudo-static design methods and global stability analyses appeared to be conservative.

Otani, Mega and Matsui (1996) also studied MSE walls affected by the Kobe earthquake. They found that none of the walls suffered catastrophic damage and suggested that the design seismic acceleration coefficient of 0.3g corresponds to the estimated maximum horizontal acceleration of about 0.5 to 0.6g measured in the field.

The Izmit earthquake in Turkey (1999), horizontal ground acceleration of 0.4g, affected a reinforced earth structure located almost immediately adjacent to the epicenter. The wall was designed for a ground acceleration of 0.1g. Sankey and Segrestin (2001) reported that the wall sustained only minor damage and remained stable, although the bridge adjacent to the wall collapsed. Pamuk et al. (2004) reported that the worst damage occurred at the top of the wall, where the wall crossed a wide drainage culvert. Localized liquefaction resulting in foundation subsidence appeared to be the significant factor in the damage.

EXPERIMENTAL STUDY

As data from field measurements of MSE walls (i.e. stress-strain, amplification or deamplification of seismic acceleration within MSE mass, etc) are extremely rare, many experimental studies have been performed to simulate seismic behavior. In general such studies involve seismic simulation using low height MSE walls or centrifuge models. Although the use of such experimental data for design still needs to be confirmed with the actual field performance data, the studies are useful in helping to understand general stress-strain behavior of MSE walls during seismic event.

Siddharthan et al (2004) performed centrifuge testing on MSE walls to simulate the 1995 Kobe and 1989 Loma Prieta earthquakes. Six walls with B/H ratios ranging from 0.5 to 0.7, and various soil reinforcements were tested. The walls were 7.3 m (24 ft) high and used metallic soil reinforcements (either bar mats or ribbed strips) to simulate the types of soil reinforcement specified by Caltrans.

The experimental results show that at the end of seismic events with horizontal accelerations of 0.48 g to 0.55 g, the permanent lateral displacements of the walls ranged from 13 mm to 41 mm. The deformation was not uniform, with the middle third of the walls deforming more than at the top and bottom. Siddharthan et al (2004) also observed that in addition to sliding type movement, the walls also experienced some rotation. As expected, that the wall with the smallest B/H ratio (i.e. B/H ratio of 0.5) deformed more than walls with greater B/H ratios.

The measured horizontal acceleration response near the top of the wall within the reinforced mass was found to reach a constant value. This phenomenon is referred to as clipping of acceleration, where acceleration of the moving body remains constant. This is consistent with Newmark's sliding block model adopted by AASHTO in the formulation of deformation-based pseudo-static design. The study also found that deamplification of acceleration occurred within the MSE mass for acceleration levels higher than 0.4g. The reduction of acceleration from the base to the wall to ground surface can be as much as two thirds. This is broadly consistent with field observations indicating deamplication (Otani, Mega and Matsui, 1996).

PARAMETRIC STUDY RESULTS

In order to illustrate the difference between the results of external stability analyses for the ASD and LRFD methods, a series of parametric studies were performed to examine the influence of wall height, reinforcement length, acceleration coefficient, displacement, top of wall geometry. For the purposes of simplicity, only MSE walls using discrete steel reinforcing strips were analyzed.

The results show that there is little variation in the reinforcement length/wall height ratio (B/H) for various MSE wall heights for acceleration coefficients in the range 0.3g and 0.45 g (Figure 3).

Both methods give almost the same B/H ratio for a seismic acceleration of 0.3g which agrees with the minimum value of 0.7 specified in AASHTO. For higher accelerations, the LRFD method using a displacement based acceleration coefficient reduction resulted in more conservative (i.e. higher B/H ratio) designs than the ASD practice of reducing acceleration by half to account for some displacement.

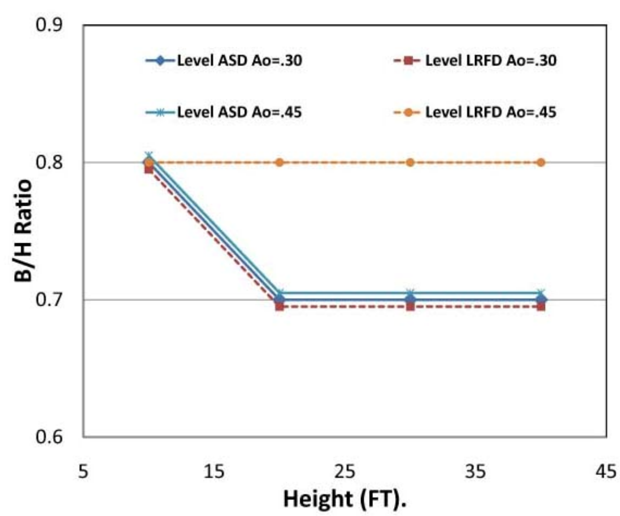


Figure 3. Effect of Wall Heights on B/H Ratio (level surcharge)

The angle of the sloping face above the top of wall had very little effect on B/H ratios from the LRFD and ASD methods (Figure 4). At A=0.30, there is no difference between the B/H ratios for the two methods. For A=0.45, the LRFD method results in B/H ratios about 13% smaller than those from the ASD method. For sloping backfill of 2H:1V, both methods give identical B/H ratios.

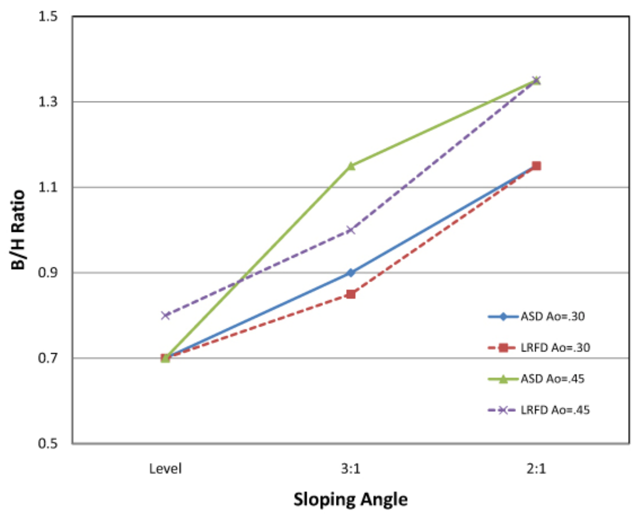


Figure 4. Effect of Top of Wall Geometry on B/H Ratio

When displacement considerations are included, the use of the simplified Newmark sliding block method in AASHTO results in more pronounced variations in B/H. Figure 5 shows that the greater the allowable displacement used in the LRFD design method, the smaller the B/H ratio compared with the ASD method. It is important to note that an allowable displacement greater than 51 mm (2.0 inches) will have very little effect in reducing B/H ratio, as shown in Figure 5.

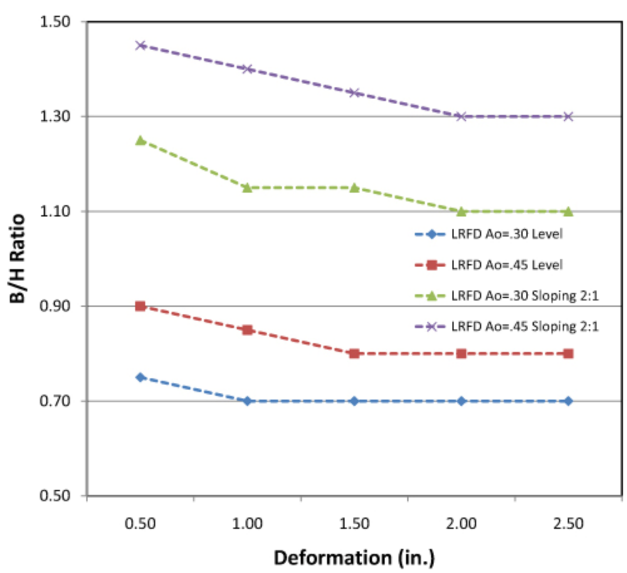


Figure 5. Effect of Wall Displacement on B/H Ratio

In Figure 6, the ASD seismic thrust is shown as a lower limit for the B/H ratio versus seismic coefficient under the ASD method. Output from the LRFD method using variations in displacement shows that the boundary indicated by the ASD method is effectively the lower limit of the LRFD output for displacements between 13 mm (0.5 in) and 64 mm (2.5 in). It is noted that the displacement outputs correspond to the base of the MSE wall and deformations at the upper wall heights will be greater. Furthermore, the lower boundary for the ASD method is more consistent with current design methodology that has proven effective in seismic design of MSE walls; this means the outcome of the LRFD results using displacement inputs is a conservative means for evaluating external stability.

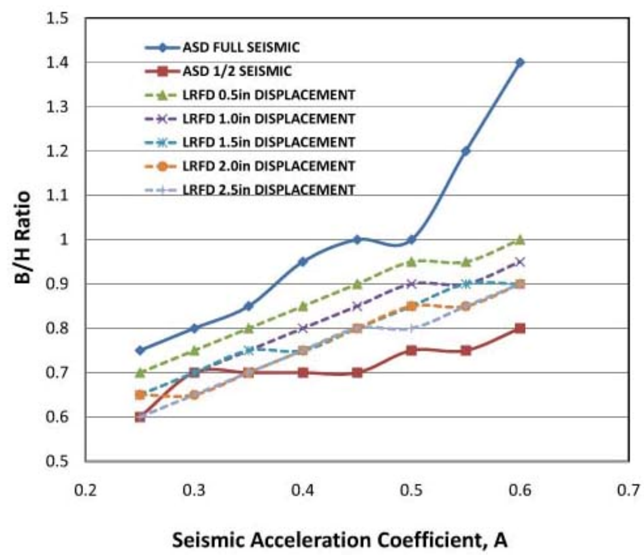


Figure 6. External Stability Comparison ASD vs LRFD (H = 6 m, level surcharge)

For the internal design for MSE walls, evaluations were performed to ensure that the tensile strength and pullout capacity of soil reinforcement are adequate for the imposed lateral dynamic force. The cross-sectional area of steel reinforcement was used to determine the capacity of soil reinforcement, though the cross section of the reinforcement was deemed important in determining tensile strength and the surface contact area of steel reinforcement and soil was important to pullout capacity.

The parametric study was normalized to look at the steel cross-sectional area in either ASD or LRFD methods by evaluating the needed resistance over a tributary area of wall face. Not surprisingly the parametric results show that variations in design outcome for reinforcement between the ASD and LRFD methods are small. Figure 7 shows very little difference in normalized lateral stresses for the two methods for walls having various sloping surcharge conditions. For A of 0.3 and 0.5, and allowable displacement of 1.5 inches, the

LRFD methods result in slightly higher internal lateral stress in all cases.

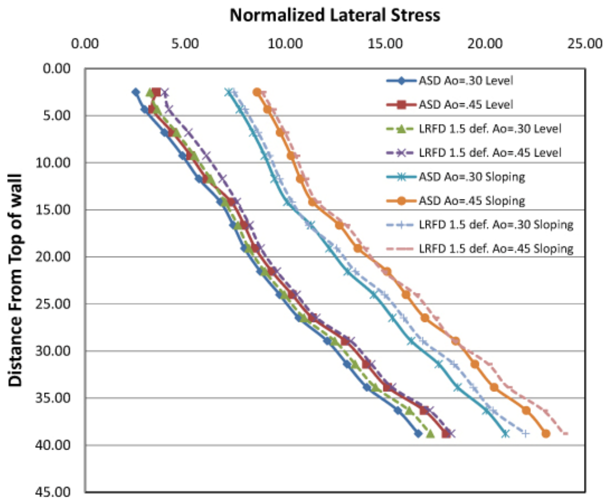


Figure 7. Effect of Sloping Surcharge on Internal Lateral Stresses

CONCLUSIONS

It is recognized that the MSE design method described in AASHTO LRFD is an empirical adaptation of current state-of-the practice ASD procedures. The results of the parametric study show that the design outcome from displacement-based LRFD method using load and resistance factors specified in AASHTO does not vary substantially from the ASD method; provided that the appropriate amount of deformation is selected.

AASHTO ASD design method has historically been used in the seismic design of steel-reinforced MSE walls. Such walls have performed well during earthquakes. Realizing that the results of AASHTO LRFD and ASD methods in the external and internal design of MSE walls are very similar, it can be concluded that the new AASHTO LRFD method is conservative.

Evaluations of field performance of MSE walls as well as experimental studies have also suggested that due to the flexible nature of MSE walls, the seismic horizontal ground accelerations correspond to lower design accelerations in pseudo-static model, suggesting that the current AASHTO pseudo-static approach is conservative.

Additional studies are recommended to survey MSE walls in locations of recorded earthquake events to further understand deformation behavior of MSE walls during seismic and to establish a baseline correlation between displacements and deformation of walls, seismic accelerations and wall geometries. It is anticipated that there is a critical B/H ratio that can be developed to further discern the flexibility of MSE walls with increasing heights. It is anticipated that with development of baseline correlations between wall displacements and corresponding seismic events will come better input and reliability to numerical and practical pseudo-static models of MSE walls.

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