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Publication Date
2018-06-07

DOI
10.1061/9780784481486.001

Peer reviewed

Acceleration Response of a Geosynthetic Reinforced Soil Bridge Abutment Under Dynamic Loading

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ABSTRACT

This paper presents results from dynamic testing of a half-scale geosynthetic reinforced soil (GRS) bridge abutment using a shaking table, with the goal of understanding the acceleration response of the backfill soil, bridge seat, and bridge beam under dynamic loading. The GRS bridge abutment model was constructed using modular facing blocks, well-graded angular sand backfill, and uniaxial geogrid reinforcement in both the longitudinal and transverse directions. A series of input motions was applied to the GRS bridge abutment system in the direction longitudinal to the bridge beam. The horizontal accelerations increase with elevation in the reinforced soil zone and retained soil zone. The average peak acceleration of the reinforced soil zone is slightly greater than the calculated value from the current design guidelines, indicating that the guidelines may not be sufficiently conservative. The acceleration response spectrum for the bridge beam indicates a slight attenuation compared with that of the bridge seat, likely due to the isolation effect of an elastomeric bearing pad between the bridge beam and bridge seat.

INTRODUCTION

Geosynthetic reinforced soil (GRS) bridge abutments are widely used in transportation infrastructure. However, the performance of this technology in high seismicity areas like California is uncertain due to the complex interactions between the reinforced soil mass, the bridge seat, and the bridge beam that provides a confining effect but adds inertial effects. Due to a prior lack of experimental data on the seismic response of these structures, the existing design guidelines are still preliminary and are primarily based on observations for GRS walls.

Experimental and numerical studies have been conducted on the response of GRS bridge abutments for static loading conditions (Abu-Hejleh et al. 2002; Wu et al. 2001, 2006; Helwany et al. 2003, 2007; Zheng and Fox 2016, 2017; Saghebfar et al. 2017; Zheng et al. 2018). However, fewer studies have investigated the response of GRS bridge abutments for dynamic loading conditions. Helwany et al. (2012) performed shaking table tests on a 3.6 m-high GRS bridge abutment subjected to a series of horizontal sinusoidal motions with increasing amplitude in the longitudinal direction. No damage was observed for horizontal accelerations up to 0.67*g* at which time several bottom blocks near the corners had minor cracks, and the abutment remained functional with more damage to the bottom corner blocks when the horizontal acceleration was further increased to 1.0*g*. Zheng et al. (2017) reported results from shaking table tests on a 2.7 m-high half-scale GRS bridge abutment for shaking in the longitudinal direction, and observed relatively small residual deformations after earthquake motions with peak horizontal accelerations (PHA) of 0.31*g* and 0.40*g*. Although these experimental studies indicate that GRS bridge abutments may have satisfactory performance regarding deformations under dynamic loading, it is necessary to further evaluate the potential acceleration amplification in these systems. The acceleration response in the GRS bridge abutment is important because, during an earthquake, the retained fill exerts a dynamic thrust on the reinforced soil zone and the reinforced soil zone is subjected to an inertial force, which should be adequately accounted for in the external and internal stability evaluation. To address this need, this paper presents results and analysis on the acceleration response from shaking table tests on a GRS bridge abutment previous reported by Zheng et al. (2017).

SHAKING TABLE TESTS

The shaking table test was conducted using the indoor shaking table at the University of California, San Diego (UCSD) Powell Structural Laboratory. Considering the size and payload capacity of the shaking table, a length scaling factor of $\lambda = 2$ was selected, defined as the ratio of prototype length to model length. In this study, the similitude relationships proposed by Iai (1989) were used for the half-scale shaking table tests. The model geometry, geosynthetic reinforcement stiffness, backfill soil modulus, bridge surcharge stress, and characteristics of the earthquake motions were scaled accordingly.

Model Configuration

The shaking table test configuration for the GRS bridge abutment system is shown in Zheng et al. (2017). The GRS bridge abutment was constructed on the shaking table and had modular block facing on three sides, including a front wall facing perpendicular to the longitudinal direction and two side wall facings perpendicular to the transverse direction. The back of the GRS bridge abutment was supported by a rigid reaction wall consisting of a steel frame with plywood face. The bridge beam was placed on a bridge seat that rested on the GRS bridge abutment at one end and on a concrete support wall that rested on a sliding platform at the other end. The bottom of the concrete support wall was rigidly connected to the shaking table using steel connection beams to transmit motions from the shaking table. The shaking table test represents the case where the ground beneath the bridge abutment is relatively rigid and transmits the rock motions directly to the GRS bridge abutment without amplification.

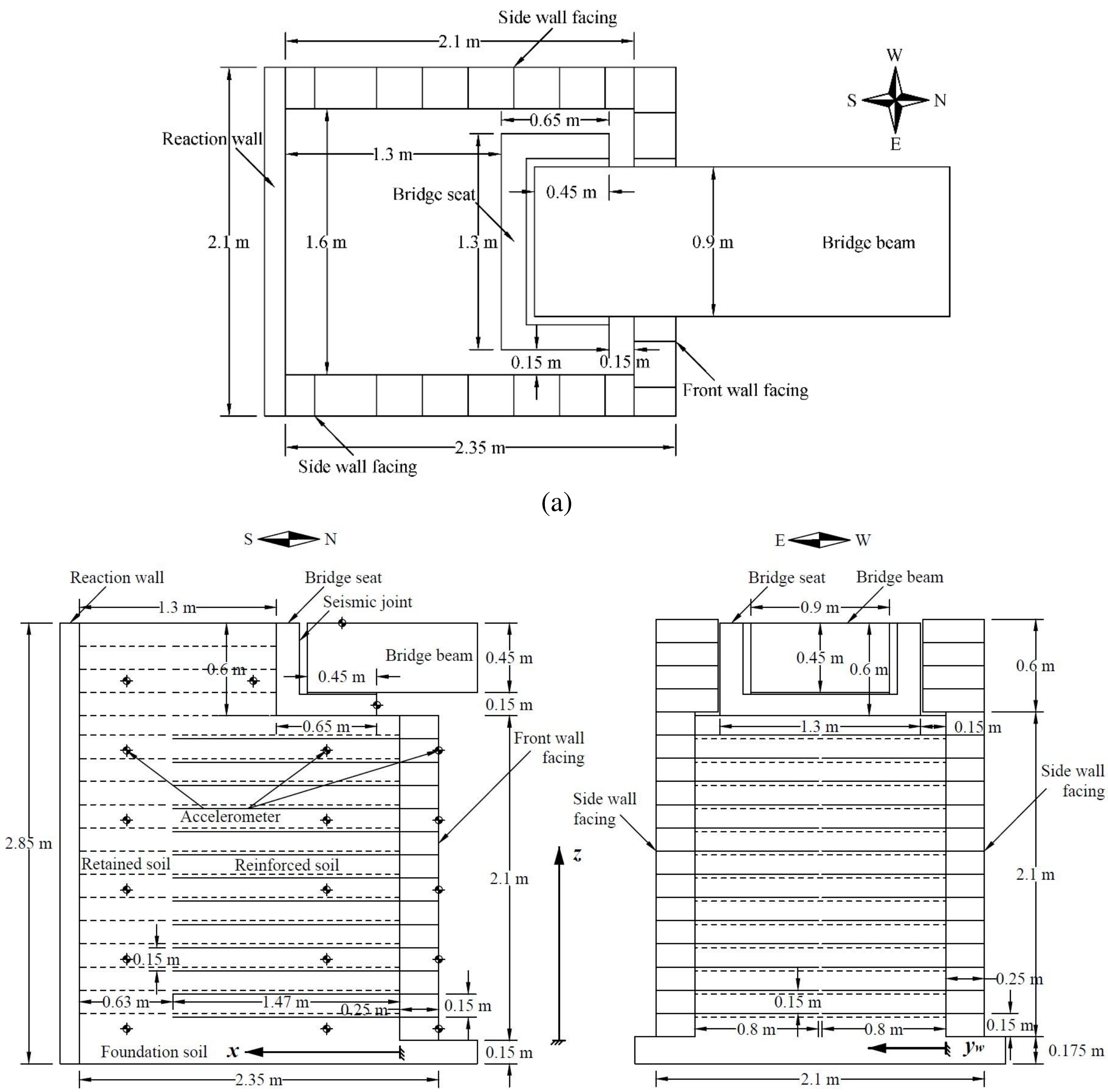
A top view diagram and cross-sectional view diagrams in the longitudinal and transverse directions for the GRS bridge abutment model are shown in Figure 1. The GRS bridge abutment has a total height of 2.7 m, consisting of a 2.1 m-high lower GRS wall and a 0.6 m-high upper wall, resting on a 0.15 m-thick foundation soil layer. The lower GRS wall was constructed in fourteen 0.15 m-thick soil lifts. Each 0.15 m-thick lift includes one layer of longitudinal reinforcement and two layers of transverse reinforcements. The longitudinal reinforcement layers are frictionally connected to the front wall facing and extend 1.47 m into the backfill soil, and the transverse reinforcement layers are frictionally connected to each side wall facing and extend 0.8 m into the backfill soil (meet but not connected in the center). The transverse reinforcement layers and side wall facing blocks are offset by 25 mm vertically from the longitudinal reinforcement layers and front wall facing blocks to avoid direct contact between longitudinal and transverse geogrid layers and maintain interaction between the geogrid and backfill soil.

The bridge seat rests on top of the backfill soil for the lower GRS wall and has a setback distance of 0.15 m from each of the three wall facings. Elastomeric bearing pads with a thickness of 25 mm were placed under both ends of the bridge beam. The bridge superstructures (i.e., bridge beam and additional dead weights) have a total weight of 98 kN, which produces an average vertical stress of 121 kPa on the bridge seat top surface. The average applied vertical stress on the backfill soil from the bridge seat bottom surface is 66 kPa, which corresponds to a prototype vertical stress of 132 kPa and is in the typical range for GRS bridge abutments in the field (Adams et al. 2011).

Material Properties

The backfill soil is a well-graded sand and has a relatively flat compaction curve. The target soil compaction conditions for construction of the GRS bridge abutment model were gravimetric water content of 5% and relative density of D_r = 70%. The target relative density was selected to meet the similitude relationships and to obtain reproducible densities using a vibrating plate compactor. The details of the selection of target compaction conditions are discussed in Zheng (2017). The dry backfill sand at D_r = 70% has a peak friction angle of 51.3° and zero cohesion according to results from triaxial compression tests.

The geosynthetic reinforcement is a uniaxial high-density polyethylene (HDPE) geogrid (Tensar LH800). The geogrid has secant stiffness at 5% strain $J_{5\%}$ = 380 kN/m and ultimate strength T_{ult} = 38 kN/m in the machine direction, and $J_{5\%}$ = 80 kN/m and T_{ult} = 4 kN/m in the cross-machine direction. The tensile stiffness of this geogrid corresponds to a value of 1520 kN/m for the prototype geogrid (scaling factor = 4), which is typically used for field structures. The geogrid reinforcement layer was placed between the facing blocks. Fiberglass pins were inserted through the geogrid apertures to assist with block alignment and are not expected to enhance the block-geogrid connection, which was essentially frictional.



(b) (c)

Figure 1. GRS bridge abutment model: (a) top view; (b) longitudinal cross-sectional view (showing locations of the accelerometers); (c) transverse cross-sectional view.

Instrumentation and Input Motions

The instrumentation locations are shown in Figures 1(b) and 1(c). Horizontal coordinate x is measured toward the south side from the back of the front wall in the longitudinal centerline section (Figure 1b), horizontal coordinate y_w is measured toward the east from the west side wall facing in the transverse section (Figure 1c), and vertical coordinate z is measured upward from the top surface of the foundation soil. Accelerometers were placed within the reinforced soil zone (x = 0.48 m) and retained soil zone (x = 1.67 m) and attached on the wall facing and structures to measure horizontal accelerations for the longitudinal centerline section, as shown in Figure 1(b). Accelerations toward the north (see orientations in Figure 1) are defined as positive. A series of white noise and earthquake motions were applied to the GRS bridge abutment system in the longitudinal direction. The earthquake motions were scaled according to the similitude relationships of Iai (1989), in which the frequencies were scaled down by a factor of $\sqrt{2}$ while the acceleration amplitude remains the same. Although several earthquake motions were applied to this model, this paper focuses on the acceleration response of the GRS bridge abutment subjected to the 1940 Imperial Valley Earthquake (El Centro station). The original record and scaled motion applied to the model are shown in Figure 2. The actual shaking table response for this test reproduced the major characteristics of the target scaled motion and had a PHA of 0.40g, which is larger than the target value of 0.31g.

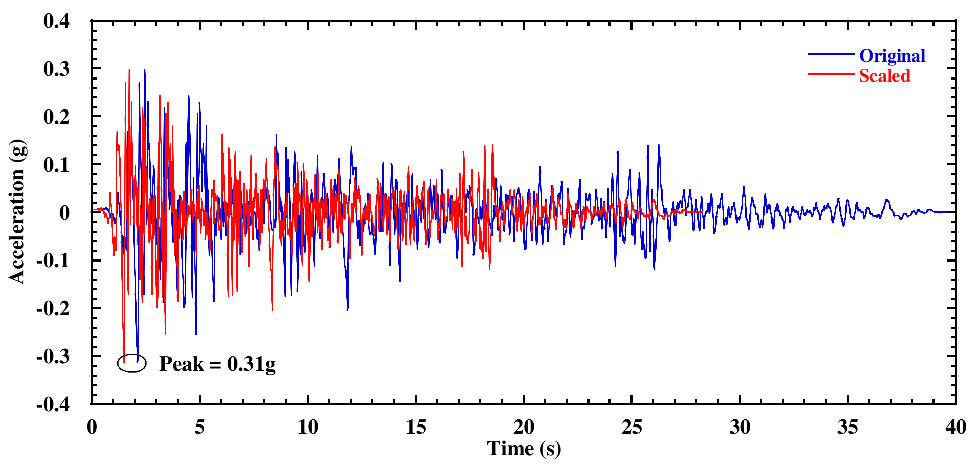


Figure 2. Acceleration time histories of the original record and scaled motion for the 1940 Imperial Valley Earthquake (El Centro station).

TEST RESULTS

Horizontal acceleration time histories at selected elevations in the reinforced soil zone and retained soil zone for the longitudinal centerline section are shown in Figure 3. Data show that the horizontal accelerations in the backfill soil increase with elevation in both the reinforced and retained soil zones, and indicate acceleration amplification toward the top of the GRS bridge abutment. The magnitudes of acceleration at the same elevations in the reinforced soil zone and retained soil zone are similar. The peak accelerations at the top (z = 1.875 m) of the GRS bridge abutment are 0.58g and 0.57g for the reinforced and retained soil zones, respectively.

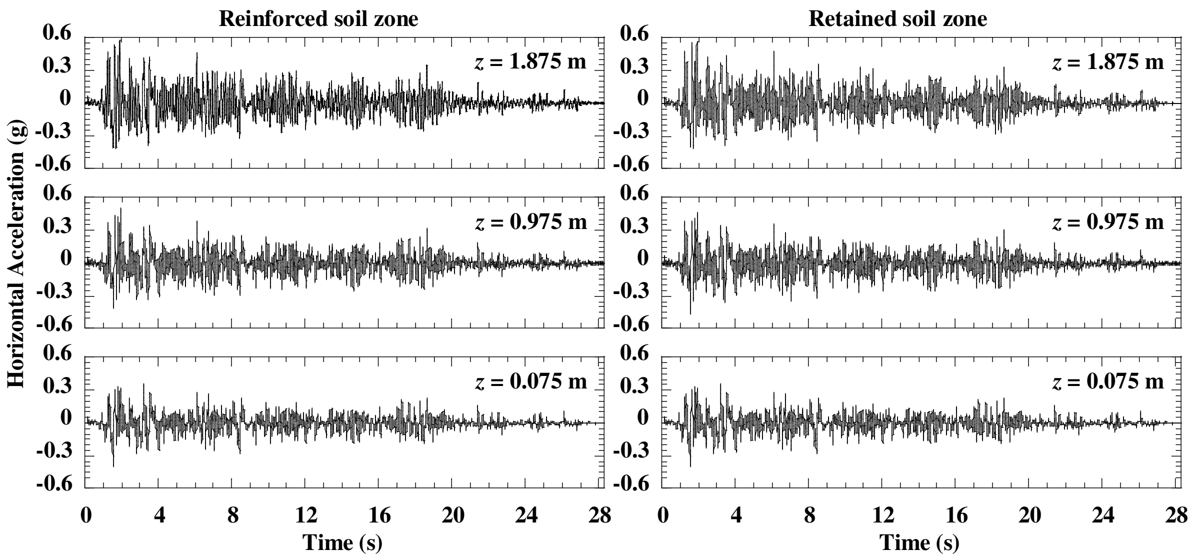


Figure 3. Acceleration time histories in the reinforced soil zone and retained soil zone.

The peak acceleration amplification profiles for the wall facing, reinforced soil zone, and retained soil zone, normalized by the actual peak acceleration of the shaking table (0.40g), are shown in Figure 4. Results indicate that the peak accelerations increase with elevation for all three sections. The amplification ratios for the wall facing are larger than for the reinforced and retained soil zones, which is likely due to the lower confinement for the facing blocks compared to the reinforced and retained soil zones. The height average peak accelerations are 0.47g and 0.46g for the reinforced and retained soil zones, respectively, corresponding to amplification ratios of 1.18 and 1.16. The slightly greater value for the reinforced soil zone could be due to the greater confinement by the bridge load.

During an earthquake, the reinforced soil zone is subjected to an inertial force, which should be accounted for in the external and internal stability evaluation. In the seismic design guidelines (The Reinforced Earth Company 1995; AASHTO 2012), the average peak acceleration for the active portion of the reinforced soil zone is $A_m = (1.45 - A)*A$, where A is the

PHA (0.40g for this test). The measured average peak acceleration of 0.47g for the reinforced soil zone is slightly greater than the calculated value of 0.42g according to the design guidelines, which indicates that the design guidelines may not be sufficiently conservative.

The acceleration response spectra (5% damping) at different elevations in the reinforced soil zone are shown in Figure 5. The acceleration response spectrum at the bottom (z = 0.075 m) of the GRS bridge abutment is essentially the same as that from the shaking table motion. However, the motion was significantly amplified at the mid-height (z = 0.975 m) and the top (z = 1.875 m) of the abutment, especially in the frequency range around 7 Hz. This further indicates the acceleration amplification in the reinforced soil zone.

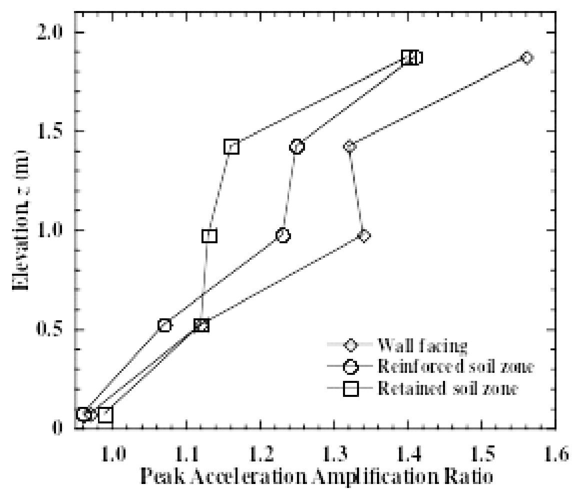


Figure 4. Peak acceleration amplification ratio profiles in the GRS bridge abutment.

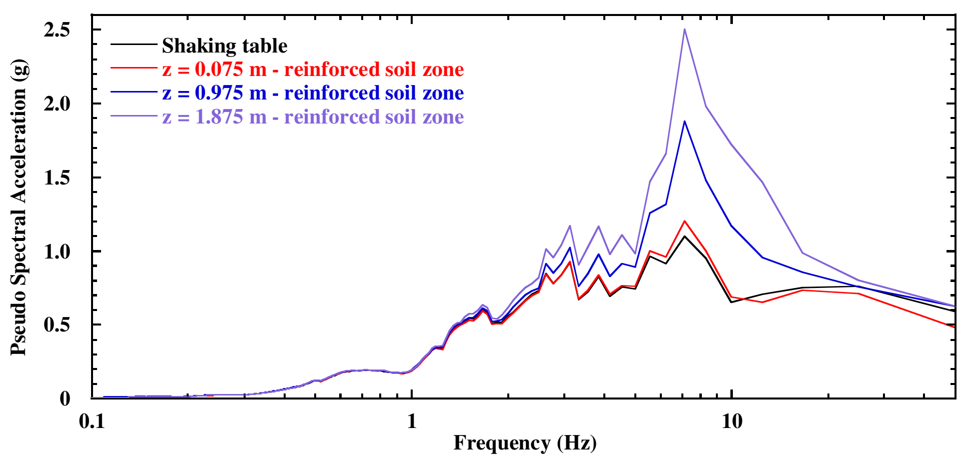


Figure 5. Acceleration response spectra (5% damping) in the reinforced soil zone.

Time histories of horizontal acceleration for the bridge seat and bridge beam are shown in Figure 6. The bridge seat had a peak acceleration of 0.63g, while the bridge beam had a smaller peak acceleration of 0.53g, which correspond to peak acceleration amplification ratios of 1.58

and 1.33, respectively. The bridge seat is typically treated as a gravity retaining wall for external stability evaluation in the seismic design. Accelerations for the bridge seat and bridge beam are assumed to be the same as the PHA in the seismic design guidelines recommended by the Reinforced Earth Company (1995) due to limited information. However, results from this study indicate that the peak accelerations for the bridge seat and bridge beam are greater than the PHA due to significant acceleration amplification in the reinforced soil zone (Figures 4 and 5).

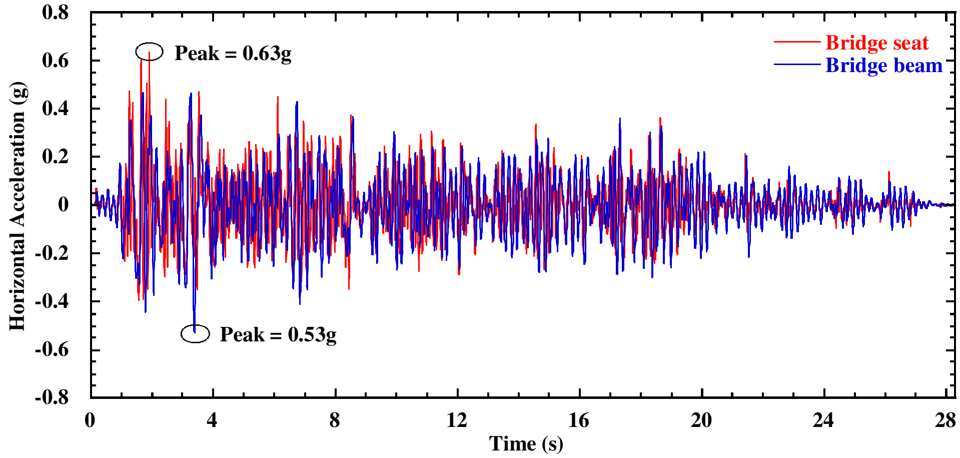


Figure 6. Acceleration time histories for the bridge seat and bridge beam.

The acceleration response spectra for the bridge seat and bridge beam are shown in Figure 7. The response spectrum for the bridge seat is similar to that observed for the top of the GRS bridge abutment (z = 1.875 m in Figure 5). However, the response spectrum for the bridge beam indicates a slight attenuation compared with the bridge seat but still shows strong amplification in the frequency range around 4 Hz. This may be attributed to the isolation effect of the elastomeric bearing pad between the bridge seat and bridge beam and indicate that the elastomeric bearing pad might attenuate the motion transmitted from the bridge seat.

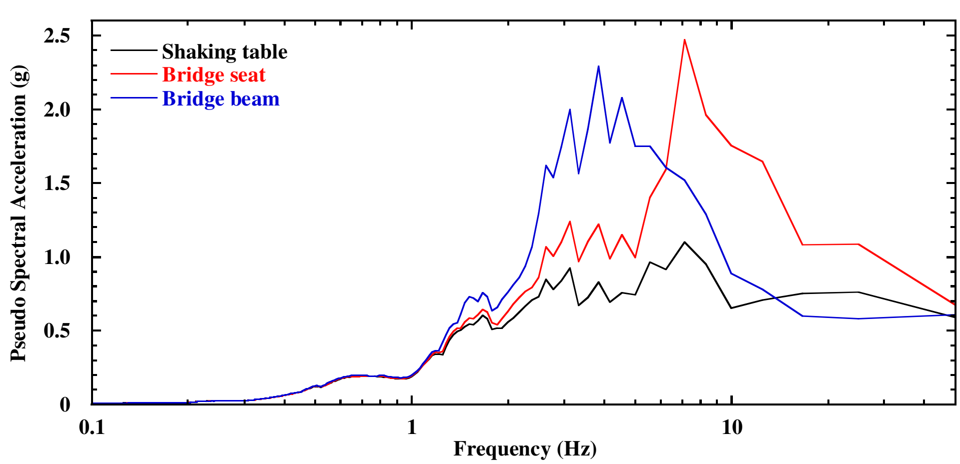


Figure 7. Acceleration response spectra (5% damping) for the bridge seat and bridge beam.

CONCLUSIONS

This paper presents results of acceleration response from a shaking table test on a half-scale GRS bridge abutment with modular block facing. The GRS bridge abutment was constructed using well-graded backfill sand and uniaxial geogrid reinforcement in both the longitudinal and transverse directions. A series of scaled earthquake motions were applied to the GRS bridge abutment system in the longitudinal direction. Experimental results indicate that the horizontal accelerations in the backfill soil increase with elevation in both the reinforced and retained soil zones. The measured average peak acceleration of 0.47*g* for the reinforced soil zone is slightly greater than the value of 0.42*g* calculated from current design guidelines, which indicates that these design guidelines may not be sufficiently conservative. The peak accelerations for the bridge seat and bridge beam are greater than the PHA due to significant acceleration amplification in the reinforced soil zone. The response spectrum for the bridge beam indicates a slight attenuation compared with that of the bridge seat, which is likely due to the isolation effect of the elastomeric bearing pad between the bridge seat and bridge beam.

ACKNOWLEDGEMENTS

Financial support for this study was provided by the California Department of Transportation (Caltrans) Project 65A0556 and Federal Highway Administration (FHWA) Pooled Fund Project 1892AEA, and is gratefully acknowledged. The first author appreciates funding from the GSI Fellowship provided by the Geosynthetic Institute. The authors thank Dr. Charles Sikorsky and Ms. Kathryn Griswell of the Caltrans for their support and assistance with the project. The authors also thank the staff and undergraduate research assistants at the UCSD Powell Structural Laboratories for their help with the experimental work. The geogrid materials used in this study were provided by Tensar International Corporation.

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