Study on Seismic Performance of Reinforced Soil Walls to Modify the Pseudo Static Method

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Abstract—This study, tries to suggest a design method based on displacement using finite difference numerical modeling in reinforcing soil retaining wall with steel strip. In this case, dynamic loading characteristics such as duration, frequency, peak ground acceleration, geometrical characteristics of reinforced soil structure and type of the site are considered to correct the pseudo static method and finally introduce the pseudo static coefficient as a function of seismic performance level and peak ground acceleration. For this purpose, the influence of dynamic loading characteristics, reinforcement length, height of reinforced system and type of the site are investigated on seismic behavior of reinforcing soil retaining wall with steel strip. Numerical results illustrate that the seismic response of this type of wall is highly dependent to cumulative absolute velocity, maximum acceleration, and height and reinforcement length so that the reinforcement length can be introduced as the main factor in shape of failure.

Considering the loading parameters, geometric parameters of the wall and type of the site showed that the used method in this study leads to efficient designs in comparison with other methods, which are usually based on limit-equilibrium concept. The outputs show the over-estimation of equilibrium design methods in comparison with proposed displacement based methods here.

Keywords—Pseudo static coefficient, seismic performance design, numerical modeling, steel strip reinforcement, retaining walls, cumulative absolute velocity, failure shape.

I. INTRODUCTION

THE first idea about reinforcing soil systems was proposed by Casagrande, but the first novel form of utilizing reinforced soil in modern soil structures was presented by Henri Vidal in 1960s. The reinforced soil term is attributed to reinforcing soil with tension elements such as rebar, steel strip and geotextile. The useful effects of reinforcing soil with tension include increasing tensile and shear resistance of soil, which is the result of existing friction between soil and reinforcing material. In addition to lateral load capacity, reinforced soil retaining walls have vertical load capacity. Therefore, because of the passing traffic on walls in road construction projects, these kinds of walls are seriously suggested by engineers to be utilized in the projects. Ease of implementation and appropriate ductility of these walls in comparison to concrete retaining walls indicate the benefits of using these kinds of walls. 3 critical elements, including soil, reinforcing elements and facing are used as shown in Fig. 1[1].

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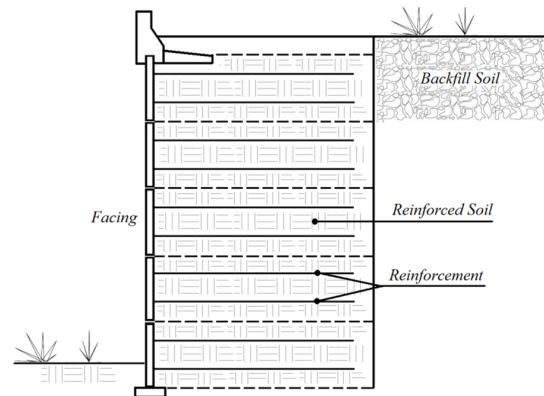
Fig. 1 Cross section of a reinforced soil system

Increasing the flexibility and ductility of reinforced soil system in comparison to other retaining systems, have revealed the improvement resulted from the seismic performance of this type of system. Owing to this reality, more need of identifying effective parameters of seismic performance of these kinds of structures would be needed [1].

Some of these verifications are cited here. One of the first researches was accomplished by Richardson and Lee [2], in which steel reinforcing systems had been verified. They then worked on several reinforced soil walls subjected to horizontal acceleration.

Dynamic acceleration resulted in smoother fracture surface, larger horizontal force and nonlinear distribution of resulting force from dynamic loading on facing. The first full-scale model was verified by Richardson et al. [3], in order to simulating the earthquake impact on this 6-meter height wall, which was implemented by explosion. Designing the wall with conservative methods for static case was the reason of acceptable behavior of wall in dynamic situation. Maximum dynamic forces of strip include primary static force plus the dynamic force from the explosion. Generally, in longer strips, forces that are more dynamic would be induced. Maximum measured dynamic force is considerable lesser than calculated forces by seismic design method of Richardson and Lee [2] and the reasons include the influence of length, array and congestion of reinforcing elements in embankment.

Howard et al. [4] performed centrifugal tests for wall samples, which were reinforced by galvanized steel mesh with length between 0.5 to 1.4 times of wall height. Finally, they proposed a bilinear fracture mode, on the basis of their centrifugal test results (Fig. 2).
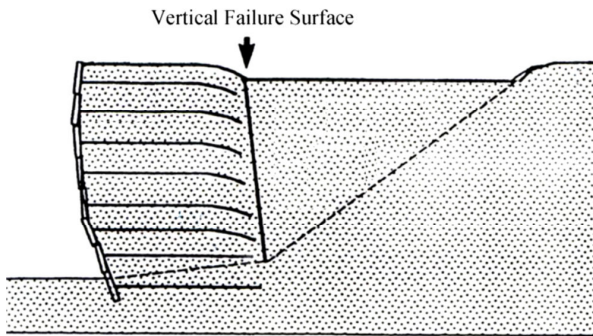
Fig. 2 Fracture surfaces in centrifugal tests

In the studies investigated by Hatami [5], it has been resulted that the stiffness of the reinforcing elements has slight effect on wall response under static loads, but the distribution and force on backside of the wall in earthquake is thoroughly influence by this stiffness.

Bathrust et al. [6] verified the effect of stiffness, length and vertical distance of reinforcing elements on response of walls to dynamic forces in shaking table tests. The results indicate that by increasing the stiffness of reinforcing elements, the displacement of wall reduces considerably. Furthermore, imposing forces on wall and reinforcing elements are highly influenced by arrangement of reinforcing layers, reinforcing system type, and layer distances from each other.

Shaking table tests on 3 models presented by Nandkumaran et al. [7] were also carried out. In these tests, it has been observed that rigid and flexible walls have different behavior during earthquake. It was also shown that dynamic active earth pressure distribution is completely nonlinear on back surface of the wall. Influence point of this pressure in rigid wall is lesser, so that the active earth pressure effect point in flexible walls fell between 0.364H and 0.433H. Based on this study, it has been suggested that in pseudo static method, horizontal acceleration coefficient correspond to (1).

$$k_h = \frac{2\pi f}{g} V_{max} \qquad (1)$$

II. SELECTION OF PSEUDO STATIC COEFFICIENT AS A FUNCTION OF SEISMIC PERFORMANCE

Most of the design methods for reinforced soil walls are based on limit equilibrium methods and displacement-based methods are rarely utilized. Therefore, pseudo static methods have become more popular due to the ease of use and lower expenses, in comparison to time history analysis. On the other hand, pseudo static methods lack exactness and present more conservative results, because of ignoring major loading parameters, geometric properties of wall and seismic performance. Time history dynamic analyses are more precise, owing to consider all the aforementioned parameters, but have high expenses and time-consuming process, which have attenuated the popularity of utilizing this method. Regarding to above-mentioned reasons, in this paper it has been tried to

run dynamic analyses to determine pseudo static coefficient values on the basis of parameters such as seismic performance of wall and geometric properties (2).

$$k_h = f\left(Seismic\ Performance\right) \qquad (2)$$

It should be noted that in all common methods and valid codes' suggestion, pseudo static coefficient is merely defined as function of maximum acceleration (3), which put doubt on reality and precision of obtained results.

$$k_h = \left[1.45 - \frac{a_{max}}{g}\right] a_{max} \qquad (3)$$

In order to define pseudo static coefficient as a function of the effective parameters on seismic performance, firstly, wall height and strip length were selected among geometric properties of the system and base acceleration and cumulative absolute velocity were selected among loading properties as crucial variables. Then, the seismic performance of each of these models whit different heights and strip lengths was determined under harmonic loading whit different base accelerations and cumulative absolute velocities. Finally, using the pseudo static coefficient values of each models, the pseudo static coefficient value was defined as a function of maximum normalized horizontal displacement and performance levels (4).

$$k_h = f\left(Displacement\right) \qquad (4)$$

III. STEPS OF THE RESEARCH

Regarding to predefined purposes, the steps of this study would be introduced here.

A. Selection of Numerical Models under Investigation

1. Geometric Properties of the Numerical Model

In order to run analyzes finite difference software FLAC is utilized here. Using various behavior models of soil, capability of material interaction modeling, considering nonlinear behavior of materials, appropriate modeling of materials during earthquake and capability of programming by users are all of advantages attributed to this software [8].

As the height of structure performs a significant role in seismic behavior of reinforced soil system, height of the structure is chosen as the major variable considered here in this paper. Therefore, verifying the impact of the structure's height on pseudo static coefficient is carried out by selecting six categories including 3, 4.5, 6, 7.5, 9 and 10.5 meters for height of the structure.

For the sake of omitting the influence of defined boundaries on analysis results, and based on implemented sensitivity analysis, height of the soil bulk at the back of the wall is considered 5 times of wall height and 2 times of wall height in front of the wall in each model. In addition, regarding to considerable effect of foundation dimensions on system deformations, and for considering this effect and omitting the

influence of soil type, a foundation with a height equal to 0.25 times of the structure height is utilized by sensitivity analysis. The schematic illustration of a reinforced soil system with the aforementioned heights is illustrated in Fig. 3 [9].
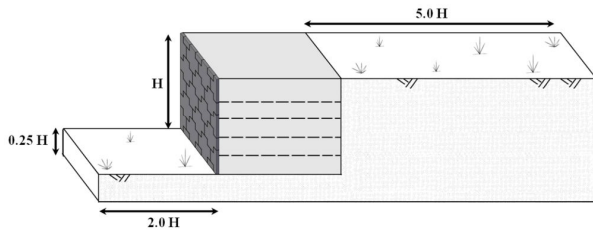

Fig. 3 Dimensions of fabricated models

Furthermore, in order to pass wave through the model and prevent from numerical deconstruction, mesh size is nearly considered equal to the largest input wave frequency [8].

2. Geotechnical Parameters

In this paper, it has been tried to consider soil type effects by introducing 3 kinds of soil profiles, which are represented as 1 to 3 in 2800 standard of Iran. Considering and using geotechnical parameters from many boreholes representing 3 kinds of soil type in different regions of Iran, geotechnical parameters for modeling the foundation and reinforced soil are chosen.

Soil materials used in reinforced soil walls almost constitutes granular materials and cohesive or non-cohesive granular soil, which should have reached to at least 95 percent of compactness. Geotechnical parameters considered here are listed in Table I.

Equivalent linear method is utilized for running analyzes. In this method, material behavior model is assumed linear. Soil stiffness and damping values are proportional to strain. Regarding to the soil nature, which is considered granular and the suggested grading range for materials, maximum shear modulus values (in low strains) and hysteresis-damping values would be determined.

TABLE I
GEOTECHNICAL PROPERTIES OF REINFORCED SOIL

| Parameter | Soil Type | | | Reinforced Soil | Reference |
	Type I	Type II	Type III		
Shear Wave Velocity *(m/s)*	925	560	375		[10]
N_{SPT}	76	34	12		[11], [12]
Shear Modulus *(psf)*	$G_{max} = 20 \times 1000(N_1)_{60}^{1/3}(\sigma'_m)^{1/2}$			$G_{max} = 3080 \dfrac{(2.17 - e)^4}{1 + e}(\sigma'_0)^{0.8}$	[13]
Specific Weight *(kg/m³)*	2187	1903	1691	2050	[14]
Maximum Internal Friction Angle *(°)*	$\phi_{max} = 1.85^0(\dfrac{N}{0.7 + \sigma'_v/P_a})^{0.6} + 26^0$			39	
Dilation Angle *(°)*	$\psi_{max} = (\dfrac{1}{6} \sim \dfrac{1}{8})\phi_{max}$			5	[15]
Ultimate Internal Friction Angle *(°)*	$\phi_{residual} = \phi_{max} - 0.8\psi$			35	
Poisson's Ratio	$v = \dfrac{1 - sin\phi}{2 - sin\phi}$			0.3	[16]
Cohesion *(kPa)*	5			5	

Shear modulus and Internal Friction Angle values, as shown in Table I, is a function of confining stress, which varies by depth. FISH programming ability of FLAC software is utilized here for modeling in this paper, in order to modify shear modulus and Internal Friction Angle for each element considering confining stress [9].

In order to prevent from lengthening calculation time because of using interface elements, applying soil bulk interface with cover is performed by using continuous elements with equivalent properties [17].

3. Reinforcing and Facing Elements

Vertical and horizontal distances of reinforcing strips, strips length and its dimensions have an effective impact on behavior of reinforce soil walls. In this paper, regarding to apply shell elements in cross shapes, their dimensions and implementation methods, distance of strips in horizontal and vertical dimensions are equally selected 75 centimeters. The procedure is that two horizontal and vertical reinforcements with equal distances are erected on each 1.5 meter shells.

Strips are selected in common 60 x 5 millimeters dimensions for modeling and lengths as a function of structure's height

The wall facing was modeled as continuous concrete panels with a thickness of 0.15 m that was hinged together. Other specifications of steel strips and facing are listed in Table II [18].

Interaction between strip and soil is one of the most important parameters in modeling the strips. For this purpose, steel strips are selected from elastic-plastic STRIP elements with negligible compressive strength. Stripe element has appropriate ability in modeling yield in tension and steel rupture limit. In addition to this, nonlinear modeling of interaction between reinforcement and soil is one of the major merits of this element kind [18].

The wall-soil interface was modeled using a thin soil column, 0.05 m thick, directly behind the facing panel. A no-slip boundary was used between the thin soil column and the facing panel. The properties of soil-wall interface column material were presented in Table III [19], [20].

TABLE II
FACING AND STRIP PROPERTIES

Parameter	Strip	Facing	Unit
Specific Weight	7800	2500	kg/m^3
Modulus of Elasticity	200	20	Gpa
Dimensions	6 x 0.5	150 x 150 x 15	cm
Rupture Stress	235	21	Mpa

TABLE III
WALL-SOIL INTERFACE PROPERTIES

Parameter	Equivalent parameter
Specific Weight	$\gamma_{interface} = \gamma_{soil}$
Shear Modulus	$G_{interface} = G_{soil}$
Poisson's Ratio	$v_{interface} = v_{soil}$
Cohesion	$C_{interface} = 0$
Internal Friction Angle	$\varphi_{interface} = 2/3\ \varphi_{soil}$

4. Boundary and Support Conditions

Boundary conditions encompass great significance in static and dynamic analysis. In static state, roller supports are utilized in modeling of environs soil. This means that in lateral wall supports, movement of soil in horizontal direction is prevented, but is free in vertical direction and in bottom support of the model, the reverse is true. This analysis method would lead the modeling to be near to the reality. In dynamic analysis, regarding to the possibility of wave reflection through in the model and severe decrease in precision of results, static boundaries would be replaced by quiet boundaries.

5. Damping

As cited previously, damping which is used here is a function of strain level. In the aforementioned software, by using available patterns and also regarding to the assumed soil type, related damping curve and shear modulus would be applied to the model, which is illustrated in Fig. 4 [13].
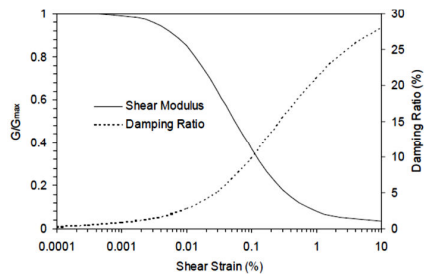

Fig. 4 Damping and modulus variation curve

B. Static and Dynamic Analyzes

1. Model Construction Corresponding to the Reinforced Soil System Applying Method

FLAC software has the ability of step by step modeling technique. It means that, as the embankment construction of reinforced soil walls are implemented step by step, the modeling process should correspond to the real construction process. The first step is that the lower part of the wall, namely foundation, made stabilized. In the next step, the first layer of block is installed and then strips are erected and embanked. Then, the second step of static analysis should be initiated. All these steps should continue until the end of embankment and construction process (Fig. 5). In this step, system is analyzed for gravity loads or surcharge loads. In the static step, dynamic loads have no role in the system and static displacement would be removed at the end of the process.

2. Dynamic Loading

Dynamic analyses of a reinforced soil wall subjected to simulated horizontal foundation shaking due to an harmonic loading with various amplitudes in this study. The selected harmonic load should be near to the real situation, which means that the amplitude should gradually increase and then decreased. In this way, it represents an appropriate model during earthquake occurrence. This harmonic load corresponds to (5) and shown in Fig. 6.

$$\ddot{Z} = \sqrt{\beta.e^{-\alpha t}.t^{\xi}}.Sin(2\pi f t) \qquad (5)$$

In which, f is the loading frequency and ζ, α and β are factors that demonstrate loading shape and the number of cycles.

Regarding to the studies investigated by Yazdandoust [9], harmonic load frequency, maximum acceleration and cumulative absolute velocity Instead effective time, are considered in the base of soil type and regions seismic risk categories (Table IV).

The numerical grid for the reference geometry, boundary condition, wall-soil interface and reinforcing elements in the current study is illustrated in Fig. 7.

3. Dynamic Analysis

In order to ensure of modeling accuracy, the established model is verified with results of Richardson and Lee [2] tests. After model verification, the models made are applied for dynamic analysis.

After accomplishment of static analysis and reaching to equilibrium for reinforced soil system, harmonic Loads are applied to the foundation level and the dynamic analysis would be performed. During each of the dynamic analyses, the history of horizontal wall displacements as a representative of the seismic performance are recorded (Fig. 8). Then the impact of each of the study variables Investigated on horizontal displacements history.

IV. RESULTS AND DISCUSSIONS

A. Investigation the Effect of Length Stripe on Seismic Displacements

As a sample, horizontal displacements histories for 3, 6 and 9-meter high structure in specified seismic situation are presented in Fig. 9. The results show that the maximum of residual displacements in one specific acceleration and CAV level would progressively increased by reducing the strip length. This viewpoint is clarified that there is a small effect of reinforcement length on the maximum wall displacement displacements with the being greater of L/H = 0.8. Then L/H = 0.8 is the critical length in seismic condition.
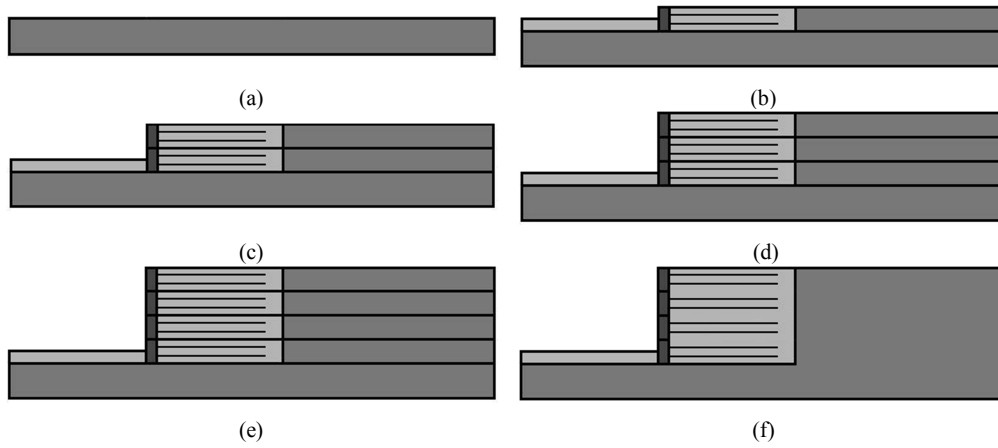
Fig. 5 Construction Process: (a) Construct the Foundation; (b) Construct the First Layer; (c) Construct the Second Layer; (d) Construct the Third Layer; (e) Construct the Forth Layer; (f) End of the Construction Process
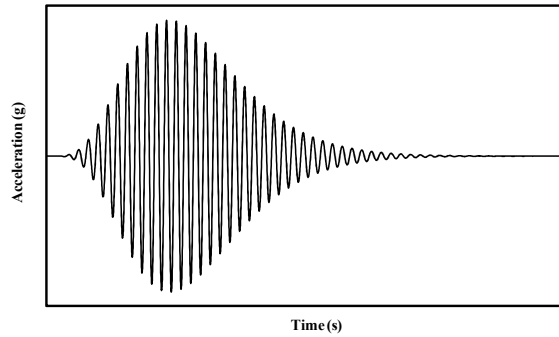
Fig. 6 Harmonic Load

TABLE IV
SEISMIC PROPERTIES FOR VARIOUS ZONES

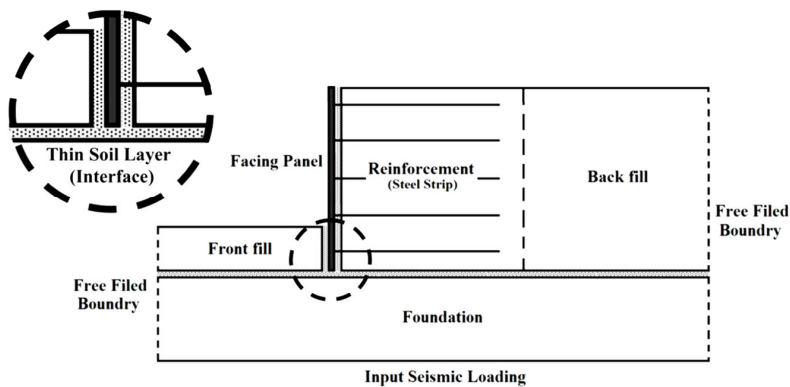|	Seismic Risk Categories								
	Very High Risk Level			High Risk Level			Medium Risk Level		
Soil Type	Type I	Type II	Type III	Type I	Type II	Type III	Type I	Type II	Type III
Predominant Period (s)	0.15	0.20	0.30	0.15	0.20	0.30	0.15	0.20	0.30
Maximum Acceleration (g)	0.45	0.53	0.67	0.40	0.47	0.61	0.35	0.40	0.55
CAV (cm/s)	304~1697	496~1063	1063~2808	327~1115	395~1720	724~2416	204~892	428~1502	604~1981

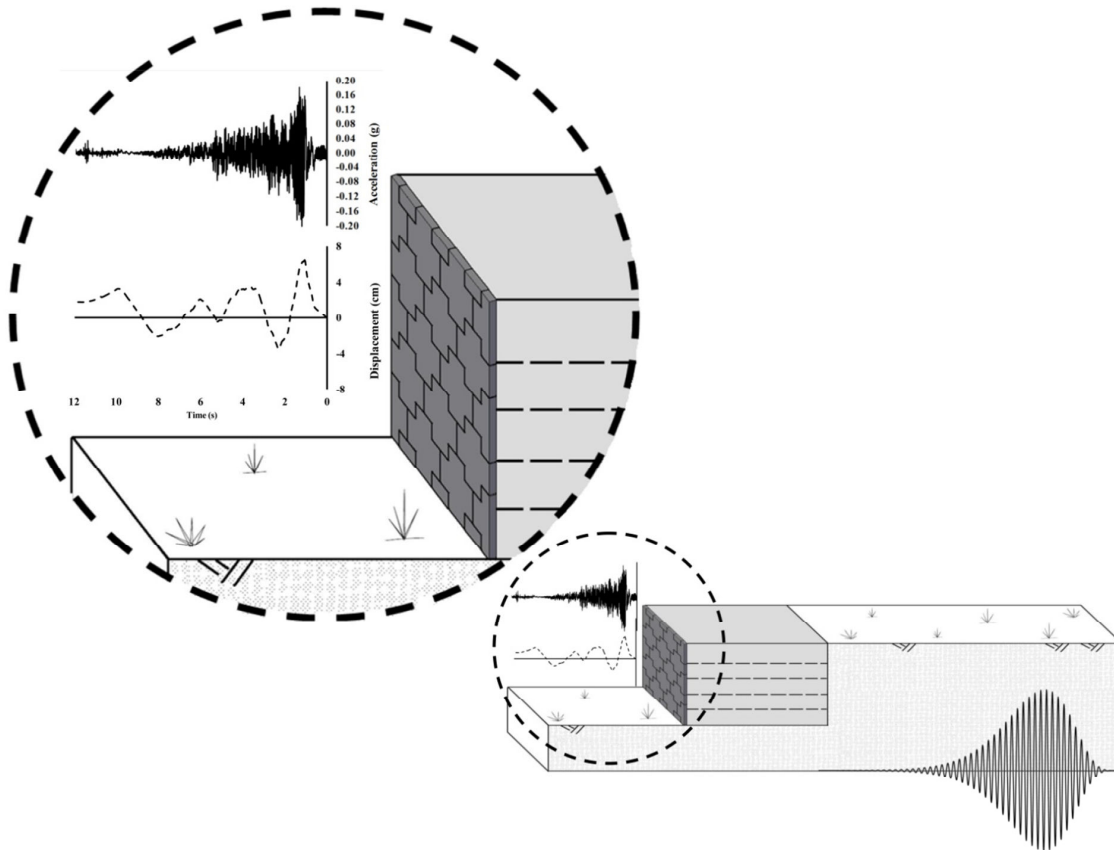Fig. 7 Numerical grid for the reference reinforced soil wall

Fig. 8 Apply the harmonic Load and recording the history of horizontal wall displacements & the history of acceleration response

Theoretically, residual displacements are produced in structure, when the acceleration value exceeds Ky. This would be true in numerical analysis, as well, but the difference is that an exact value for Ky could not be considered in numerical analysis. When the structure is prone to dynamic load, if the applied acceleration level be lower than the theoretical value of Ky, induced residual displacement in the structure would differ with the theoretical one, but major induced residual displacement relates to the situation, in which acceleration level exceeds from Ky. This would result in progressive enhancement in residual displacements of the structure, when the difference between the applied acceleration level and critical acceleration level, be considerable. From the aforementioned diagrams, tangible difference in mode of displacement of wall shell is extracted. The discussion about mode of displacements for reinforced soil walls are relatively sophisticated, due to variety of reinforcing systems, stiffness and various facings. In reinforced soil structures, displacement mode is a function of total stiffness of reinforced soil, which itself is a function of soil compactness, stiffness of reinforcing elements and vertical distance of reinforcing elements from each other.

The more the stiffness of reinforced soil and vertical distance of reinforcing elements be, the mode of displacements would tend to be convex. It is also obvious that as the reinforced soil becomes stiffer, displacement mode would tend to overturning mode. In the reinforced soil structures under investigation, as the height of the shells are relatively high in comparison to the height of the structure, the convex mode becomes intangible. Most of the structures, in which convex mode are governing, maximum displacement is related to the first and second shells from the bottom. Another important point is that by increasing height of the structure, length of reinforcing elements would increase, inasmuch as the length of reinforcing elements had been assumes as a function of structure's height.

On the other hand, increasing the height would result in length enhancement of the strips. As shown here, normalized displacement values would decrease by increasing the height of the structure. This point infers that for reaching to similar behavior, we could increase the strip length, nonlinearly. It means that, considering strip length as the linear function of height in reinforced systems with steel strips, which have bilinear fracture surface, is not suitable for structure's behavior.

B. Investigation the Effect of Cumulative Absolute Velocity on Seismic Displacements

Example horizontal displacement histories of the wall facing for histories for 3, 4.5 and 6-meter high structure in different seismic situation whit different CAV are presented in Fig. 10.
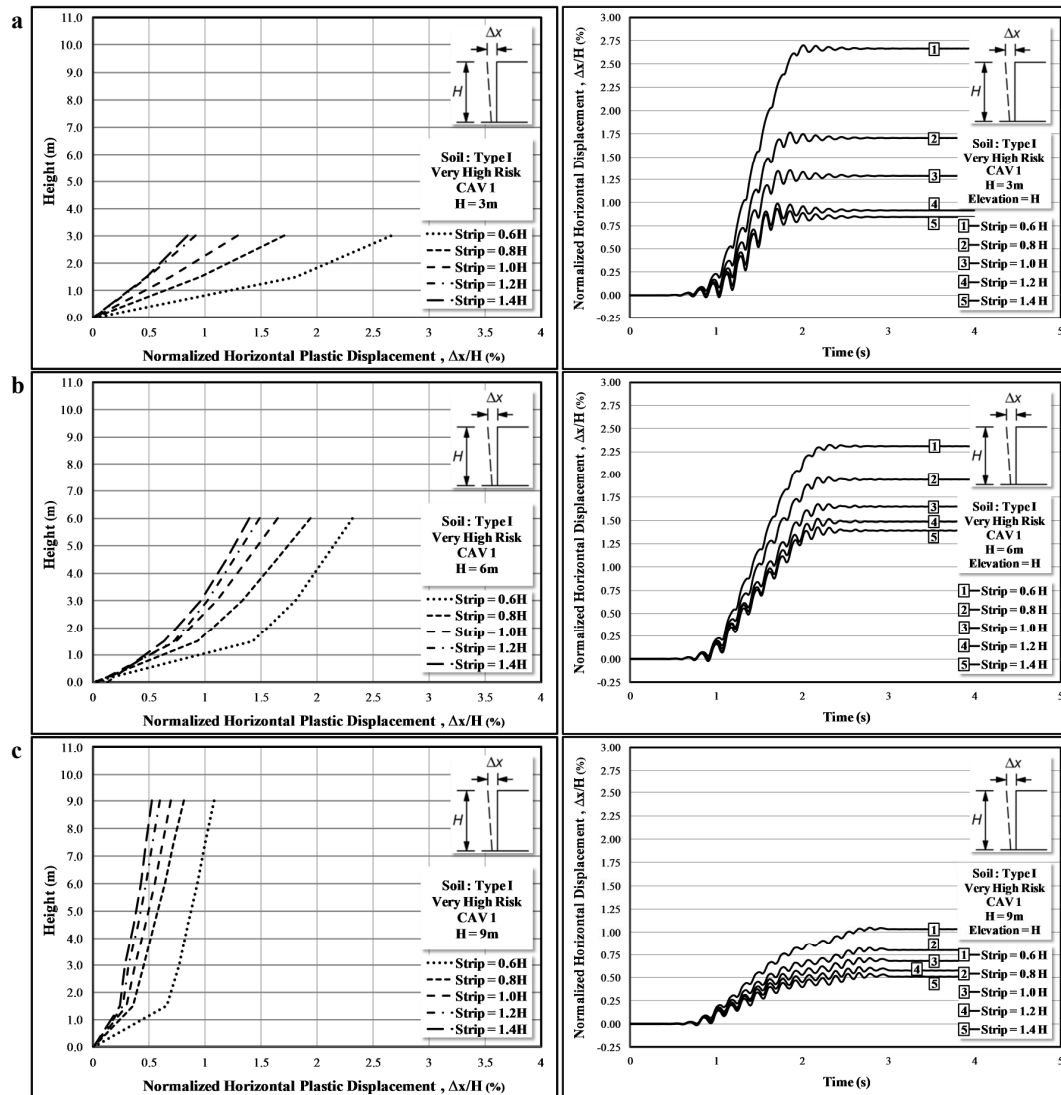
Fig. 9 History of normalized horizontal displacements and horizontal displacement profiles walls with different strip lengths: (a) The wall with 3m height; (b) 6m height; (c) 9m height

The results obtained demonstrate the CAV parameter has a significant effect on system seismic displacements so that the maximum of residual displacements would progressively increased by increasing the amplitude of cumulative absolute velocity. But the amount of this impressments increases dramatically during increase the height of system.

Since in a situation with constant acceleration, only increasing parameter with increasing number of loading cycles is possible, Therefore increasing this parameter can be caused fatigue phenomenon and ultimately increasing plastic deformations occurring in reinforced soil system.

Also, the magnitude of elastic lateral wall displacements is greatly influenced by CAV than plastic displacements.

In other hand, there is the optimal convergence between CAV and residual displacements that cause, CAV is the suitable criterion for selecting design earthquake in performance base design method (Fig. 11).

C. Investigation the Effect of Maximum Acceleration on Seismic Displacements

As a sample, horizontal displacements histories for 7.5, 9 and 10.5-meter high structure in different seismic situation whit different PGA are presented in Fig. 12.

Based on the results, the maximum of residual displacements would progressively increased by increasing the amplitude of maximum acceleration. But the amount of this impressments decreases dramatically during increase the height of system. This phenomenon is due to the increased rigidity of reinforcement mass in effect exponential growth of the structural reinforcement elements during the height of system. Also, the magnitude of plastic lateral wall displacements is greatly influenced by PGA than elastic displacements, so that the amplitude of maximum acceleration can be seen as a major factor in failure.
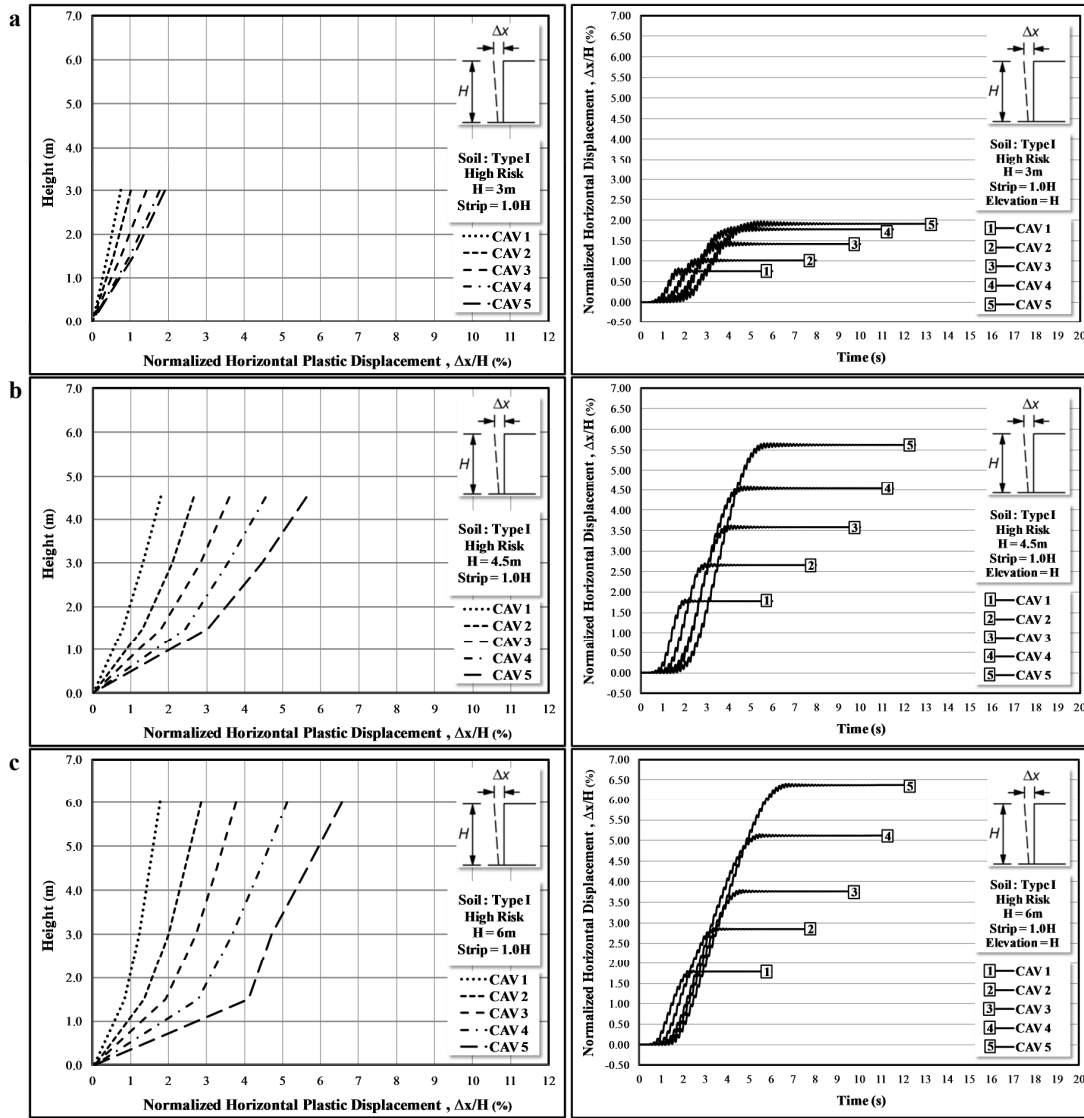
Fig. 10 History of normalized horizontal displacements and horizontal displacement profiles at different CAV: (a) The wall with 3m height; (b) 4.5m height; (c) 6m height
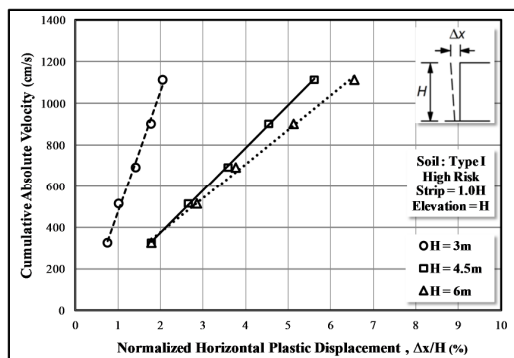

Fig. 11 Optimal convergence between CAV and residual displacements

In other hand, there is the Inappropriate convergence between PGA and residual displacements that cause, PGA is not the suitable criterion for selecting design earthquake in performance base design method (Fig. 13).

D. Investigation the Effect of Length Stripe on the Magnitude of Acceleration Amplification

The effects of the length stripe on the magnitude of acceleration response in reinforced zone and facing are presented in Fig. 14. The data show that increasing the length stripe can reduce the peak acceleration at selected locations in the reinforced soil zone and facing. Also, due to the flexibility of facing, the magnitude of acceleration amplification in it can be seen greater than reinforced soil zone.
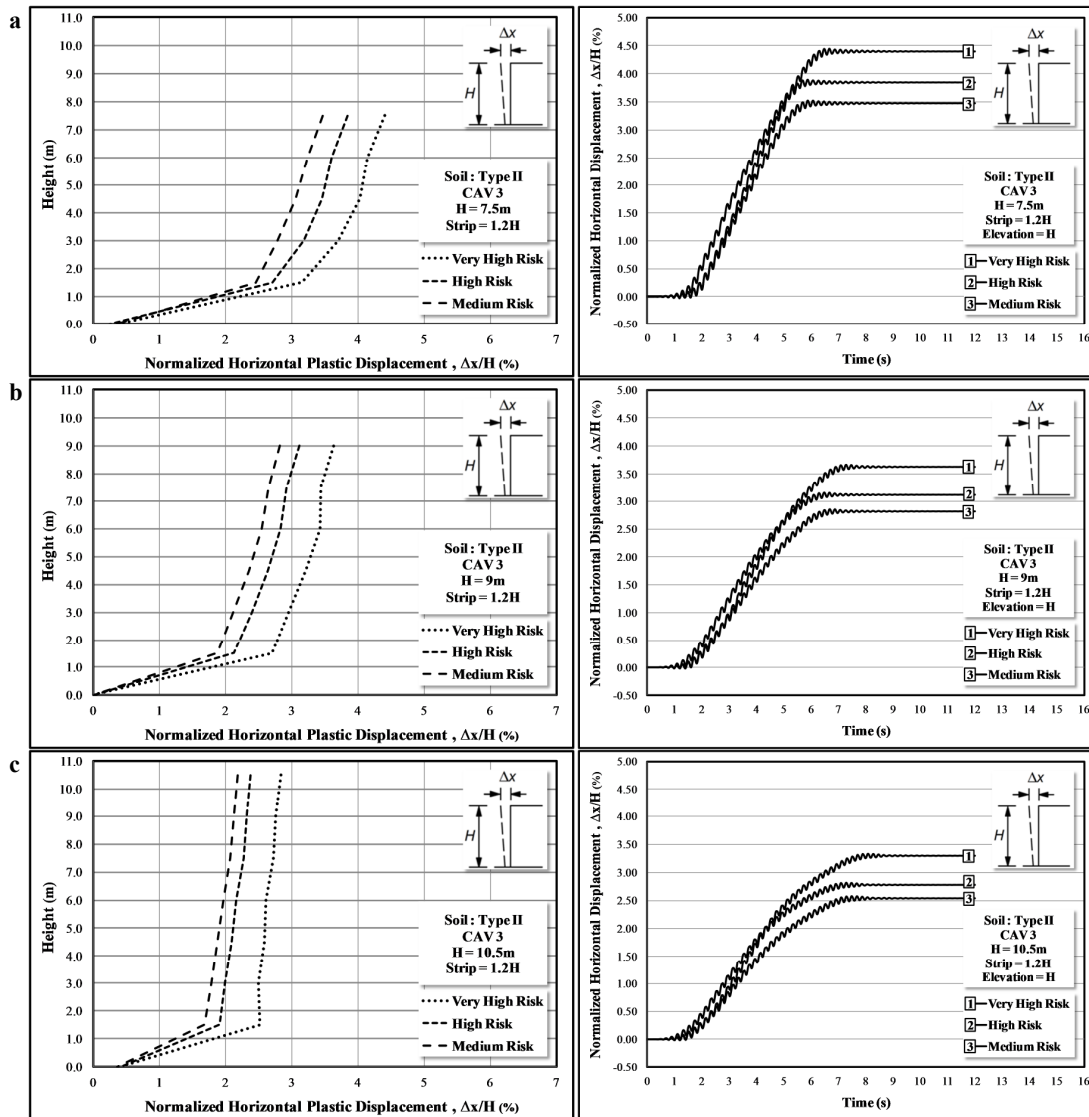
Fig. 12 History of normalized horizontal displacements and horizontal displacement profiles at different PGA: (a) The wall with 7.5m height; (b) 9m height; (c) 10.5m height

E. Investigation the Effect of Length Stripe on Failure Zone

Fig. 15 shows typical plots of shear zones within the reinforced soil zone and in the retained soil for structures whit different length of reinforced elements. This can be seen in Figures that large shear strains has happened at the wall-soil interface and at the reinforced retained soil interface.

The failure volume in each simulation can be approximated by a bilinear wedge with a break point at the inside of the reinforced soil zone. The break point was observed to be at a same elevation for all of the lengths of strips (H/2), while the distance from break point to wall surface gradually reduced by reducing the lengths of strips. Also shown in Fig. 15 are the linear failure surfaces in the retained soil that are predicted from solutions for slip surface orientation using Mononobe-Okabe earth pressure theory [21]-[23]. Orientations of 52° and

62° from the horizontal correspond to computed mean wedge accelerations of approximately 0.65g and 0.4g, respectively, and are in reasonably good agreement with shear zone boundaries. Hence, pseudo static equilibrium methods may be useful to estimate minimum widths for numerical grids if the influence of yielded soil zones on the wall response is to be captured in numerical simulations.

To further clarify the failure mechanism, the geometry at failure is idealized in Fig. 16 as a function of the length of reinforcing elements the where the two soil blocks are divided into three zones. The block comprised of the reinforced soil mass is now divided into a upper zone I and lower zone II, and the active earth pressure wedge block is indicated by zone III. So that, the soil in Zones I and II moved outward as a block held together by the upper half of the band of strips.
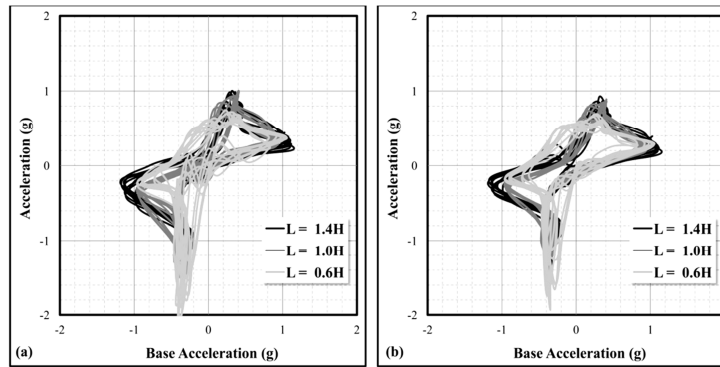
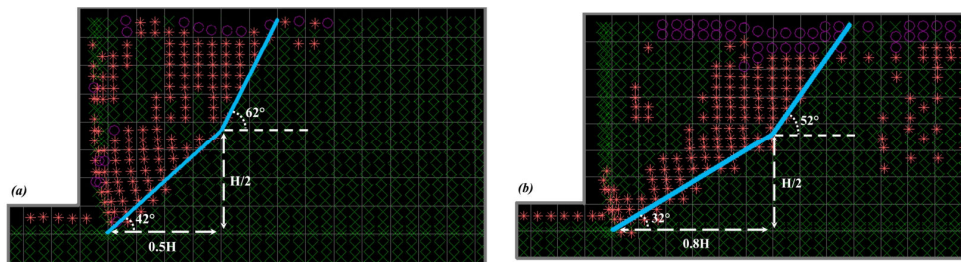Fig. 14 Magnitude of acceleration response: (a) Facing; (b) Reinforced zone

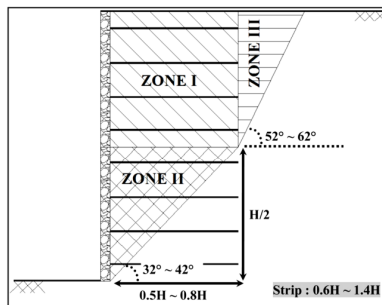Fig. 15 Shear zones at different length of strips: (a) L/H = 0.6; (b) L/H = 1.4

Fig. 16 Failure surfaces and soil zones involved in failure mechanism

F. Pseudo Static Coefficient Based on Seismic Performance

Using available codes and accounting for safety factors, each of the analyzed structures mentioned in previous sections of this paper, are representatives of one horizontal pseudo static coefficient. In order to determine this coefficient related to any reinforced soil structure whit specific height, length of strip, the limit equilibrium method is used, the pseudo static coefficient related to the height of structure, and length of each strip is determined, accounting for the safety factor equal to one [9]. After determining this pseudo static coefficient, one could express this coefficient as a function of CAV and performance levels in different categories of soil type and regions seismic risk. (Fig. 17).

To select the limits of performance levels for reinforced earth walls, result of studies conducted by Partovian [24], is used. In these studies, $\Delta x/H = 1\%$ as a start plastic limit and $\Delta x/H = 6\%$ as a start failure limit is introduced.

Recollecting the results, it is obvious that the assumptions of available codes are somehow conservative in seismic design, which is due to ignoring allowable displacements after an earthquake. Therefore, a design based on allowable displacement would lead to more suitable and economical designs.

V. CONCLUSIONS

Based on implemented studies on the behavior of reinforced soil walls, one could judge that these kinds of structures had demonstrated an appropriate behavior and acceptable performance in past earthquakes, due to suitable flexibility and ductility, which they possess. Most of the available design methods are based on limit state equilibrium equations. Considering the safety factors for internal and external instabilities, the forces in reinforcing elements might vary. Owing to simplification assumptions in available design codes, mostly these methods are extremely conservative and non-economical. In this paper, using induced allowable displacement concept, these methods are improved and modified. The results illustrate that by increasing the applied maximum acceleration, CAV and decreasing the length of reinforcing elements, displacements grow consequently. Also it is clarified that there is a small effect of reinforcement length on the maximum wall displacements with the being greater of L/H = 0.8 so that L/H = 0.8 is the critical length in seismic condition. In other hand, there is the optimal convergence between CAV and residual displacements that cause, CAV is the suitable criterion for selecting design earthquake in performance base design method.

In addition, the geometry of the observed failure mechanisms in all of models was similar that thus indicating a common failure pattern that can be expected for prototype

structures under prototype loading conditions. This failure pattern includes three soil zones and two failure surfaces as sketched in Fig. 15. According to this figure, the reinforced mass moves laterally as an intact block in response to the inertial forces caused by horizontal shaking. However, these movements are restricted due to the anchoring mechanism of the bottom-row strips that extend beyond the failure surface.

Consequently, the reinforced mass of zones I and II initially rotates about the connection between the facing and the bottom-row strips. During continuous strong shaking, the pullout capacity of the bottom-row strips is eventually reached and incremental sliding along an approximately bilinear failure surface beings. Due to the significant reduction of the lateral stresses behind zone I, an active earth pressure wedge (zone III) develops.
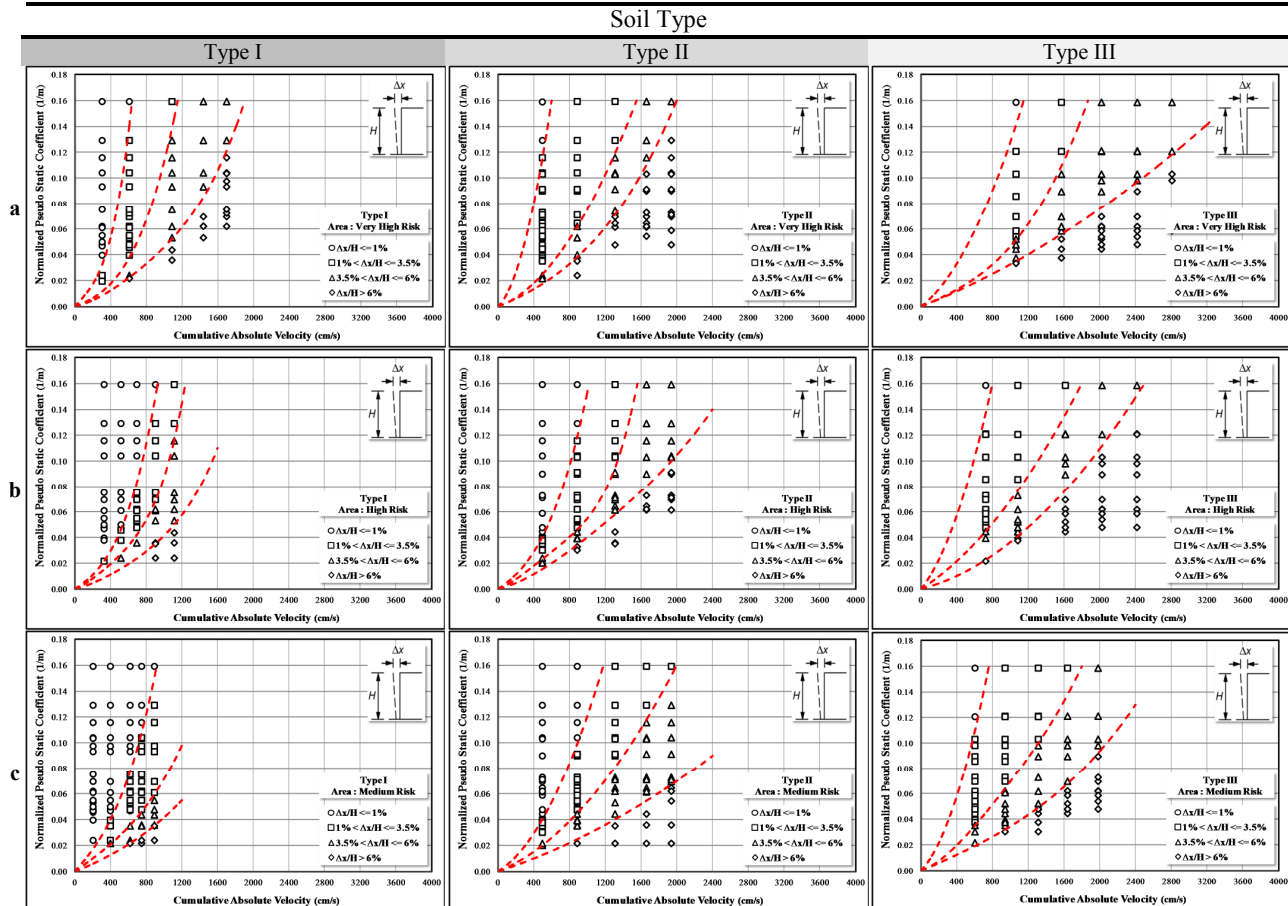

Fig. 17 Pseudo static coefficient based on seismic performance: (a) Very high risk level; (b) High risk level; (c) Medium risk level

Implementing numerical investigations, pseudo static coefficient values are determined, based on various performance levels. In conclusion, the results state that the assumptions of available seismic design codes are highly conservative, due to ignoring allowable displacements after earthquake occurrence. Consequently, performance based design concept would result in more suitable and economical structure.

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