Title

Numerical Study of the Compaction Effect on the Static Behavior of a Geosynthetic Reinforced Soil-Integrated Bridge System

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Experimental Design for Half-Scale Shaking Table Test of a Geosynthetic-Reinforced Soil Bridge Abutment

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ABSTRACT

This paper presents an experimental study on the seismic performance of a half-scale geosynthetic-reinforced soil (GRS) bridge abutment. Experimental design of the scale model followed established similitude relationships for shaking table testing in a 1 g gravitational field. This involved scaling of model geometry, reinforcement and backfill stiffness, bridge load, and characteristics of the earthquake motions. The GRS abutment was constructed using well-graded sand, modular facing blocks, and uniaxial geogrid reinforcements with a vertical spacing of 0.15 m in both the longitudinal and transverse directions. The bridge deck was placed on the GRS abutment at one end and supported by a concrete wall resting on a sliding platform off the shaking table at the other end. The table was connected to the base of the support wall with steel beams to transmit the table motions. The measured lateral facing displacements and bridge seat settlements during application of a series of earthquake motions in the longitudinal direction are presented and indicate good seismic performance of the GRS bridge abutment.

INTRODUCTION

In recent years, geosynthetic-reinforced soil (GRS) retaining walls have been used as bridge abutments with the bridge load applied to the top of a reinforced soil mass through a shallow footing. GRS bridge abutments have many advantages over traditional pile-supported designs and are being commonly used in the United States. In-service GRS bridge abutments to date have shown good performance under static loading in terms of lateral facing displacements, bridge seat settlements, and differential settlement between the bridge and approach roadway (Abu-Hejleh et al. 2002; Lee and Wu 2004; Adams et al. 2011). However, there are still concerns regarding the use of GRS bridge abutments in high seismicity zones like California.

Extensive experimental studies have been conducted to investigate the static response of GRS abutments under surcharge loads (Adams 1997; Ketchart and Wu 1997; Wu et al. 2001, 2006; Helwany et al. 2007; Nicks et al. 2013, 2016). However, less experimental data is available for the seismic response of GRS walls and abutments. El-Emam and Bathurst (2004; 2005; 2007) performed a series of fourteen shaking table tests on 1/6-scale GRS walls having a full-height rigid facing panel to investigate the influences of facing panel mass, inclination angle and toe condition, and reinforcement length, spacing and stiffness on the seismic response of GRS walls. Ling et al. (2005, 2012) reported a series of large-scale shaking table tests for 2.8 m high GRS walls having modular block facing using both sand and silty sand as backfill soils. Results indicate that increasing reinforcement length for top layers and reducing reinforcement vertical spacing could improve seismic performance. The apparent cohesion within the silty sand backfill was also found to contribute to the seismic performance of the walls. Fox et al. (2015) conducted full-scale shaking table tests on a 6.1 m high GRS wall with modular block facing, and found that the residual lateral facing displacements were relatively small under seismic loading. Helwany et al. (2012) reported the only shaking table test for a GRS abutment and found the structure could sustain sinusoidal motions with horizontal accelerations up to 1 g without significant distress.

More experimental investigations are needed to further understand the seismic response of GRS bridge abutments before this technology can be adopted in high seismicity zones. Accordingly, this paper presents an experimental study of the seismic performance of a half-scale GRS bridge abutment using shaking table. The similitude relationships for the model are described first, and then model configuration, instrumentation, and input motions are presented. Preliminary results, including lateral facing displacements and bridge seat settlements, are presented and discussed.

SIMILITUDE

Reduced-scale model tests provide a more economical option than full-scale tests and have been widely used in geotechnical engineering to investigate the behavior of complex systems. For shaking table tests in a 1 g gravitational field, a proper similitude that describes the relationships between the scale model and prototype structure should be used to design the scale model. In this study, model design follows the similitude relationships proposed by Iai (1989), which are widely used for shaking table tests of GRS walls (El-Emam and Bathurst 2004, 2005, 2007; Guler and Selek 2014). Considering the size and capacity of the shaking table in the UCSD

Powell Structural Research Laboratory, a length scaling factor of λ = 2, defined as the ratio of prototype length to model length, was selected for this study, with the abutment height being the governing parameter. A prototype 5.4 m high GRS abutment with a typical clearance height of 4.5 m was selected. Accordingly, a half-scale GRS abutment model having a height of 2.7 m that has a clearance height of 2.25 m was constructed and tested in this study. The model geometry, reinforcement stiffness, bridge load, and characteristics of the earthquake motions were scaled according to the similitude in Table 1.

Table 1 Similitude for 1 g shaking table testing (Iai 1989).

	Theoretical Scaling Factor	Scaling Factor for λ=2
Length	λ	2
Density	1	1
Strain	1	1
Mass	λ^3	8
Acceleration	1	1
Velocity	$\lambda^{1/2}$	1.414
Stress	λ	2
Modulus	λ	2
Stiffness	λ^2	4
Force	λ^3	8
Time	$\lambda^{1/2}$	1.414
Frequency	$\lambda^{-1/2}$	0.707

HALF-SCALE MODEL DESCRIPTION

Model Configuration

The test setup for the bridge system in the longitudinal direction is shown in Figure 1. The bridge deck has dimensions of 6.4 m × 0.9 m × 0.45 m (length × width × height), and is placed on a bridge seat resting on the GRS abutment at one end and supported on a concrete wall at the other end. This deck represents a longitudinal slice of an actual bridge deck and has reduced length to accommodate the available laboratory space. Elastomeric bearing pads with plan dimensions of 0.45 m × 0.9 m were placed under both ends of the deck. The seismic joint between the bridge deck and back wall of the seat is 25 mm wide. The weight of concrete deck is 65 kN, and additional dead weights (steel plates) of 33 kN were evenly distributed and rigidly attached to the deck to produce the desired scaled bridge load while keeping the center of the mass of the bridge deck relatively low. These dead weights result in an average vertical stress of 121 kPa on the bridge seat. The bridge seat has a weight of 7 kN. Considering the 3D nature of the structure, the average vertical stress over the entire bridge seat bottom surface due to dead weights of bridge seat, deck, and steel plates is 64 kPa. The GRS abutment has a modular block facing on three sides, while a steel reaction frame with plywood supports is used to form the back of the abutment structure. The steel reaction frame was designed to be sufficiently stiff to provide at-rest conditions during compaction and resist the lateral earth pressures during shaking. There is a retained soil zone having a length of 0.63 m between the reinforced soil zone and the steel reaction frame, which helps to minimize boundary effects on the seismic response of the abutment. The total weight of the entire bridge system is approximately 450 kN.

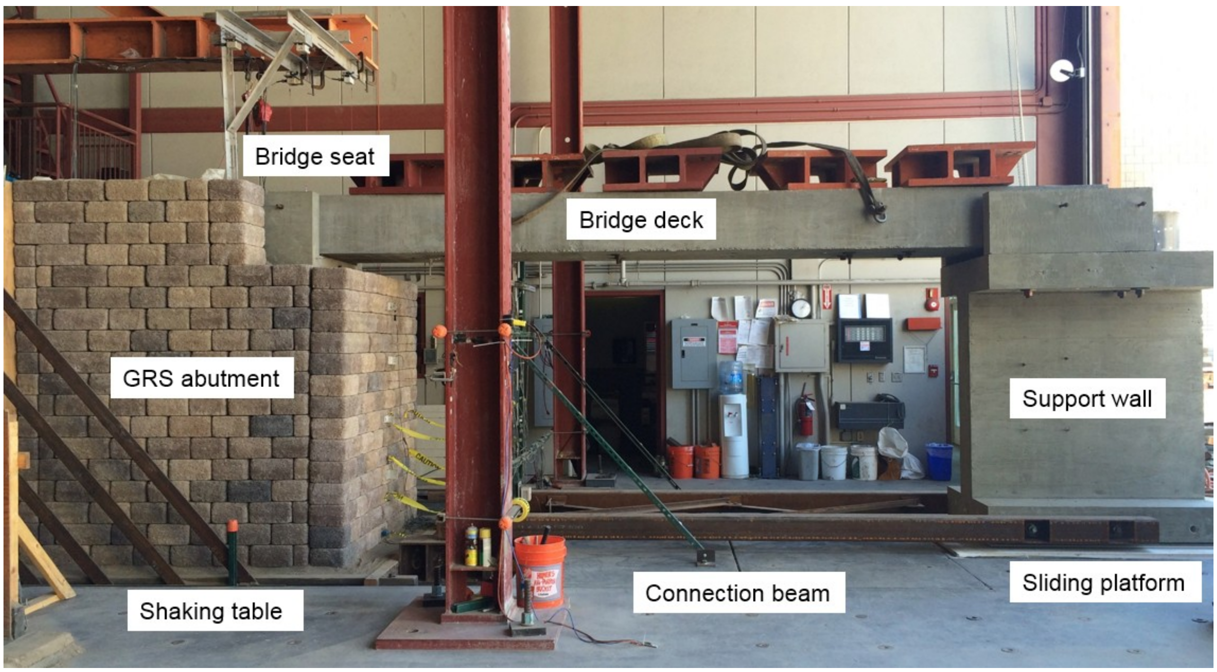


Figure 1. Test setup for bridge system in the longitudinal direction.

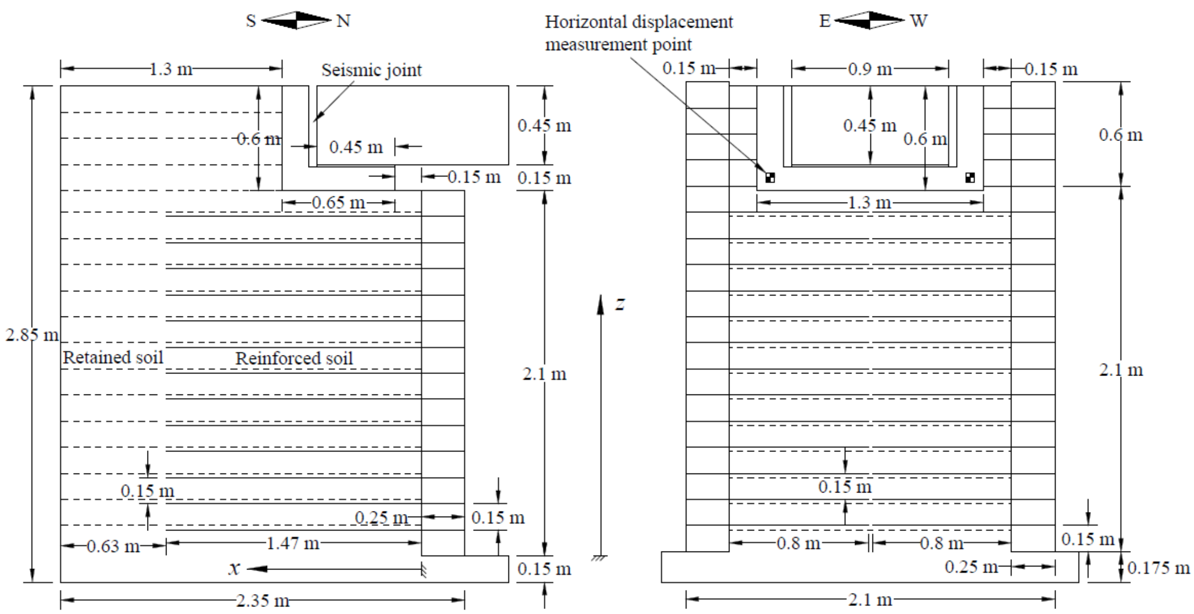
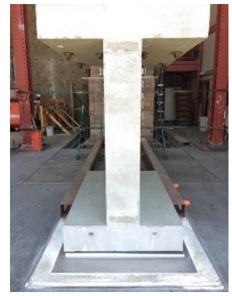


Figure 2. GRS bridge abutment model: (a) longitudinal section; (b) transverse section (reinforcement layers in the perpendicular direction shown as dashed lines).

Cross-sectional views of the GRS bridge abutment in the longitudinal and transverse directions are shown in Figures 2(a) and 2(b), respectively. The model has a total height of 2.7 m, consisting of a 2.1 m-high lower GRS wall and a 0.6 m-high abutment wall, resting on a 0.15 m-thick foundation soil layer placed directly on the shaking table. The lower GRS wall was constructed using 14 soil lifts. Each 0.15 m-thick lift includes one layer of longitudinal reinforcement and transverse reinforcements that extend from either side wall that meet in the center. The transverse reinforcements (as well as the facing blocks) are offset by 25 mm vertically from the longitudinal reinforcements. The retained fill also has transverse reinforcements that are not connected to the transverse reinforcements in the reinforced zone. The longitudinal reinforcements are 1.47 m long and the transverse reinforcements extend inward 0.8 m from each of the side walls.

The bridge support wall rests on a sliding platform shown in Figure 3. Based on a concept used by Fox et al. (1997, 2006), this platform consists of 273 stainless steel rolling balls sandwiched between two stainless steel plates, which creates a sliding boundary with very low friction. The support wall was placed on a layer of rubber sheet that was glued to the upper steel plate, in order to accommodate the potential rocking of the support wall during shaking. The bottom of the support wall was connected to two steel beams using post-tension rods. The steel beams were connected with the shaking table and transmit the table motions to the base of support wall such that the entire bridge system is shaken uniformly.

(a)



(b)

Figure 3. Bridge support wall photos: (a) sliding platform; (b) end view.

Materials

The backfill soil for this study is a well-graded sand (SW) according to the Unified Soil Classification System (USCS) with a coefficient of uniformity C_u = 6.1, and a coefficient of curvature C_c = 1.2. The friction angle is 49.3° for a relative density of 70% used in this study. A series of triaxial compression tests were performed on dry specimens of the sand under different relative densities, and it was found that the secant modulus at 0.5% axial strain for the specimen with a relative density of 70% under a confining stress of 34.5 kPa was approximately one half that of a specimen with a relative density of 85% under 69 kPa confining stress. It is assumed that a relative density of 85% is within the range of compaction specifications typically used in prototype GRS abutments prepared with actual construction equipment. The reinforcement was carefully selected to meet the scaling requirements in Table 1, and is a uniaxial high density polyethylene (HDPE) geogrid with a secant stiffness of 280 kN/m at 5% strain and ultimate strength of 35 kN per unit width, as provided by the manufacturer. The tensile stiffness of this geogrid corresponds to a stiffness of 1120 kN/m, which is typical of geogrids used in full-scale GRS walls and abutments. The concrete facing blocks have dimensions of 0.25 m × 0.3 m × 0.15 m and fiberglass pin connections.

Construction

The first layer of blocks was carefully placed and leveled on the foundation soil. The side wall blocks were offset vertically from the front facing blocks by 25 mm. This avoided reinforcement layers in the longitudinal direction from directly contacting those in the transverse direction, while still ensuring interaction with the backfill. The longitudinal and transverse reinforcement layer placements are shown in Figures 4(a) and 4(b), respectively. During construction of each lift, backfill soil was placed with a gravimetric water content of approximately 5%, and a vibratory plate compactor was used to compact the soil lift to a target relative density of 70%.

Sand cone tests were performed at selected elevations to measure actual soil density. The shear pins between the facing blocks were passed through the geogrid layers as shown in Figure 4(a). Although typically grouted in the field, the upper row of blocks was ungrouted in this test.

INSTRUMENTATION AND INPUT MOTIONS

Instrumentation consisted of potentiometers for lateral facing displacements and bridge seat settlements, accelerometers for accelerations, total pressure cells for horizontal and vertical stresses, load cells for horizontal contact forces between the bridge deck and seat, and strain gauges for geogrid reinforcement strains. Instrumentation for the longitudinal section at the centerline is shown in Figure 5. This paper focuses on the results from sensors used to measure lateral facing displacements and bridge seat settlements.

(a)

(b)

Figure 4. Construction of GRS bridge abutment: (a) longitudinal reinforcement; (b) transverse reinforcement.

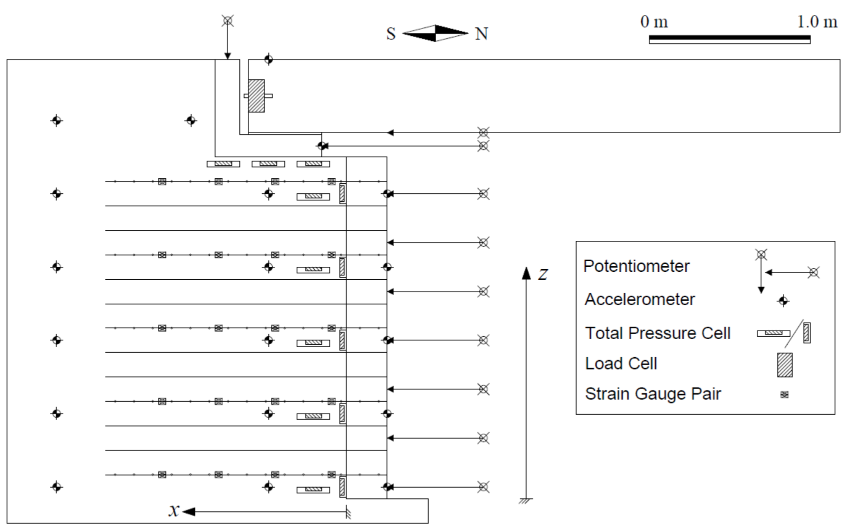


Figure 5. Instrumentation for centerline section in longitudinal direction.

A series of motions, including white noise, earthquake motions, and sinusoidal motions, were applied to the bridge system in the longitudinal direction. The earthquake motions include a strike-slip earthquake (1940 Imperial Valley at El Centro station, PGA = 0.31 g) and a subduction zone earthquake (2010 Maule earthquake at Concepcion station, PGA 0.4 g). The acceleration amplitudes of the earthquake motions were not scaled, but the frequencies were scaled down by a factor of $\sqrt{2}$ (Table 1). An example record for the 1940 Imperial Valley earthquake is shown in Figure 6. The shaking table was operated in displacement-control mode, and indicated excellent performance response despite the large self-weight load on the table.

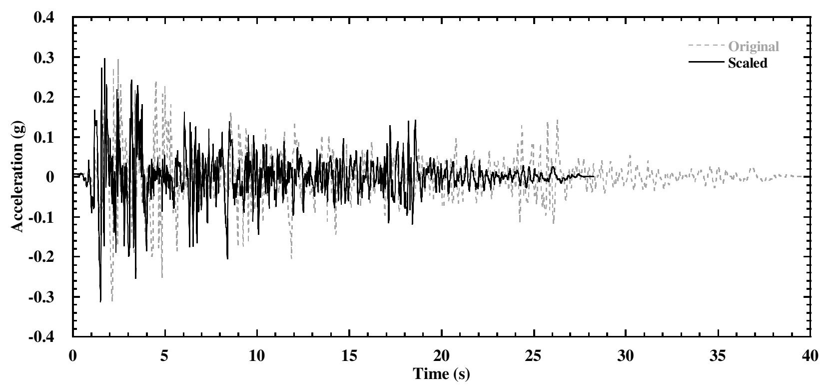


Figure 6. Acceleration time history for the 1940 Imperial Valley Earthquake.

TEST RESULTS

Lateral Facing Displacements

The incremental lateral facing displacements for the half-scale model at the end of construction and after application of the Imperial Valley and Maule earthquakes are shown in Figure 7. At the end of construction, the maximum lateral displacement is 3.2 mm (model scale) and occurs at an elevation of 1.6 m. During seismic shaking, the incremental maximum lateral displacements for the Maule earthquake are larger than for the Imperial Valley earthquake, and the maximum values are 4.9 mm and 3.7 mm, respectively (model scale). However, the seismic-induced residual displacements (approximately 1 mm) are similar for the two earthquakes.

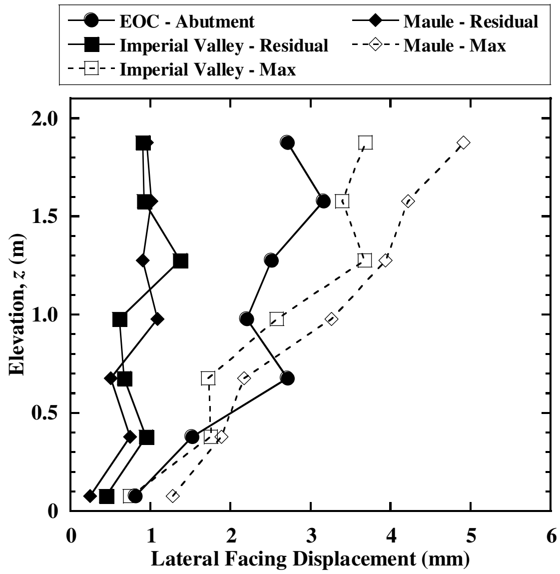


Figure 7. Incremental lateral facing displacements (model scale) for different stages.

Bridge Seat Settlements

The average incremental bridge seat settlements for the half-scale model at the end of different stages (i.e., residual values) are provided in Table 2. The settlements during and after shaking events were measured using string potentiometers connected to the top four corners of the bridge seat. The settlement due to placement of the bridge dead weight (98 kN) is 2.1 mm (model scale), which corresponds to a vertical strain of 0.1% for the 2.1 m-high GRS wall. This is within the acceptable limit for static performance of the GRS abutment. The incremental seismic-induced residual settlements were 1.4 mm (model scale) for each earthquakes, which correspond to prototype-scale settlements of 2.8 mm for the full-scale GRS bridge abutment. These settlement values are small and indicate good performance of GRS abutment under seismic loading.

Table 2. Average incremental residual bridge seat settlements (model scale).

Testing Stage	Bridge Load Application	1940 Imperial Valley Earthquake	2010 Maule Earthquake
Settlement (mm)	2.1	1.4	1.4

CONCLUSIONS

This paper presents the results from a shaking table test performed to characterize the seismic performance of a half-scale GRS bridge abutment. The model geometry, reinforcement stiffness, bridge load, and characteristics of earthquake motions were scaled according to established similitude relationships for reduced-scale models in a 1 g gravitational field. The results indicate that the GRS bridge abutment had good seismic performance in terms of lateral facing displacements and bridge seat settlements. Analysis of results from other instrumentation in the GRS abutment is underway, and further testing is being performed to investigate the effects of design parameters such as reinforcement spacing, reinforcement stiffness, and bridge load.

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