Numerical study of seismic behavior of reinforced soil bridge abutments located on soft clays of Southwest Iran

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ABSTRACT: This paper investigates the seismic response of several reinforced soil abutments of railway overpass bridges constructed in southwest of Iran. Due to high seismicity and soft to medium stiff clayey deposits in the region, evaluating the stability and deformation behavior of these abutments against probable earthquakes were significant. The reinforcing elements are uniaxial extruded geogrids and reinforced soil is composed of granular material.
A 2-D, explicit finite difference scheme is used to perform the dynamic analyses of a single-span reinforced soil abutment with a wrap-around facing, making use of hysteretic damping feature of the numerical model. The seismic response of an instrumented reduced-scale reinforced soil retaining wall tested on a shaking table is used for numerical modeling verification purposes. The comparisons between predicted results from the numerical model and the measurements of the physical model have shown good agreement.
Parametric studies are performed to investigate the effects of system parameters on the abutment seismic behavior. The results show that characteristics of foundation and reinforced soil, stiffness and length of geogrids and also input acceleration amplitudes are having significant impact on the maximum horizontal displacement and maximum vertical displacement of bridge deck footing. The overall response of the reinforced soil abutment subjected to seismic excitations is evaluated and discussed.

Keywords: Reinforced bridge abutment, seismic response, geogrid, numerical model, dynamic analysis

1 INTRODUCTION

Reinforced earth walls have gained numerous applications in geotechnical engineering. Reinforced bridge abutments are similar to earth walls except that they carry a surcharge load near the top edge (from the bridge deck) in addition to the soil horizontal pressure behind the reinforced earth zone. Since bridges are among the significant infrastructures, they are expected to remain in service after earthquakes.

Finite-difference-based numerical models have been widely used to predict the performance of reinforced soil walls. Bathurst and Hatami (1998) used FLAC 2D to study the effect of reinforcement specifications (such as length and stiffness) on the response of reinforced earth walls subjected to seismic excitations. They found that both reinforcement forces and facing displacement increase over time and the maximum reinforcement load occurs at the facing connections.

The extensive damage of highway bridges in the recent earthquakes have led to significant advances in bridge seismic design. In reinforced bridge abutments, a significant case study project was the fully instrumented segmental bridge abutment constructed and monitored during and after construction in Denver, Colorado, named Founders/Meadows (Abu-Hejleh et al., 2000, 2001).

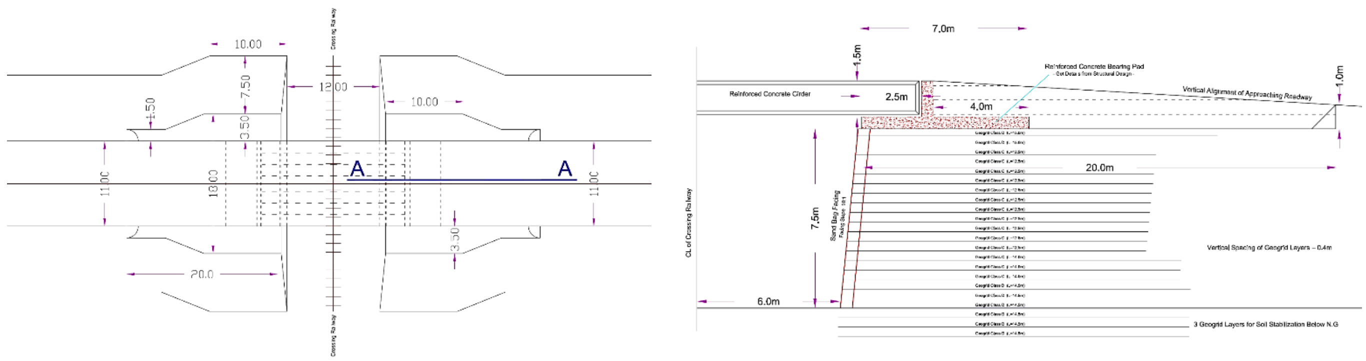
Fakharian and Attar (2005, 2007) used FLAC2D v4.0 to simulate the dynamic response of the Founders/Meadows reinforced bridge abutments to seismic excitations. The results showed that the facing horizontal displacements increase with number of cycles and reach values at the end of seismic excitations considerably greater than static magnitudes. The vertical displacement and rotation of the bridge deck footing, however, were not significant. Fakharian and Aghania (2014) continued the study on Founders/Meadows reinforced bridge abutment, incorporating the deck inertia and its interaction with the footing and abutment. One of the instabilities observed in their model was the excessive rotation of the bridge deck footing caused by cyclic horizontal loads, which is exerted from bridge deck to the abutment footing. The other instability observed was the backward rotation of upper facing segmental blocks of the abutment facing.

Helwany et al. (2012) published a report on seismic designing of the reinforced bridge abutments both with numerical modeling using the finite element software Abaqus (2002) and physical modeling of the bridge system, and finally compared the results. They also modeled the elastomeric bearing pad as a base isolator under the bridge deck. Fakharian and Nasrolahzadeh (2017) investigated the influence of base isolators between bridge deck and deck footing in Founders/Meadows reinforced bridge abutments subjected to seismic excitations and realized they significantly contribute to reduce the excessive rotation of the deck footing and instabilities of upper blocks of footing as reported by Fakharian and Aghania (2014).

This paper investigates the behavior of reinforced soil abutments of single-span overpass railway bridges under seismic excitations. The reinforced soil abutment is one of the overpass bridges having a single-span of 19 m and 7 m abutment height constructed in Khuzestan Province in southwestern Iran. Foundation soil type was soft to medium stiff clayey deposit with high groundwater level. FLAC 2D was used to simulate the seismic excitations. Post-cyclic facing horizontal displacements and bridge deck footing displacements are presented and discussed first, followed by parametric studies of the abutment seismic behavior.

2 OVERPASS BRIDGE ABUTMENTS OF SOUTHWEST IRAN

The reinforced soil abutments of single-span overpass railway bridges were completed near Ahvaz-Khorramshahr highway located southwest Iran, to accommodate two traffic lanes and two sidewalks. Figures 1a and 1b show a plan view and a typical section of the reinforced abutment. It is observed that the bridge deck/girder load is carried by a strip footing on top of the reinforced abutment. Hence bridge loads are transferred directly to the reinforced soil zone.



(a) (b)

Figure 1. Overpass railway abutments: (a) Plan view (b) Typical section of the reinforced abutment

The facing was constructed using wrap-around geogrids in combination with sand bags. The reinforcing geogrids (HDPE) were placed with uniform spacing but varying length, as illustrated in Figure 1b. The foundation soil consists of soft to medium stiff clayey deposits and not improved before construction process. Groundwater level is reported at 2 meters depth. The retained and reinforced zones were constructed using granular material.

2.1 *Numerical model*

Figure 2 shows the geometry, boundary conditions, loading, reinforcement arrangement, interface elements, and finite difference grid for numerical simulation of abutment. The total abutment height is 9 m, including 7.5 m of exposed facing and 1.5 m of buried wall height. The total width of the backfill is taken 75 m far from the deck end to minimize interactive boundary effects on the system response. The staged construction is considered in the model. The foundation soil was generated and settled under self-weight to satisfy the initial equilibrium, before starting the placement of reinforced soil larger by larger. Interface elements are added between foundation soil and the reinforced soil.

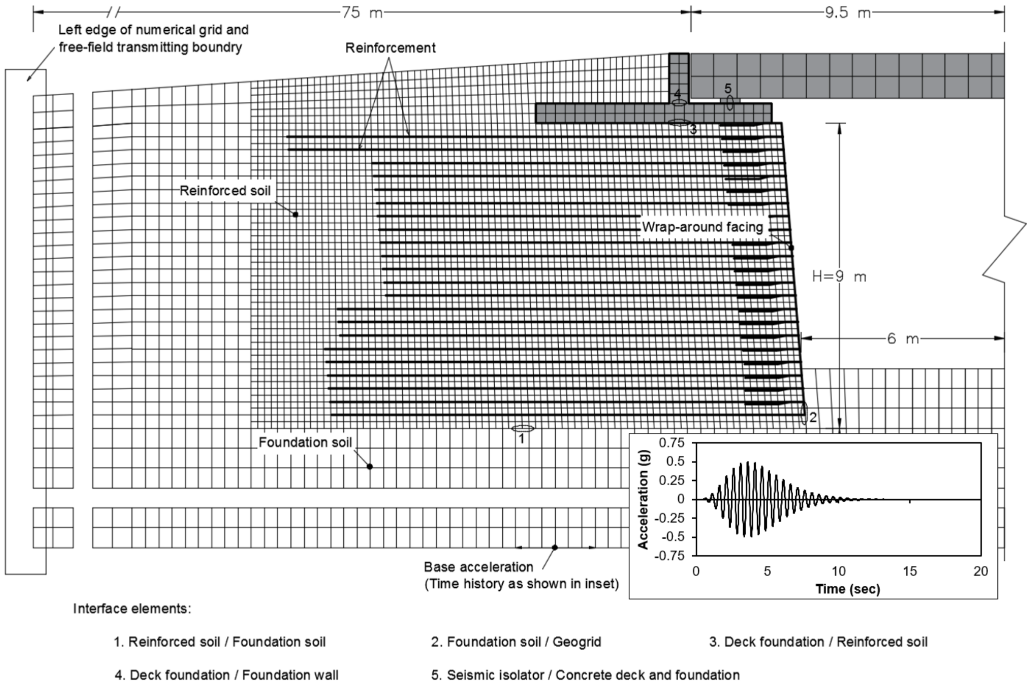


Figure 2. Grid, interface elements, boundary conditions and seismic loading of the numerical model

The initial condition for the seismic analysis is the static stability of the entire system that is maintaining the initial stresses while all the deformations are reset to zero. The grid geometry is the same throughout the static and seismic analyses (Figure 2), but the boundary conditions and strees-strain relations are different.

The granular soil model is nonlinear elastic (hyperbolic model) with M-C (Mohr-Coulomb) failure criterion under static condition, but softening effects after failure are also considered. To improve the model results, a hyperbolic stress–strain model has been used to simulate the nonlinear stress–strain behavior of many soils prior to failure (Duncan et al. 1980). This model has been verified in several numerical studies related to reinforced soil structures (e.g. Fakharian and Attar (2007); El-Emam and Bathurst (2004, 2005)).

Soil parameter values for the static model are reported using large-scale triaxial tests for reinforced and retained soil, and in-situ tests for foundation soil. Soil properties are summarized in Table 1.

Table 1. Soil parameter magnitudes for static analysis

Soil zone	Duncan/ Mohr- Coulomb parameters									
	γ (kN/m^3)	γ' (kN/m^3)	φ' (°)	c' (kPa)	K	n	R_f	p_a (kPa)	υ	ψ (°)
Reinforced and Retained soil	22.1	-	35	0	800	0.7	0.8	100	0.35	5
Foundation Soil	18.0	10.0	24	10	-	-	-	-	0.40	-

In the dynamic analysis, the loading/unloading nonlinear hysteretic Masing rule was used to model the shear behavior of the soil, and to capture soil damping. This model has been used by Cai and Bathurst (1995) for dynamic finite element analysis of segmental retaining walls and by Fakharian and Attar (2005) to model the seismic response of segmental bridge abutments.

The reinforcement geogrids are modeled by the elastoplastic cable elements in FLAC without compressive strength.The frictional behavior between soil and geogrid was simulated by assuming a grout layer thickness of zero, an interface soil/geogrid friction angle of 0.75φ, and zero cohesion. The reinforcement parameters used in the analysis are presented in Table 2.

Maximum allowable axial reinforcement forces are shown in Table 2. For simplicity, geogrid elastic modulus is calculated using secant method at the point of 5% strain using the tensile stress-strain graphs provided by the manufacturer.

Table 2. Geogrid parameters

Geogrid type	GT-1	GT-2
Long term design strength (kN/m)	42.4	56.5

A variable-amplitude sinusoidal harmonic motion (Figure 2) is used as an input at the bottom nodes of foundation soil for seismic excitations according to Eq. 1.

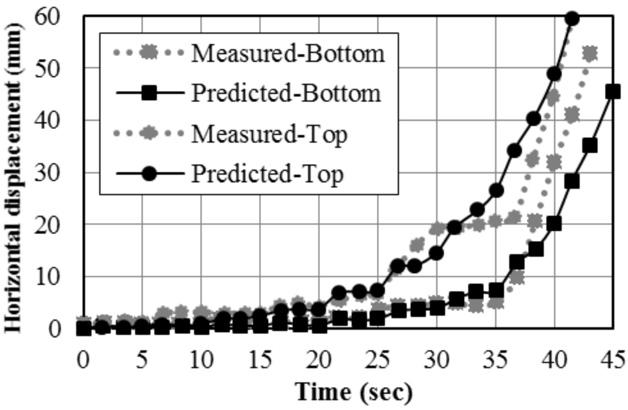
$$\ddot{u}(t) = k/2 \times \sqrt{\beta\, e^{-\alpha t}\, t^{\xi}}\, \sin(2\pi f\, t) \tag{1}$$

Where: α=2.2, β=0.39, ζ=8 are constant coefficients; f=frequency, t=time, k=peak amplitude of the input acceleration assumed as 0.5g in base model and 0.1g-0.6g in parametric models, and frequency, f=2 Hz. The time, t, varies between 0 and 20 seconds.

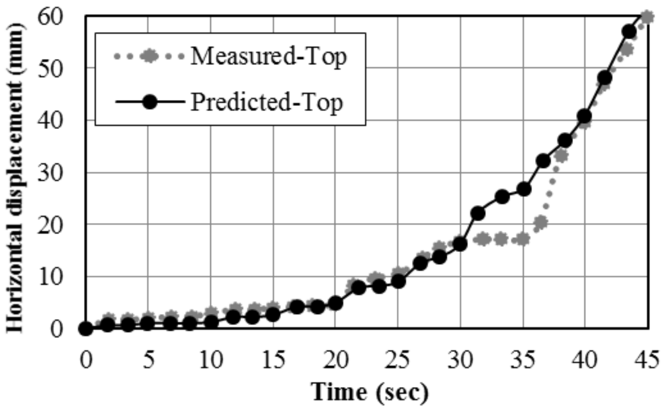
2.2 *Model verification*

FLAC has been used successfully to model reinforced soil walls under both static and seismic conditions (e.g. Hatami and Bathurst 2001, 2005, 2006; El-Emam et al. 2004). In this paper, the results of a well-documented/instrumented reduced-scale model shaking test of a reinforced wall system reported by El-Emam and Bathurst (2004, 2005) are used. El-Emam and Bathurst (2004, 2005) presented the results of one-sixth scale model shaking table tests for seismic response of geogrid-reinforced walls. To study the effect of facing toe condition on the model seismic response, the tests were conducted with hinged and sliding toe configurations. Numerical modeling details are available in the papers by Fakharian and Attar (2007).

Figure 3a presents measured results from the physical model and the numerical model predictions for the time histories of horizontal displacement above the top and bottom of the wall face for the sliding toe condition. A similar comparison is made for the horizontal displacement of the top of the wall face for the hinged base condition, as shown in Figure 3b. Good agreement is observed between the measured and predicted results.



(a)



(b)

Figure 3. Measured and predicted lateral displacement of shaking table tests: (a) Sliding toe (b) Hinged toe

3 RESULTS

3.1 *Dynamic and static analysis results of base model*

In order to investigate the actual behavior of Iran railway overpass abutments at the end of construction and after seismic excitations, modeling of the system with specifications mentioned in the previous sections was carried out under both static and dynamic conditions.

Figure 4 shows the horizontal displacement profiles of facing at the end of construction, at the end of dynamic analysis and also dynamic net displacement. The general deformation form at the end of static analysis is having a bulging pattern and the maximum horizontal displacement occurs at the elevation of

0.55H; results also show that as the abutment height increases, horizontal displacements have decreased as well; horizontal displacement at elevation of 0 to 1.5 meter is limited by the soil of middle area. The net horizontal deformation pattern at the end of dynamic analysis is almost linear. According to Figure 5 showing displacement vectors at the end of dynamic analysis, the reinforced soil mass has rotated counter-clockwisely on the potential failure surface (in the form of an arc extending from the rear of reinforced zone to abutment toe).

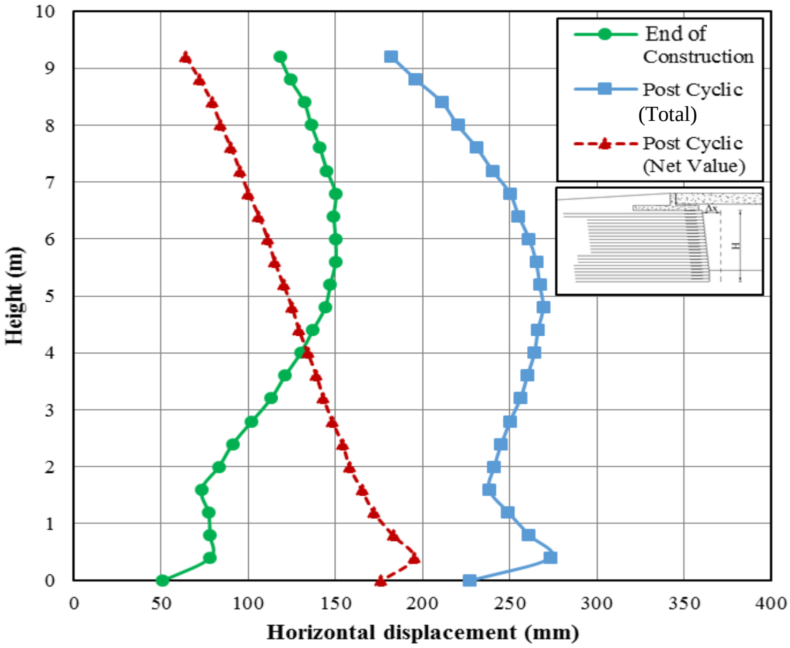


Figure 4. Facing displacement profile before and after seismic excitations

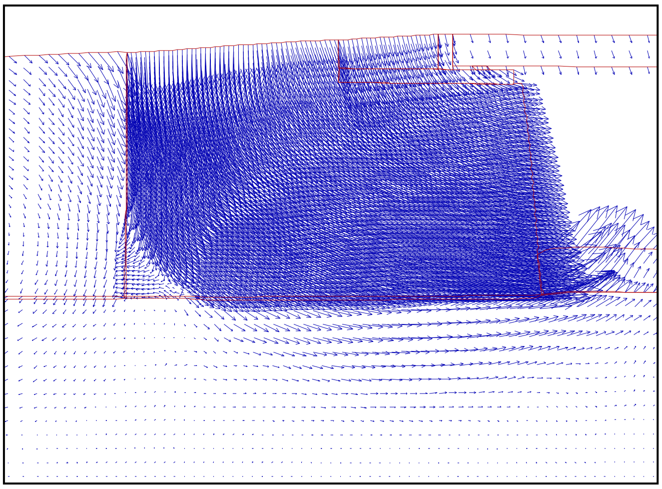


Figure 5. Displacement vectors at the end of dynamic analysis and rotation pattern

Figure 6 shows the vertical displacement of the bridge deck footing at the end of static and dynamic analyses and also net dynamic displacement. As it is seen, due to the eccentricity of deck load in static condition, the bridge deck footing has a clockwise rotation at the end of static analysis; However, by applying the seismic wave, as the net dynamic displacements show (Figure 6), the bridge deck footing is subjected to a counterclockwise rotation. This is because by increasing the horizontal displacement of the facing due to the applied harmonic acceleration, the bridge deck footing also tends to move in the same horizontal direction; but the distance between edge of the deck and edge of the bridge deck footing wall is limited and moves to zero during dynamic analysis. Therefore, from that moment, the bridge deck footing wall prevents the deck movement, and consequently, the subsequent sequential cycles of the earthquake causes counterclockwise rotation of the bridge deck footing.

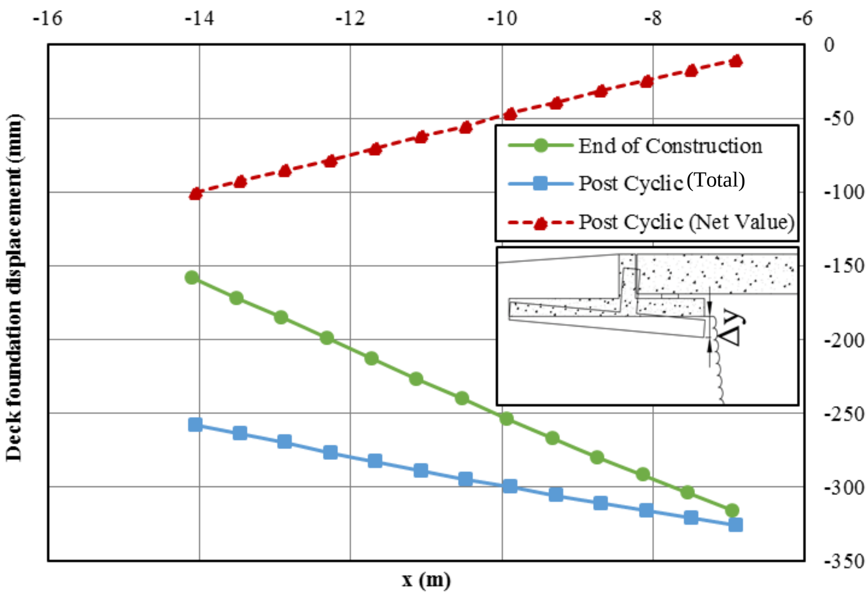


Figure 6. Bridge deck footing vertical displacements and rotation

3.2 *Parametric analysis results*

Parametric studies have been carried out to investigate the effect of different characteristics and parameters on the performance of abutment system and its deck. Foundation soil strength (c', φ'), retained and reinforced soil strength (φ), stiffness and length of geogrids are the main parameters selected for the sensitivity study according to Table 3.

Table 3. Parameters and range of variations in parametric study

Parameter	Range of variations			
Foundation soil strength	c'=5 (kPa) , φ'=18 (°)	c'=10 (kPa) , φ'=24 (°)		c'=15 (kPa) , φ'=30 (°)
Retained and reinforced soil strength	φ=30 (°)	φ=35 (°)		φ=40 (°)
Geogrid average stiffness	J=600 (kN/m)	J=900 (kN/m)	J=1300 (kN/m)	J=1600 (kN/m)
Geogrid average length	L_0/H=1.2	L_0/H=1.5		L_0/H=1.7

In order to investigate the effect of parameters on seismic behavior of the system, maximum horizontal displacement of facing and maximum vertical displacement of bridge deck footing are compared during dynamic stimulation in models influenced by maximum acceleration from 0.1 to 0.6 g.

3.2.1 *Horizontal displacement of facing*

The location of maximum horizontal displacement of facing in static analysis is in mid height of facing wall. However, in dynamic analysis as a result of the sismic wave, maximum horizontal displacement of facing occurs at the lowest point of the elevation, because of reasons explained in the previous section.

As shown in Figure 7, generally as the maximum acceleration increases, maximum facing displacement also has increased. According to Figure 7a, with increase of effective cohesion and internal friction angle of the foundation soil, amount of horizontal displacement of facing decreases. According to Figure 7b, increasing the friction angle of retained and reinforced soils has reduced horizontal displacement of facing, meaning that increasing the soil density during construction process will reduce facing displacement. Figure 7c shows that increasing the stiffness and strength of geogrids to a certain extent will reduce the displacement, but further stiffening of geogrids will not help reducing horizontal displacements. As shown in Figure 7d, length of geogrids (according to the studied range) does not have significant influence on the facing horizontal displacement.

3.2.2 *Displacement of bridge deck footing*

Dynamic analysis results show that the bridge deck footing has a counter-clockwise rotation at the end of dynamic analysis. As shown in Figure 8, generally, as maximum acceleration increases, maximum vertical displacement of bridge deck footing also increases. Figure 8a shows that with increase of effective cohesion and internal friction angle of the foundation soil, magnitude of bridge deck footing displacement decreases. According to Figure 8b, increasing the friction angle of retained and reinforced soil reduces displacement of bridge deck footing, meaning that increasing the soil density during construction process will reduce the bridge deck footing displacement. Figure 8c shows that increasing the stiffness and strength of geogrids will reduce bridge deck footing vertical displacements. According to Figure 8d, at accelerations between 0.1g and 0.4g, displacement of the deck footing has reduced with increasing length of geogrids, but at accelerations greater than 0.4g, increasing the length of geogrids did not significantly affect the reduction of displacements.

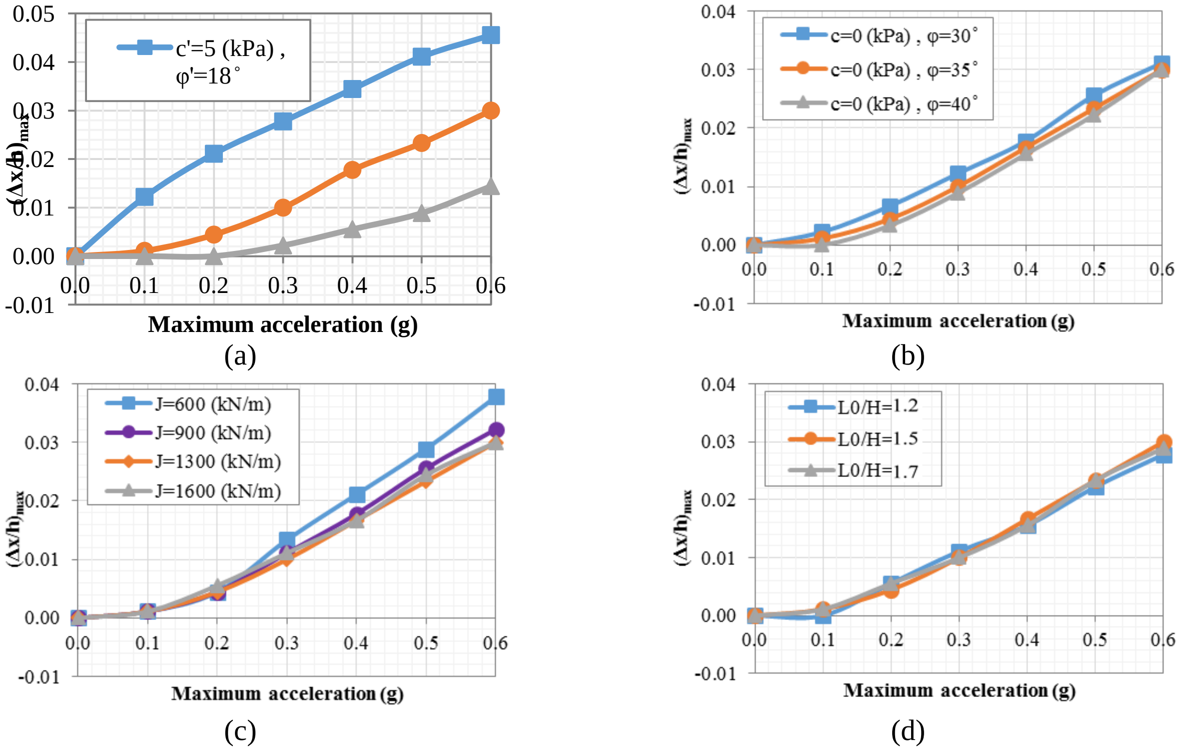


Figure 7. Normalized maximum horizontal displacement of facing (Δx/H)

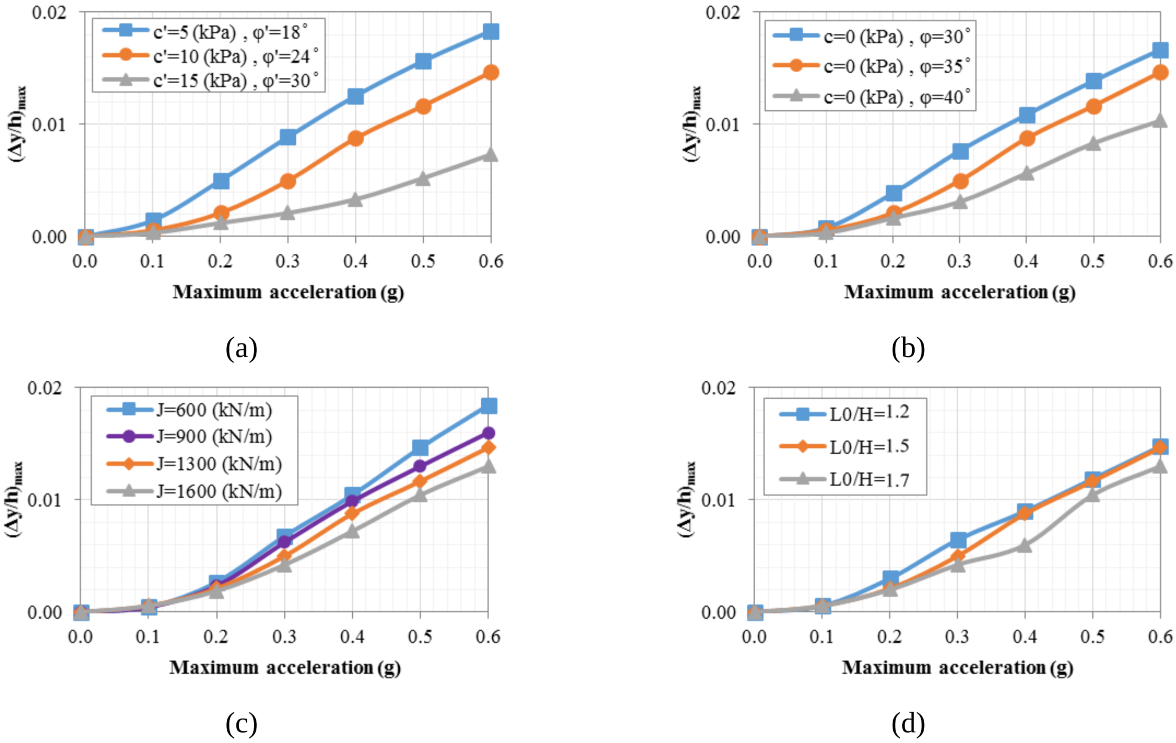


Figure 8. Normalized maximum vertical displacement of bridge deck footing (Δy/H)

4 DISCUSSION

More detailed studies presented in Kashkooli (2017) has shown that the natural frequency of the entire system is 0.5 Hz. At such a frequency, both the horizontal deformation of the facing as well as vertical deformation and rotation of the deck footing were significantly higher. For example, at such a frequency, the maximum horizontal displacement reached 0.07H at the end of seismic excitations which is too high and way beyond acceptable magnitudes. The frequency of 2 Hz selected in this paper is believed to be close to the dominant frequency of most earthquakes and on the basis of presented results, one can judge on the performance of the constructed overpass single-span bridges.

The sliding pattern of Figure 5 shows that for the constructed reinforced abutments, deep rotational failure is likely to occur. In other words, the internal instability is not likely and during strong excitations the external instability could be threatening the abutments. This is probably due to the soft to medium stiff condition of the foundation soil. To overcome such an instability mode, the foundation soil could have been improved down to few meters. Results on effect of foundation soil improvement of reinforced abutments are presented in Fakharian and Attar (2006). Further studies showed however, that the constructed overpass bridges are safe enough against the probable earthquake of the region.

The results of Figures 7a and 8a have shown that the foundation soil properties is significantly influencing both facing wall deformation as well as vertical displacement and rotation of the deck footing. Therefore, other than stability issues that was pointed out in the previous paragraph, the foundation soil improvement prior to construction of the abutments could improve the serviceability performance of entire system as well.

More studies on this subject are underway to evaluate the effect of length of deck footing, frequency and resonance, amplitude at different elevations of the system and also real earthquake records instead of the harmonic excitations, the results of which shall be published soon.

5 CONCLUSIONS

In this paper, modeling results of two reinforced soil abutments and bridge deck were presented under static and dynamic conditions. Parametric studies were carried out to understand the effect of foundation soil strength, reinforced and retained soil strength, length and stiffness of geogrids and also the maximum amplitude of input acceleration, on the system seismic behavior.

The most important findings are summarized below:

1- The results at the end of construction showed that maximum horizontal deformation of facing occurs at the midpoint of height. Due to the eccentricity of deck load in static condition, the bridge deck footing has a clockwise rotation at the end of construction.
2- The maximum horizontal deformation occurs at the lowest elevation of facing height in dynamic condition, and the horizontal displacement pattern is almost linear in height, due to overall soil mass counter-clockwise movement on potential failure surface. During dynamic analysis, the deck has tendency to turn in a direction opposite of the movement of the facing.
3- Foundation soil strength, retained and reinforced soil and stiffness of geogrids have significant influences on maximum horizontal deformation of facing. The increase in reinforcement length in studied range does not have a significant effect on maximum horizontal deformation. Generally, with increasing of maximum acceleration amplitude, the maximum horizontal facing displacement increases.
4- Foundation soil strength, retained and reinforced soil strength, stiffness and length of geogrids influence the maximum bridge deck footing displacement. Generally, with increasing of maximum acceleration amplitude, the maximum bridge deck footing displacement increases.
5- The dynamic analysis results show that despite existence of soft to medium clayey deposits of foundation soil with high groundwater level, the railroad overpass bridges are stable during dynamic analysis. However, excessive facing deformation and bridge deck footing displacement should be considered for design purposes.

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