# Determination of MSE Wall Pseudo Static Coefficient Based on Seismic Performance

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# DETERMINATION OF MSE WALL PSEUDO STATIC COEFFICIENT BASED ON SEISMIC PERFORMANCE

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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. In this case, loading characteristics, such as magnitude, frequency, peak ground acceleration and geometrical characteristics of reinforced soil structure 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.

In addition, the authors has tried to simply suggest the equivalent harmonic loading of selected acceleration records. Considering the loading parameters, mechanically stabilized earth wall parameters and type of the site showed that the used method in this study leads to most efficient designs in comparison with other methods which are generally suggested in cods that are usually based on limit-equilibrium concept. The outputs shows the over-estimation of equilibrium design methods in comparison with proposed displacement based methods here.

## 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 rebars, 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.

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.

Some of these verifications are cited here. One of the first researches was accomplished by Lee et al (1973), in which steel reinforcing systems had been verified. Richardson and Lee (1975) then worked on several reinforced soil walls subjected to horizontal acceleration.

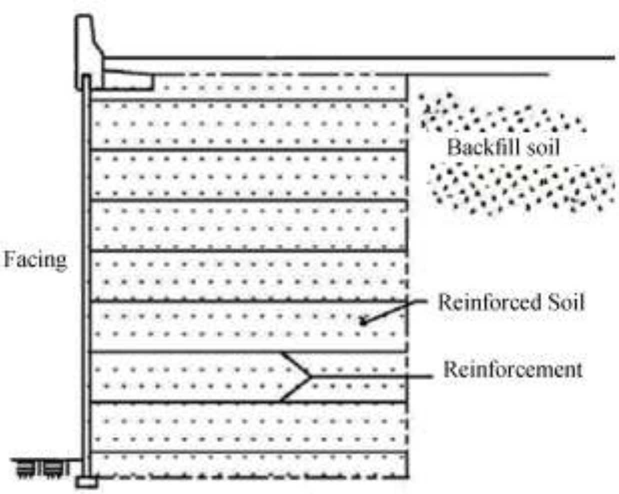



*Figure 1: Cross section of a reinforced soil system*

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 (1977), 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, more dynamic forces would be induced. Maximum measured dynamic force is considerable lesser than calculated forces by seismic design method of Richardson and Lee (1975) and the reasons include the influence of length, array and congestion of reinforcing elements in embankment.

Howard et al (1999) 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.

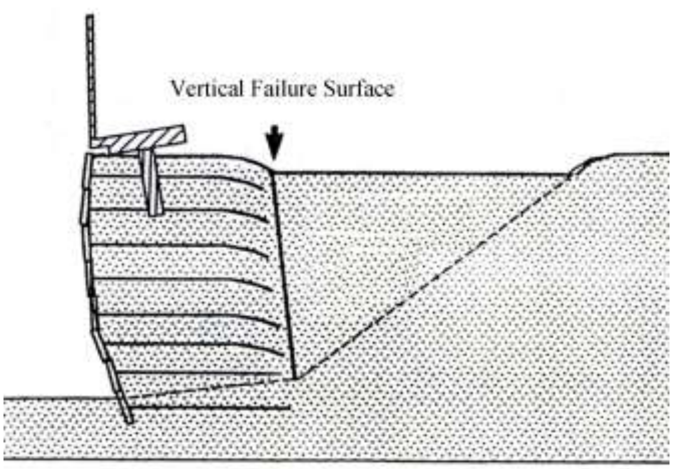



*Figure 2: Fracture surfaces in centrifugal tests*

In the studies investigated by Hatami (2003), 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 back side of the wall in earthquake is thoroughly influence by this stiffness.

Bathrust et al (2006) 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 (1974) 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. On the basis of this study, it has been suggested that in pseudo static method, horizontal acceleration coefficient correspond to Eq. 1.

$$k_h = \frac{2\pi f}{g} V_{\max} \tag{1}$$

In this equation, $V_{max}$ demonstrates the maximum velocity and $f$ demonstrates loading frequency.

## SELECTION OF PSEODU STATIC COEFFISIENT 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 (Eq.2).

$$k_h = f\left(Seismic\ Performance\right) \tag{2}$$

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

$$k_h = \left\lfloor 1.45 - \frac{a_{\max}}{g} \right\rfloor a_{\max} \tag{3}$$

In order to define pseudo static coefficient as a function of the effective parameters on seismic performance and consider the influence geometric properties of structure and seismic loading parameters on seismic behavior of reinforced soil system, height of the structure and strip length are selected among geometric properties of the system (height of structure, slope of the structure, length and arrangement of reinforcing elements, facing properties) and also loading type as crucial variables.

In order to run analyses, harmonic load with variable amplitude is utilized. Firstly, 30 earthquake records were selected and the response of reinforced soil structures to these records was determined. Then, by equating system performance of harmonic loading to mean performance of records, equivalent harmonic load to these 30 records would be chosen for analysis. Finally, pseudo static coefficient value, as a function of maximum displacement and seismic performance would be determined. (Eq. 4)

$$k_h = f(Displacement) \tag{4}$$

STEPS OF THE REASERCH

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

1-Selection of numerical models under investigation

1-1-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.
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 three categories including 4.5, 6 and 7.5 meters for height of the structure.
For the sake of omitting the influence of defined boundaries on analysis results, and on the basis of implemented sensitivity analysis, height of the soil bulk at the back of the wall is considered 5 times of wall height and 1.5 times of wall height in front of the wall in each model. Also, 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 height of the structure is utilized by sensitivity analysis. The schematic illustration of a reinforced soil system with the aforementioned heights is illustrated in fig. 3.
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.

1-2- 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, and soil type 2 is specifically verified here. Considering Tehran as the location under consideration, and using geotechnical parameters from 3 boreholes representing soil type 2 in different regions, geotechnical parameters for modeling the foundation and reinforced soil are chosen.

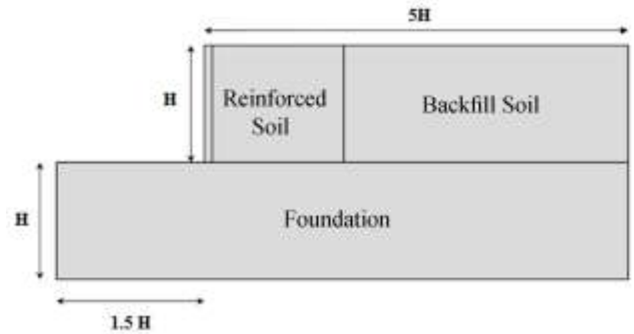



*Figure 3: Dimensions of fabricated models*

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 1.
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 also the suggested grading range for materials, maximum shear modulus values (in low strains) and also hysteresis damping values would be determined.

Table 1: Geotechnical Properties of Reinforced Soil

| Parameter | Foundation | Reinforced Soil | Unit |
|---|---|---|---|
| Specific Weight | 2000 | 2050 | $kg/m^3$ |
| Shear Modulus | $3080\,\frac{(2.17-e)^4}{1+e}\,\sigma_0^{0.8}$ | | $kpa$ |
| Poisson's Ratio | 0.33 | 0.3 | —— |
| Cohesion | 0.1 | 0 | $kg/m^2$ |
| Internal Friction Angle | 0.32 | 0.37 | *Degree* |

Shear modulus value, as shown in table 1, 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 for each element considering confining stress.
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.

1-3- Reinforcing and Facing Element

Vertical and horizontal distances of reinforcing strips, strip 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 2 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. Other specifications of steel strips are listed in table 2.
Interaction between strip and soil is one of the most important parameters in modeling the strips. For this purpose, steel strips are selected from STRIP elements. 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.

Table 2: 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* |

1-4- 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.

1-5- 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.

2- Static and dynamic analyzes

2-1- Model construction corresponding to the reinforced soil system applying method

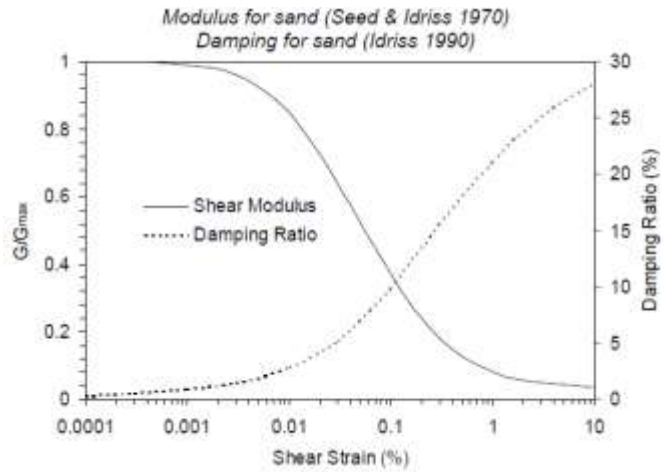



*Figure 4: Damping and modulus variation curve*

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. 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 forces of strips would be removed at the end of the process.

2-2- Dynamic loading

Two types of modified seismic loadings and harmonic loading with various amplitudes are utilized in this study. Consideration of equivalent harmonic is noticed here for the reason of ease in performing dynamic analysis. Primarily, by selecting 30 records, it has been tried to attain sediment response on the surface of ground for soil type 2, in order to run analyzes. The process initiated by choosing Tehran as allocation with very high seismic risk and the consideration of several soil profiles in different stations, which are all representative of soil type 2. The selected records are normalized to bedrock acceleration and analyzed by Deep Soil software on the ground surface. The output records would be then resulted. This task is also implemented for regions with high and moderate seismic risk. In this way, the maximum mean acceleration and dominant mean frequency on the ground surface would be acquired.
Resulted records on the ground surface could be used for running dynamic analysis of reinforced soil wall. In this paper, using response of reinforced soil wall to these records and then comparing them to the reinforced soil structure response to one or more harmonic loads, which represents all 30

records, we could reach to the results.

Supplementary analyzes for reaching horizontal acceleration factor is implemented by using these harmonic loads. Selected harmonic load frequency is determined with regard to dominant mean frequency of applying records. Among selected acceleration records, 12 of them are related to the Iran earthquakes, including Vandik (1976), Tabas (1978), Chamghooreh (2002), Bam (2003) and Baladeh (2004) earthquakes. Acceleration records of Duzce earthquake in Turkey (199) is also selected owing to the similarity to Iran quakes in earth properties. The remained selected records are related to America, including San Ferando (1971) and Northridge (1994) earthquakes. Single degree of freedom structure's response spectrum with 5 percent of damping is illustrated in fig. 5 representing an earthquake on soil profile surface. By averaging the responses, dominant frequency range would be determined, as illustrated in fig. 6.

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 eq. 5, as shown below:

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

In which, $f$ is the loading frequency and $\zeta$ , $\alpha$ and $\beta$ are factors that demonstrate loading shape and the number of cycles.

Regarding to the implemented analyzes here, harmonic load frequency value equals to 5 Hertz. For determining $\zeta$ , $\alpha$ and $\beta$ values, which represent the loading cycles' number, we could determine the soil structure response for harmonic loads with $\zeta$ , $\alpha$ and $\beta$ values to determine proper values for those parameters.

Considering the fact that each record is different in frequency content, magnitude, and effective time and … viewpoints; therefore, maximum displacements of reinforced soil structure would not be similar, as well.

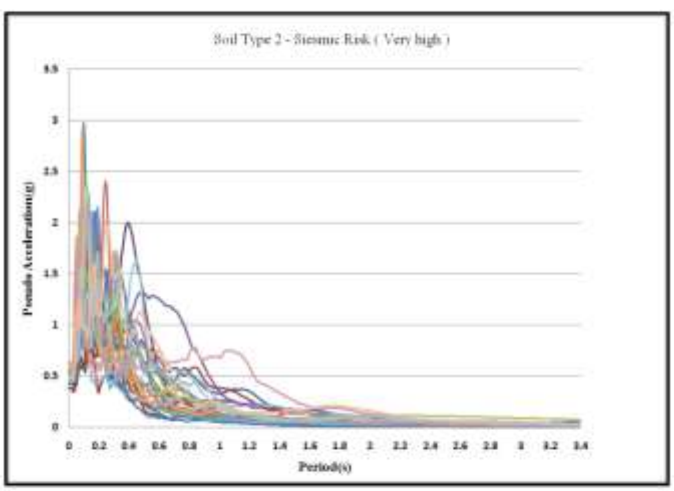



*Fig 5: Single degree of freedom structure's response spectrum for 30 records*

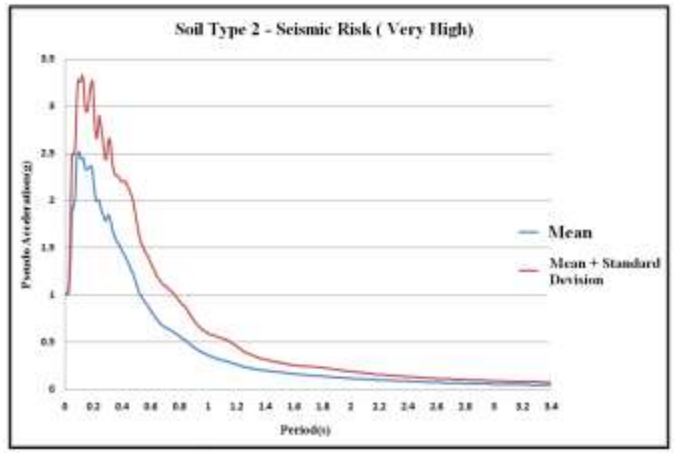



*Fig 6: Mean response spectrums and standard deviations for 30 records*

Results of maximum displacement of structure for CAV parameter for 15 records are illustrated in fig. 7. CAV parameter is proportional to magnitude and is utilized in estimating the seismic damage value which is demonstrated in eq. 6.

$$CAV = \int_0^t |a(t)| dt \qquad (6)$$

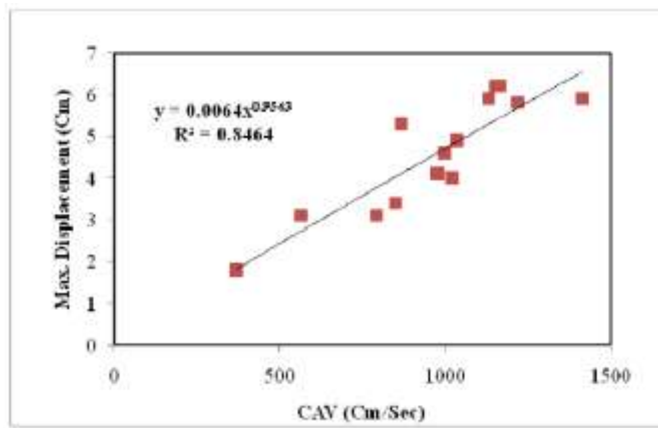



*Fig 7: Maximum displacement variations with CAV parameter*

As shown in fig. 6, the maximum residual displacement in reinforced soil has a good relationship with CAV parameter. Therefore, earthquakes are divided into two categories, with regard to CAV parameter for each soil type. With this classification, scattering and variations of structure displacement would attenuate. Therefore, by considering the mean displacements for each category, we would be able to obtain harmonic loads by trial and error process, which leads to the average displacement for each category.

In this way, two harmonic loads for 30 records would be selected. So, harmonic load 3 represents lesser CAV parameters and earthquake magnitudes and harmonic 4 represents greater CAV parameters earthquake magnitudes. Properties of these parameters, resulted from harmonic loads, are listed in table 3. In addition to this, the shapes of harmonic loads, which are normalized to acceleration, are illustrated in fig. 8 and9.

Table 3: Properties of Chosen Harmonic Loads

| Soil type | Harmonic load | $\alpha$ | $\beta$ | $\xi$ | $f$ |
|---|---|---|---|---|---|
| 2 | H-3 | 5 | 5.75 | 11.8 | 5 |
| | H-4 | 3.1 | 0.02 | 11.8 | |

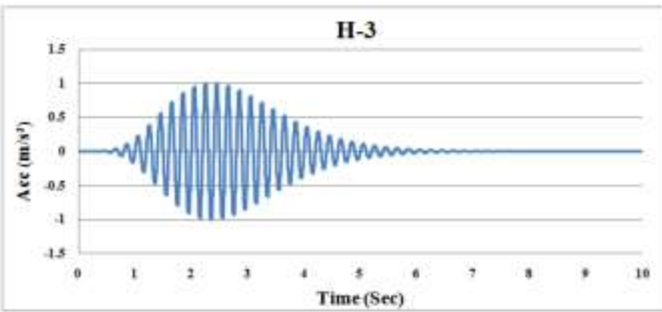



*Fig 8: Harmonic load 3*

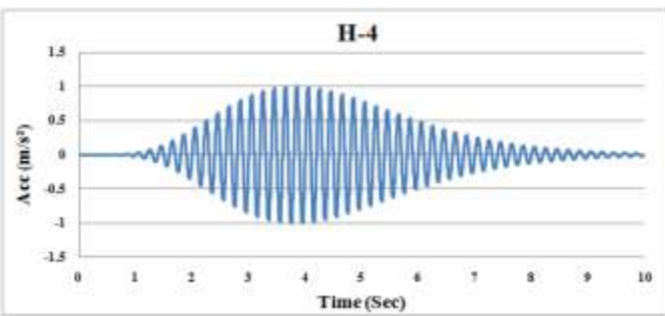



*Fig 9: Harmonic load 4*

2-3- Dynamic analysis

The established model is verified with results of Richardson and Lee (1975) tests and then applied for dynamic analysis. Effects of soil type, length of reinforcing element and earthquake horizontal acceleration of lateral displacement of wall is verified in the dynamic analysis process here. After accomplishment of static analysis and reaching to equilibrium for reinforced soil system, acceleration is applied to the foundation level and the dynamic analysis would be then performed.

In this study, earthquake acceleration is determined on the basis of analysis on sediment seismic response and also considered magnification value for various soil types. Therefore, 3 levels of acceleration, which are representatives of regions with very high, high and moderate seismic risk, are considered here. (Table 4)

Table 4: Maximum resulted acceleration for various zones

| Seismic Risk | Very High | High | Moderate |
|---|---|---|---|
| Maximum Acceleration Level (g) | 0.5 | 0.44 | 0.31 |

DISCUSSIONS

1- Displacements

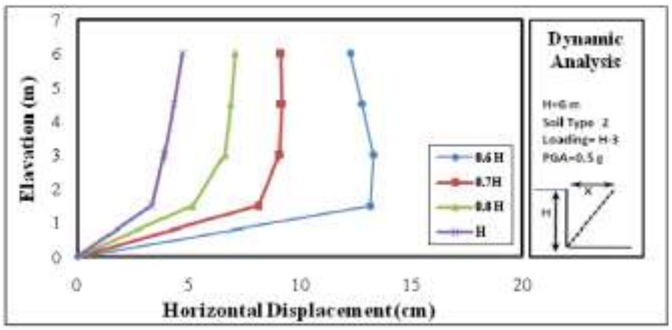
As a sample, surface displacements for a 6-meter high structure are illustrated in fig. 10. Deduced displacements from H-4 loading are averagely 75 percent more than H-3 loading. This difference would be even more in high levels of acceleration and less in lower accelerations. By increasing the acceleration level from moderate risk to very high risk, maximum of displacements would be progressively increased (fig. 11). This viewpoint is clarified for one specific acceleration level and also clear for attenuation in strip length, which means that the maximum of residual displacements in one specific acceleration level would progressively increased by reducing the strip length.

Theoretically, residual displacements are produced in structure, when the acceleration value exceeds $K_y$ . This would be true in numerical analysis, as well, but the difference is that an exact value for $K_y$ 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 $K_y$, 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 $K_y$. 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.

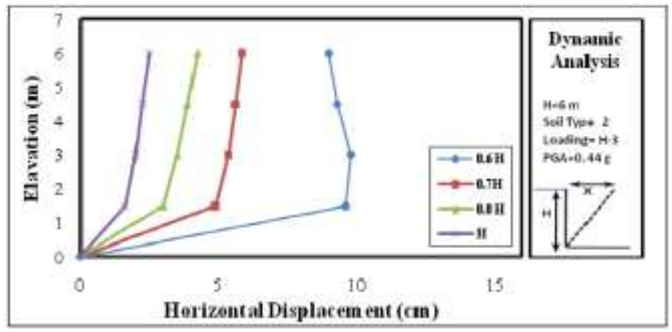
Another important parameter, which determines displacement mode includes facing, facing stiffness, facing height and the connection technique of shells to each other.

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.

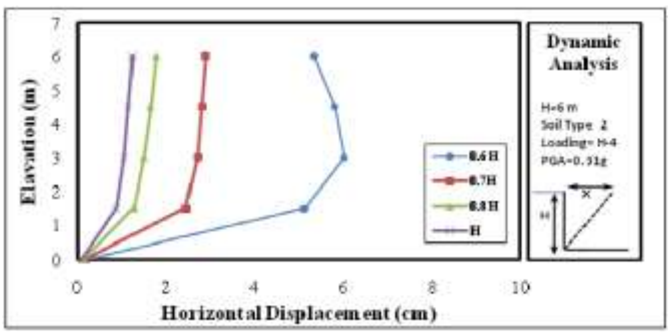
On the other hand, increasing the height would result in length enhancement of the strips. As shown here, normalized displacement values would decrease by increasing the height of the structure. This point infers that for reaching to similar behavior, we could increase the strip length, nonlinearly. 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.



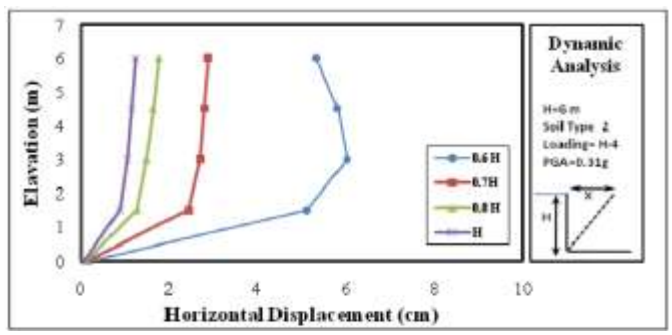
Facing displacement: 6-meter wall height, very high acceleration level (H-3)



Facing displacement: 6-meter wall height, high acceleration level (H-3)

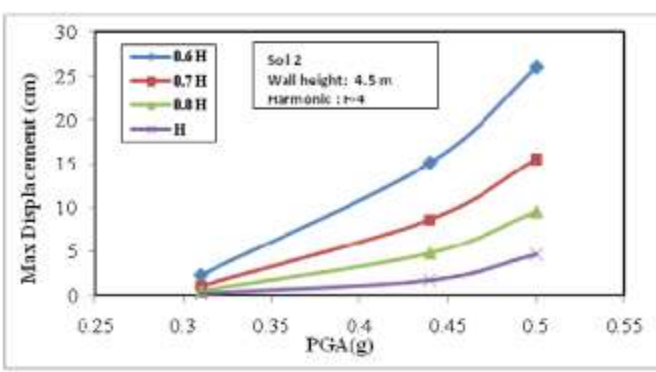


Facing displacement:6-meter wall height , moderate acceleration level (H-4)

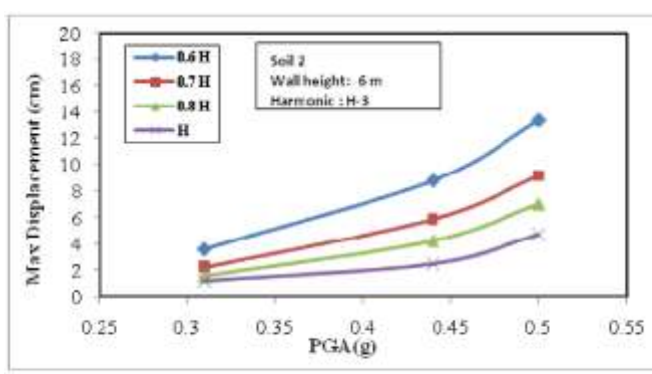


Facing displacement : 6-meter wall height , high acceleration level (H-4)

*Figure 10: seismic performance of reinforced soil wall in horizontal facing displacements*



Maximum displacement : Ground maximum acceleration 2 , 4.5-meter wall height (H-4)



Maximum displacement : Ground maximum acceleration 2 , 6-meter wall height (H-3)

*Figure 11: variations in displacement of reinforced soil structures versus maximum accelerations*

2- History of displacement

As a historical paradigm, displacement of two points of a wall versus time is illustrated in fig. 12. It is observed that the displacement initiates from an onset point and after fluctuation, it teaches to a constant value. In this way, a residual displacement would be produced in the structure. This process cites that the wall has reached to a sort of stability at the end of the analysis process, inasmuch as the displacement has become constant and the system converged. Firstly, by initiating the loading process, the displacement value varies cyclically. Low acceleration levels in analysis, results in no residual displacement in system. Therefore, by increasing the acceleration level in each loading cycle, residual displacements would appear in the system.

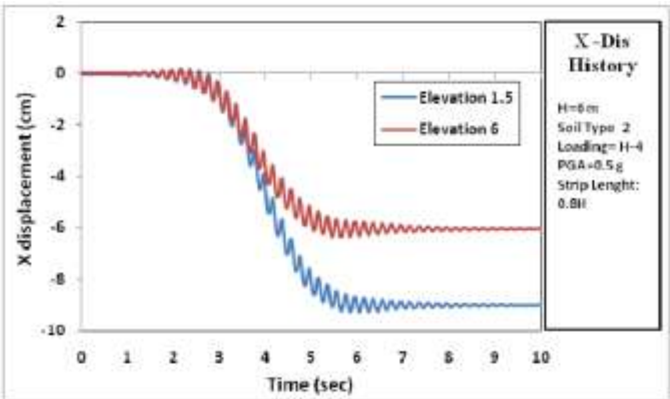



*Figure 12: displacement history for 6-meter height, (H-4) high acceleration level, and 0.8H strip length*

3- Displacement-based determination of pseudo static coefficient

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 acceleration coefficient. In order to determine the horizontal acceleration coefficient related to any reinforced soil structure, based on design methods of AASHTO and FHWA codes, a programming is carried out in Excel software and the horizontal acceleration coefficient related to the length of each strip is determined, accounting for the safety factors. after determining this horizontal acceleration coefficient, one could express the horizontal acceleration coefficient as a function of height, dynamic loading type, applied maximum and displacements formed in the structure.
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.

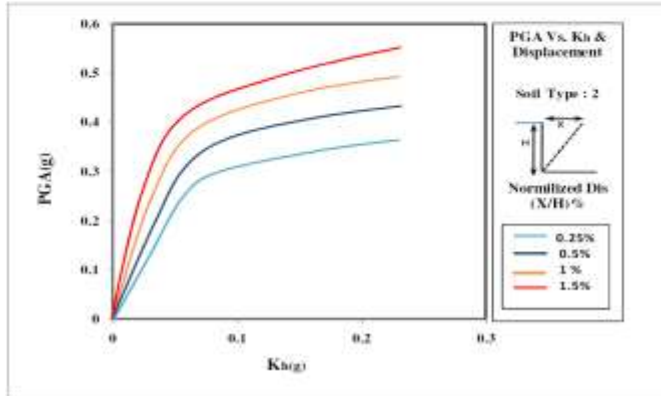



*Figure 13: Horizontal acceleration coefficient on the basis of structure's displacement and maximum acceleration*

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 and decreasing the length of reinforcing elements, displacements grow consequently. By decreasing the stiffness, displacement mode would also tend to be in convex pattern. Implementing numerical investigations, horizontal acceleration coefficient values are determined, based on various performance levels and then compared whit values cited in FHWA. 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, displacement-based design concept would result in more suitable and economical structure.

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