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APPLICATION OF DISPLACEMENT-BASED SEISMIC DESIGN APPROACH FOR MSE WALLS WITH UNEVEN REINFORCEMENT

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ABSTRACT

Mechanically Stabilized Earth (MSE) walls have been used increasingly as retaining systems since the late 1960s. Inclusion of reinforcement within soil has made it possible to construct steep slopes and embankments. The paper describes the application of a numerical model to evaluate permanent lateral seismic displacement of MSE walls with variable lengths of reinforcement. The numerical model was developed by extending an existing model that was proposed for MSE walls with uniform reinforcement. The approach accounted for the vertical variation in acceleration within the backfill. The predictive capability of the numerical model was verified using centrifuge tests carried out in the large centrifuge at the National Geotechnical Centrifuge at the University of California, Davis.

One of the important observations from this study was the beneficial role of longer reinforcements near the surface in reducing permanent wall movement. The proposed numerical model captures many aspects of the characteristic deformation behavior of MSE walls observed in the centrifuge tests. Subsequently, the analytical model was used to assemble a database of lateral permanent wall displacement of MSE walls of height varying between 5 and 10m subjected to base excitations from M6.5 and M8 earthquake events. Important design information such as the required length of reinforcement needed to keep the wall displacement within a certain specified limit can be interpreted from the database.

Introduction

One of the most cost effective earth retaining structures used in transportation applications around the United States is the mechanically stabilized earth wall system, which are

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commonly referred to as MSE walls. These wall systems are comprised of a wall facing, typically concrete, that is oriented in a near vertical to vertical direction. Behind the facing exists the MSE backfill, which has reinforcement inclusions. Such walls are typically found in tight intersections where room for open slopes is not available. The soil reinforcements provide tensile resistance which is an ability that the soil does not have. These soil reinforcements and their interaction with the soil contribute to the stability of the wall.

Huang and Tatsuoka (2001) provided a summary of MSE wall behavior during the 1999 Chi-Chi earthquale (M7.6). They reported damage to three MSE walls located within 40 km of the epicenter. Though these walls were short, measuring less than 3.2 m in height, one of them completely collapsed. The sites at these walls experienced a maximum acceleration, a_{max} in excess of 0.44g. Damage to a 100 m-long MSE wall system (Arifiye Overpass) located in the city of Adapazari, Turkey during the1999 Kocaeli Earthquake (M7.4) was significant (Erdik 2004). The largest acceleration recorded at a nearby station (4 km away) was about 0.41g. The wall height, H varied to as much as 10 m and a section of the wall with H = 10 m displaced laterally by more than 25 cm. More recently, Gazetas *et al*. (2004) also reported on similar seismic behavior relative to other recent earthquakes. The dominant failure mode in all MSE wall cases was permanent lateral deformations, with peak values occurring either near the top or mid-section of the wall. Though conventional MSE walls have been built with uniform reinforcements spanning the entire height of the wall, centrifuge tests have revealed that longer reinforcements near the soil surface can substantially reduce the permanent wall movement (Siddharthan *et al*. 2004a). The paper briefly describes a numerical model to evaluate permanent seismic displacement of MSE walls with variable lengths of reinforcement. The model was developed by extending an existing model that was proposed for MSE walls with uniform reinforcement (Siddharthan *et al*. 2004a, b). The predictive capability of the numerical model was verified using centrifuge tests carried out at the large centrifuge at the National Geotechnical Centrifuge at the University of California, Davis (Siddharthan and Vishnan 2008).

Subsequently, the analytical model was used to analyze typical field MSE walls of height varying between 5 and 10m and the walls were subjected to a variety of excitations from M6.5 and M8 earthquake events. A simple readily usable design procedure that attempts to limit the permanent seismic wall displacement can be developed from the database of permanent displacements presented in the paper.

Summary of Centrifuge Testing and Results

Model Description and Testing

In the centrifuge study reported by Siddharthan *et al*. (2004a) three test models were constructed and tested on the large centrifuge at UC Davis. In each test model two wall configurations were constructed so that the walls were back to back (Fig. 1). Among a total of six such walls tested, one wall had uneven reinforcement. Since analysis of the MSE wall behavior with uniform reinforcements has been addressed in detail by Siddharthan *et al*. (2004a, b), only the test results relative to the wall with uneven reinforcement is presented here. The models tested in this study represented 7.32 m (24 ft) high walls in prototype, supporting a dry granular backfill. The centrifuge acceleration (scale factor) used in these tests was N = 24. Correspondingly, the model wall height was 1 ft. It was decided that a bar mat system much like the ones widely used by Caltrans would be modeled. The reinforcement was modeled by using commercially available galvanized steel wire mesh.

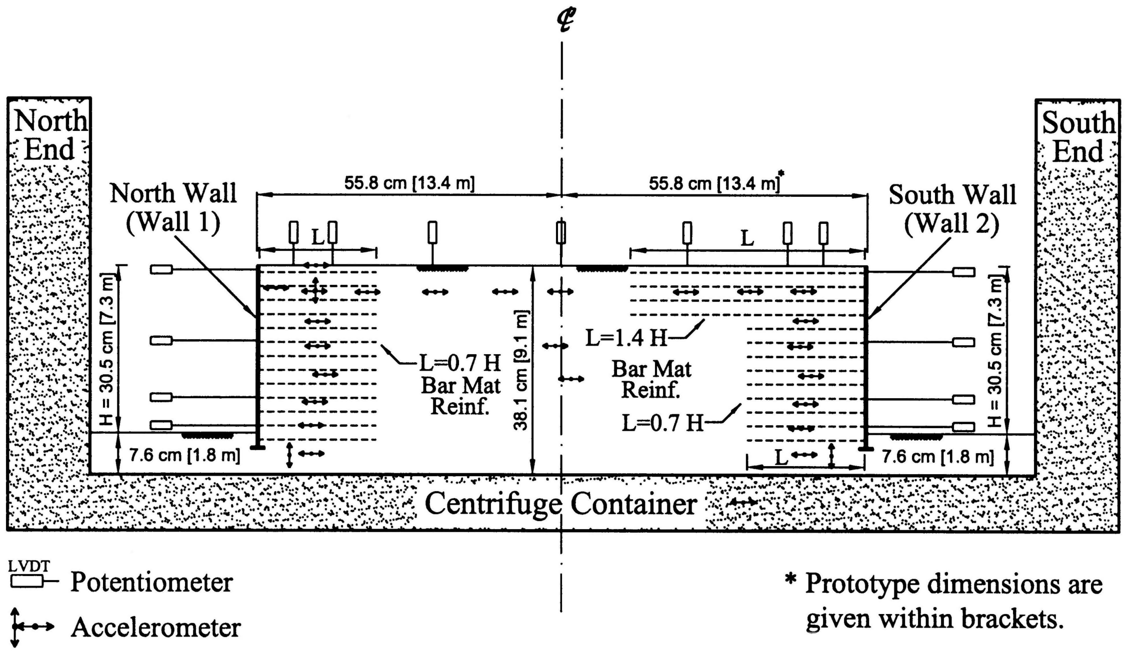

Figure 1. Centrifuge testing of MSE walls with uniform (Wall 1) and uneven reinforcements (Wall 2).

In each of the models tested the backfill consisted of fine dry Nevada sand at an initial relative density (D_r) of 65%. The sand was placed by pluviating from the large pluviater at the UC Davis National Geotechnical Centrifuge center. More details on the model description can be found in Howard *et al*. (1998) and Ganeshwara (1998). The walls tested in test series MSE-01 have uniform (Wall 1) and uneven reinforcements (Wall 2) as shown in Fig. 1. The Wall 1 had a reinforcement length (L) of 0.7 times the height (H), while the Wall 2's L varied from 1.4H near the surface to 0.7H at the bottom.

Once the model was spun up to attain the required gravity level of 24g, several types of base motions, including step waves, cosine sweeps, and recorded earthquake motions were used in the testing program. The typical order of shakes started with small amplitude step wave to verify that the accelerometers were functioning correctly. Subsequently, a base excitation history (1995 Hyogogen-Nanbu/Kobe excitation) with strength, a_{max} of as much as 0.9g was used. In MSE-01 series, the number of strong excitations with a_{max} more than 0.3g was three. Only under these higher excitations the walls showed appreciable permanent displacements.

Analytical Model for Wall Deformation

Based on the observed behavior from the centrifuge tests a failure mechanism described below has been selected to study the deformation behavior of MSE walls with uneven reinforcements. The proposed failure mechanism consists of three rigid blocks (Fig. 2); two top blocks that are rectangular and a bottom block that is triangular. The length of the blocks varies depending on the length of the reinforcements. The interface between any two blocks is horizontal. The bottom block (Block III) is assumed to have both rotational (θ_3) and translational

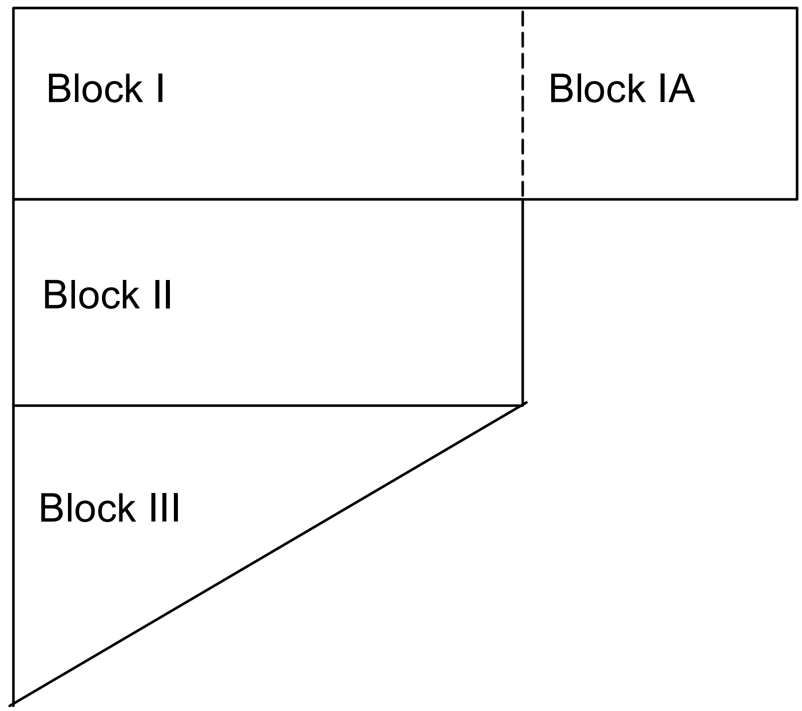

Figure 2. Proposed three block model for permanent lateral wall displacement.

(x_3) degrees of freedom. In order to keep the total degrees of freedom to a minimum, the top two blocks were assigned only translational degrees of freedom (x_1 and x_2). This leads to a total of four degrees of freedom for the failure mechanism. The bottom failure plane associated with Block III is a slope, which starts at the top of the bottom most reinforcement. The lengths of the top blocks equal the lengths of the reinforcement in these blocks. The top block is divided into two parts: Block I and Block IA; but in the deformation analysis both of these blocks are assumed to move as one block, and there is no vertical interface separation between Block I and Block IA. The subdivision of the top block is only necessary when uneven reinforcement is considered. It should be noted that since the face panels are rigid, the interface between blocks, which are horizontal, cannot be arbitrary. Therefore, the interface between failure blocks should coincide with the interface between face panels. An iterative approach has been developed for computational purposes. The slope of the failure plane in Block III and the horizontal interface between the top two blocks are variable, and the wall deformation at the end of a given excitation is computed for each trial combinations. The combination that gives the largest permanent wall face deformation is considered to be the correct failure mechanism.

The failure mechanism adopted above is based on the observed behavior of many walls in centrifuge tests. Since the bar mat reinforcement may be considered to be inextensible, its flexibility is not included in the model. The focus is on overall lateral permanent wall deformation, which is an external stability issue. Local failure modes such as pull-out of individual reinforcement and failure at the face panel-reinforcement connections, which are issues of internal stability, are not addressed.

The dynamic equilibrium equations for horizontal and vertical directions for all the blocks and rotation of Block III about the base were assembled to arrive at four equations of motion in terms of the unknowns x_1, x_2, x_3, and θ_3. These equations are coupled and are ordinary differential equations of second order. A step by step procedure in time domain, similar to the one described by Siddharthan *et al*. (2004b) has been used to obtain the solutions (x_1, x_2, x_3, and θ_3). The acceleration can vary in the backfill due to amplification (or deamplification). The seismic lateral forces on the blocks which are affected by the backfill acceleration have been computed using the appropriate acceleration (average) at the level of the blocks. A computer program DIS-MSE2, has been developed to compute the block deformations for many failure mechanisms by varying the slope angle of Block III and the location of separation between Blocks I and II. Integration in the time domain was carried out using Newmark's scheme of constant average acceleration (Vishnan 2004).

Centrifuge Deformation Response and Validation

Figure 3 shows the measured lateral displacements near the middle of the Walls 1 and 2 in the test series MSE-01, when the model wall subjected to the cosine sweep base excitation from event B. This excitation had an a_{max} of 0.17g with constant amplitude and period (0.4s). Figure 3 shows that the lateral displacement at any time consists of a permanent and a cyclic component. At the end of the excitation, there was only permanent displacement and no cyclic component of displacement.

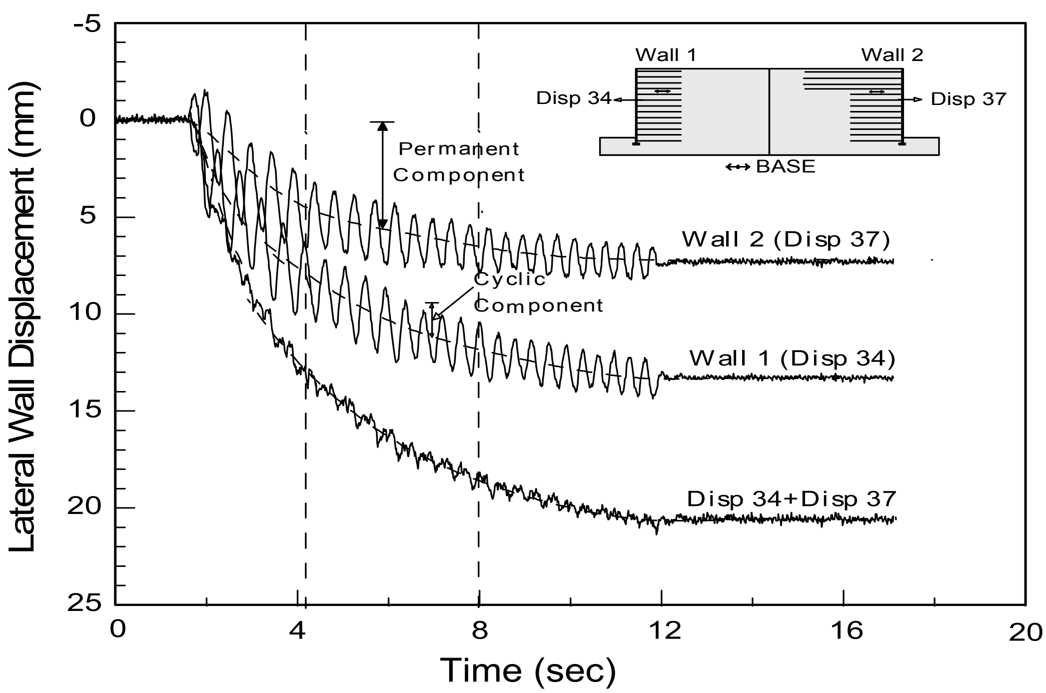

Figure 3. Measured lateral response in event B (a_{max} = 0.17g).

The permanent component represents the progressive movement of the wall away from the backfill whereas the cyclic components represent the instantaneous response to the base excitation. The vertical arrows drawn on the figures facilitate the comparison of the cyclic component and permanent component of displacements at the middle of Walls 1 and 2. This figure also provides the summation of both wall displacements. Unlike the individual displacement which had significant oscillations, the summation plot displays negligible cyclic component. Such behavior leads to the conclusions that (1) the permanent displacement of the wall stretches laterally in a progressive manner away from the backfill with negligible cyclic components; (2) the cyclic components are indeed a result of the entire test model moving laterally in response to inertia forces generated by the shaking; and (3) wall movement into the backfill (i.e., passive) is negligibly small.

As typical plots, the computed and measured wall displacements of Wall 2 (uneven reinforcement) during events O (a_{max} = 0.7g) and P (a_{max} = 0.83g) are shown in Figs. 4a and 4b, respectively. Different values for backfill friction angle φ were considered to get a match between the experimental and analytical results. Computed displacements during the event O for φ values of 36^o and 38^o bracket the measured displacement. Similarly, in the case of event P, the corresponding φ range is 38^o and 40^o. Lower value of φ range for fit in the event O than the corresponding φ range in event P is consistent with the relative density increases that do occur with back-to-back base shaking. The comparison between the computed and measured wall displacements is very good, thus validating the applicability of the proposed analytical model. It should be noted that since only one wall with uneven reinforcement was tested in the centrifuge, caution should be exercised in extending the findings of this study to other cases.

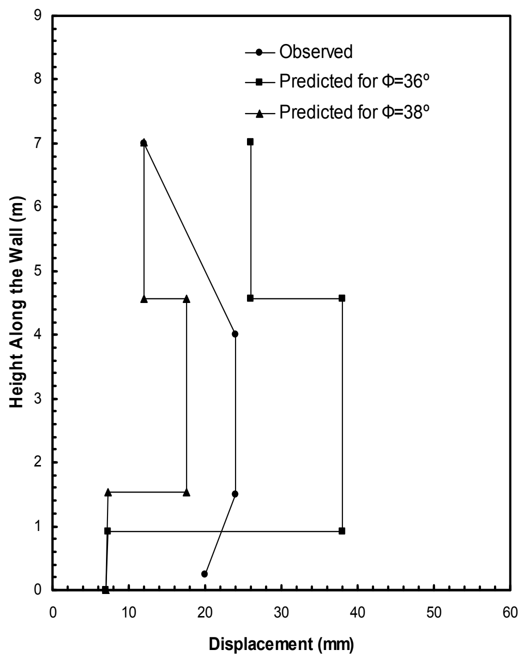

Figure 4a. Lateral wall displacement in event O (a_{max} = 0.7g).

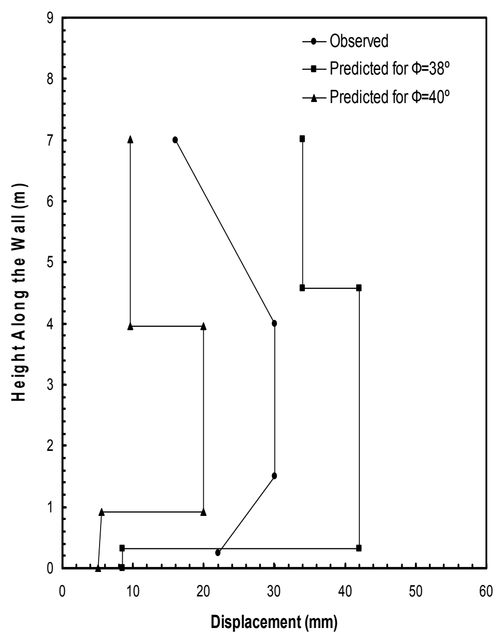

Figure 4b. Lateral wall displacement in event P (a_{max} = 0.83g).

Application of the Proposed Model to Typical Field Cases

The proposed model can be used to predict the lateral permanent deformation of any MSE wall. To analyze any MSE wall, the characteristics of the wall (dimensions of the wall, reinforcement type and its length, etc.), properties of the backfill soil, and the time history of the base excitation are needed. In addition, the variation of acceleration amplification in the backfill is also required. The program DESRA was used to calculate the variation of amplification in the backfill.

As an illustration of the application of the proposed deformation model, permanent seismic deformation of three typical MSE walls (designated as Wall A, Wall B, and Wall C) of different height and different lengths reinforcement computed using modified DIS-MSE2 are presented below. The heights (H) of Walls A, B, and C used in the study are 4.9 m (16 ft), 7.3 m (24 ft), and 9.8 m (32 ft), respectively. All three walls have been assumed to be reinforced with bar mats having uniform and variable lengths of reinforcement (L). When variable reinforcement lengths were used, the lengths varied from 0.7H to 1.7H. The backfill soil considered is assumed to have a friction angle φ of 38^{o} and a unit weight of 18.85 kN/m^3.

As a first step, factors of safety for static stability were determined for the walls using the methods described in FHWA design manual (Elias *et al*. 2001). Static analyses showed that the external stability for all three walls with the lowest reinforcement length of L = 0.7H (uniform reinforcement) was satisfactory. The calculated factors of safety are above the recommended requirements. The walls were checked also for internal static stability. Factors of safety against rupture of reinforcement and pullout of reinforcement were calculated at each reinforcement level. The internal static stability checks for Walls A, B, and C also satisfy the FHWA recommended requirements. Therefore, it is concluded here that all three walls are stable under the static condition.

To analyze the permanent seismic displacement of these field walls, the acceleration amplification factors in the free-field have to be determined. The variation of acceleration amplification in the backfill was obtained using the program DESRA. To analyze Wall A (H = 4.88 m), a backfill height (H_s) of 6 m was considered. Similarly for Wall B (H = 7.31 m) and Wall C (H = 9.76 m), the heights of backfill were 9 m and 12 m, respectively.

The program DIS-MSE2 was used to generate a database of lateral permanent wall deformations under two different excitations. The two base motions that are representative of magnitudes M = 6.5 and M = 8 were used in the study. For M = 6.5, a recording from 1983 Coalinga earthquake (M = 6.5), and for M = 8.0, a recording from 1999 Chi Chi event were initially selected. Each of these records were spectrally matched to a target spectrum using the program RASCAL. Both of these records had a recorded maximum acceleration of about 0.6g and are designated as HPVY045 (M = 6.5) and TCU065 (M = 8) in the database maintained by the Pacific Earthquake Engineering Research (PEER) Center. The target spectra for M = 6.5 and M = 8.0 were selected based on Applied Technology Council (ATC-32) recommendation for Type D conditions. Following spectral matching, baseline correction and filtering were performed. Cut-off frequencies for the M = 6.5 record were 0.1 and 20 Hz, whereas for M = 8.0, the corresponding values were 0.15 and 20Hz. A slightly higher lower cut-off frequency was needed for M = 8.0 excitation to achieve a satisfactory baseline correction. Figure 5 shows the target ATC-32 spectra and the spectra of the selected excitations (damping 5%) for both earthquake magnitudes. The spectral matches have been very good. All motions were scaled to yield an a_{max} of 0.3g, 0.45g, 0.6g and 0.75g, and were applied at the bottom of the MSE backfill.

The lateral wall displacements computed for all three walls under the scaled excitations from M6.5 and M8 earthquakes are shown in Fig. 6. The cases reported are: (1) uniform reinforcement with L = 0.7H (Figs. 6a and 6b); (2) longer reinforcement of L = 1.0H within the top H/3 depth (Figs. 6c and 6d); and (3) longer reinforcement of L = 1.0H within the top 2H/3 depth (Figs. 6e and 6f). The expected trend of higher wall displacement with the increases in wall height, excitation strength and earthquake magnitude are readily seen from the figures. The effectiveness of providing longer reinforcement in reducing wall displacement is also evident

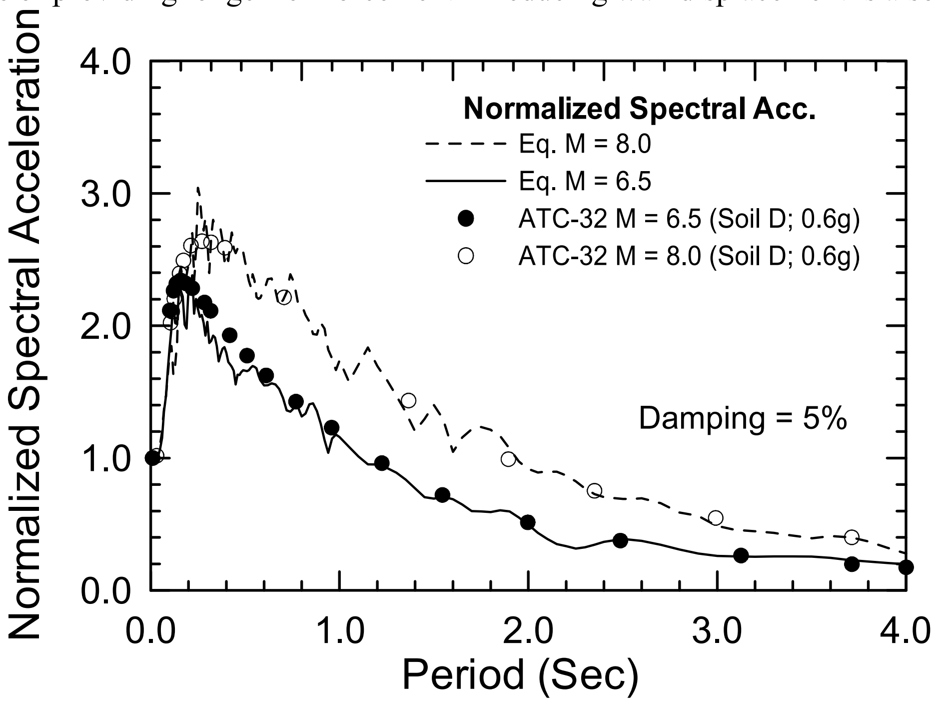

Figure 5. Target (ATC-32) and Spectrally-Matched Response Spectra for Excitations with M = 6.5 and 8.0.

when results given in Fig. 6a (uniform reinforcement) is compared with those given in Figs. 6c and 6e for M6.5 event. Similarly, for M8 earthquake the Fig. 6b should be compared with Figs. 6d and 6f.

Figures 7a and 7b show lateral wall displacement as a function of reinforcement length provided within the top H/3. It may be noted that L = 0.7H is the case of uniform reinforcement. Both earthquake excitations were scaled to 0.6g. Under these excitations, most reduction in wall displacement occurs when the length increased from 0.7H to 1.0H. Beyond this reinforcement length of L = 1.0H, further reduction is not substantial.

Often an important design information is the level of reinforcement required to limit the wall displacement within a specified amount. Such information is useful when a performance-based design approach is undertaken. The series of plots given in Figs. 6 and 7 are useful in estimating the required reinforcement lengths. For example, if the wall displacement is to be limited within 25 mm, the reinforcement length L = 1.0H provided within the top H/3 is adequate for the M6.5 event with a_{max} = 0.6g. On the other hand, for the M8 event with a_{max} = 0.6g, provision of longer reinforcement within the top H/3 for higher walls (H > 7.3m) is not satisfactory. In such cases, the depth of longer reinforcements need to be extended to deeper than H/3.

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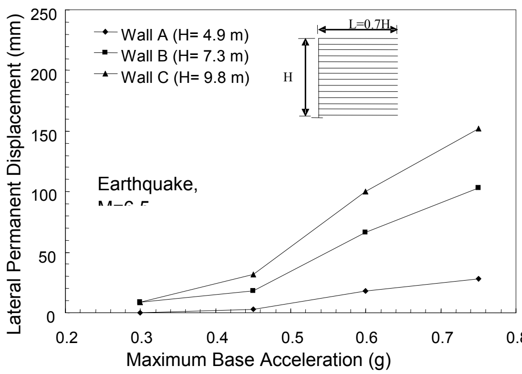

Figure 6a: Computed wall deformation with uniform reinforcement (M = 6.5)

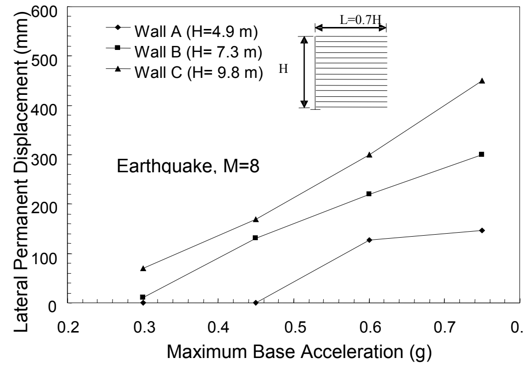

Figure 6b: Computed wall deformation with uniform reinforcement (M = 8.0)

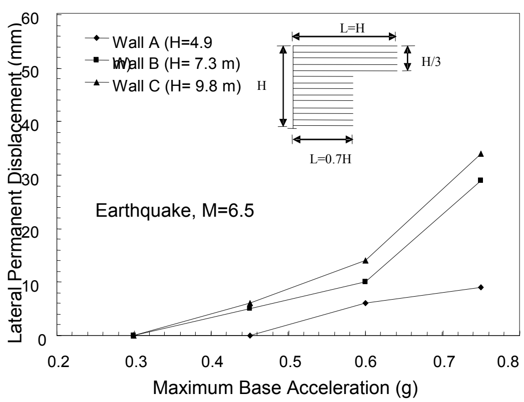

Figure 6c: Computed wall deformation with variable reinforcement (M = 6.5)

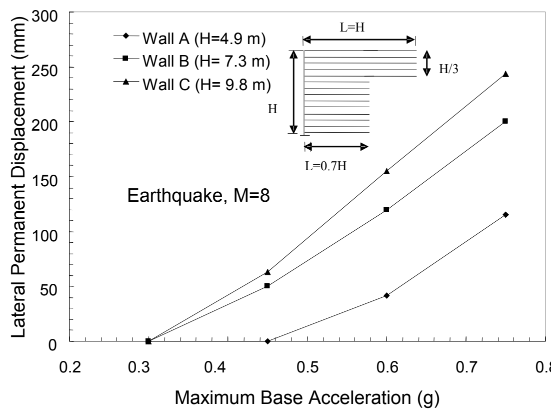

Figure 6d: Computed wall deformation with variable reinforcement (M = 8.0)

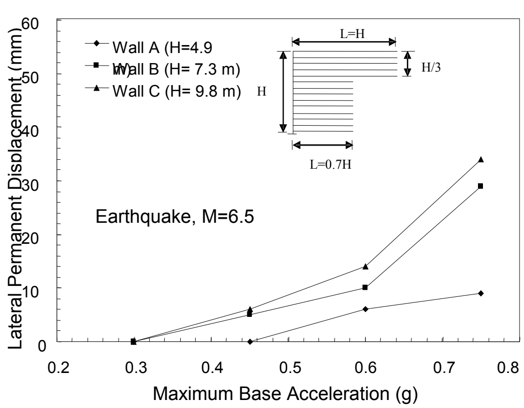

Figure 6e: Computed wall deformation with variable reinforcement (M = 6.5)

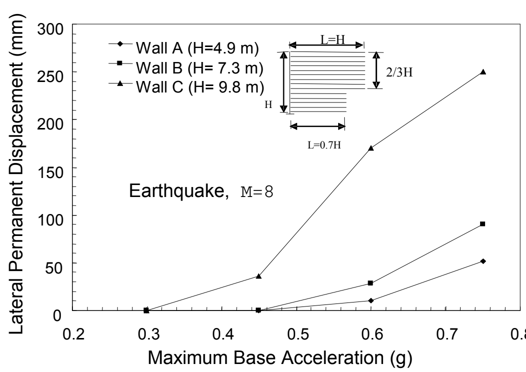

Figure 6f: Computed wall Deformation with variable reinforcement (M = 8.0)

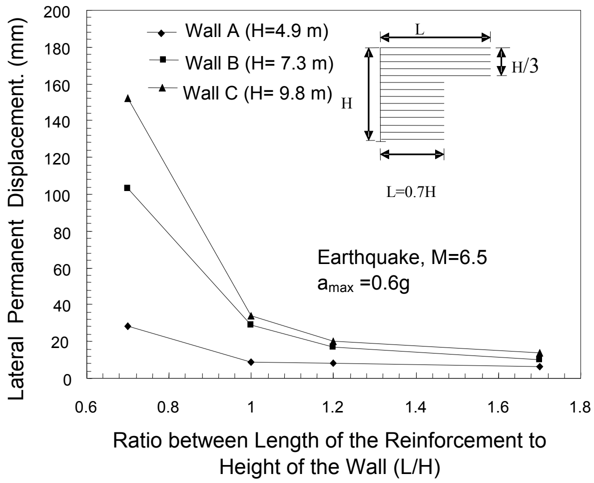

Figure 7a: Computed wall deformation for various L/H ratios (M = 6.5; a_{max} = 0.6g))

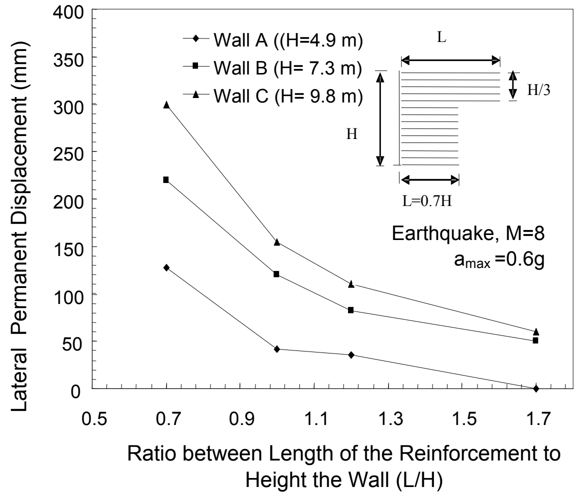

Figure 7b: Computed wall deformation for various L/H ratios (M = 8.0; a_{max} = 0.6g)

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