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SEISMIC PERFORMANCE OF HYBRID FRP-CONCRETE BRIDGE PIER COLUMNS AND BENTS

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ABSTRACT

Concrete-filled fiber reinforced polymer (FRP) tubes (CFFT) have been used as bridge columns and girders in several bridges in the US. Their applications in seismic regions, however, have been rather slow, primarily due to lack of understanding of how the system withstands large earthquake forces. While previous experiments and modeling by the authors have shown the feasibility of CFFT for seismic applications, their system-wide performance has not been yet fully evaluated. This paper provides the background, challenges, and advantages of the proposed system. It also introduces an on-going project to design the next generation of seismic-resistant bridge substructures, as part of a multi-university project, which is funded by the NSF-Network of Earthquake Engineering Simulation Research (NEESR) program.

Introduction

Fiber reinforced polymers (FRP) have in recent years been used in a variety of bridge applications. One of the most cost-effective-effective applications of FRPs is in hybrid construction with concrete, where FRP provides the pour form, lifetime protective jacket, and shear and flexure reinforcement for concrete. Mirmiran et al. (1999) reported that the entire steel reinforcement in a concrete column or pile could be replaced by the FRP tube without affecting its load carrying capacity under static loading. As a result, the concept has been applied in a number of piling applications in the US. One example is the precast piles on Route 40 bridge in Virginia (Fam et al. 2003).

On the other hand, Seible et al. (1996) showed that concrete-filled FRP tubes (CFFT) without steel reinforcement at the column-footing connection can not survive large earthquake forces. Same study revealed that CFFT columns with starter bars can provide both strength and ductility necessary for bridge columns. Shao (2003) showed for the first time that with proper design of fiber orientation in the tube, it is possible to achieve considerable ductility in concrete columns with glass FRP tubes as a replacement of the entire internal steel. The achieved ductility was attributed to the shear stresses in the tube. Extending the work of Shao (2003), Zhu (2004) suggested the use of glass FRP tubes in columns as a replacement of transverse steel, while maintaining the longitudinal reinforcement in the plastic hinge zones.

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This paper introduces the on-going project to design the next generation of seismic-resistant bridge substructures, as part of a multi-university project, which is funded by the NSF-Network of Earthquake Engineering Simulation Research (NEESR) program. The project consists of three phases, including a) single column tests, b) two-column bent tests, and c) four-span bridge model test. The purpose of this project is to compare the performance of glass, carbon, and hybrid FRP tubes; compare the flexure-dominant versus shear-dominant behavior in seismic applications; and determine the holistic performance of the entire frame system as well as bridge system made with CFFT substructure.

Experimental Database

The research group has completed two separate yet complementing studies to date. Shao (2003) tested six beam-columns under a constant axial load and four point bending (Figure 1). Lateral load was applied in displacement control with reverse motion from ±6.4 to ±127 mm. Each specimen was 2.4 m long. Two types of E-glass/epoxy tubes were tested, representing two different failure modes; brittle compression failure for the thick tubes with mostly longitudinal fibers, and ductile tension failure for the thin tubes with off-axis fibers. In each series, one specimen was made with no internal steel, while the other two included approximately 1.7% and 2.5% steel reinforcement. Tests showed a ductile elasto-plastic behavior for the thin tubes. The internal steel generally improved the response, especially for the thin tubes.

Zhu (2004) tested one RC column and three CFFT columns, including a cast-in-place CFFT with starter bars from footing, a precast CFFT with grouted starter bars from footing, and a precast CFFT post-tensioned to the footing with internal threaded rods. Columns were tested under a constant axial load and a reverse cyclic load, which was applied in displacement control as a multiple of the first yield displacement for the internal steel. All columns were 2.3 m high, and all included four 16 mm and four 19 mm diameter Grade 414 MPa mild steel bars, expect for the post-tensioned CFFT column, which had eight 19 mm Grade B-7 high strength threaded rods with a yield strength of 724 MPa. All CFFT columns showed significant improvement over the RC column in both the ultimate strength and ductility (Figure 2). The CFFT columns did not fail up to a displacement ductility of 10.

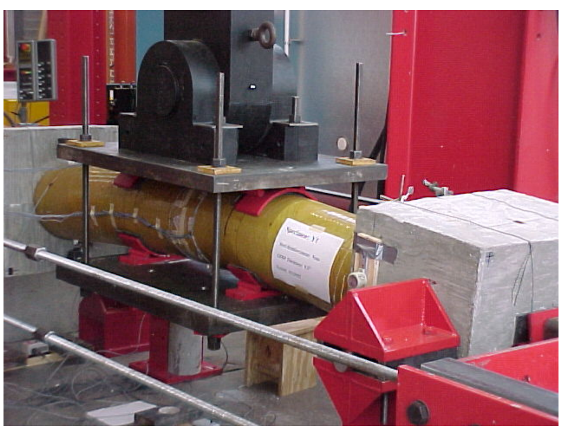

Figure 1. Member Level Tests (Shao 2003)

Figure 2. Connection Tests (Zhu 2004).

Pushover Modeling

Modeling was carried out by Zhu (2004), using Open System for Earthquake Engineering Simulation (OpenSees) developed by the University of California, Berkeley. In this model, concrete confined by spiral steel was modeled after Mander et al. (1988). The confined concrete in CFFT columns was modeled after Samaan et al. (1998). Both confinement models were cast into the modified Kent-Park model (Scott et al. 1982), which is programmed into OpenSees with a hysteretic feature. The steel bars were simulated as elasto-plastic material. A tri-linear hysteretic model was defined in OpenSees for the FRP tube. Both RC and CFFT columns were analyzed using non-linear constitutive models at the sectional level and integration of sectional curvature along the column height. It was assumed that plane sections would remain plane, and that neither steel rebars nor FRP tube would buckle. The analysis showed good agreement with the specimens of Shao (2003) and Zhu (2004). One such validation is shown in Figure 3.

Once confidence was gained in the OpenSees model, it was used to compare the lateral load-deflection of RC and CFFT columns with each other. Figure 4 shows the pushover response of RC and CFFT frames. While both systems have the same initial stiffness, as intended, the CFFT column depicts much higher strength and ductility than the RC column in contrast to their sectional responses, where RC column outperforms. This is primarily due to the fact that confinement effect of FRP helps extend the plastic hinge zone of the column, thereby engaging a larger portion of the column in the energy dissipation process. This is perhaps the most significant contribution of the FRP tube to the seismic response of RC columns.

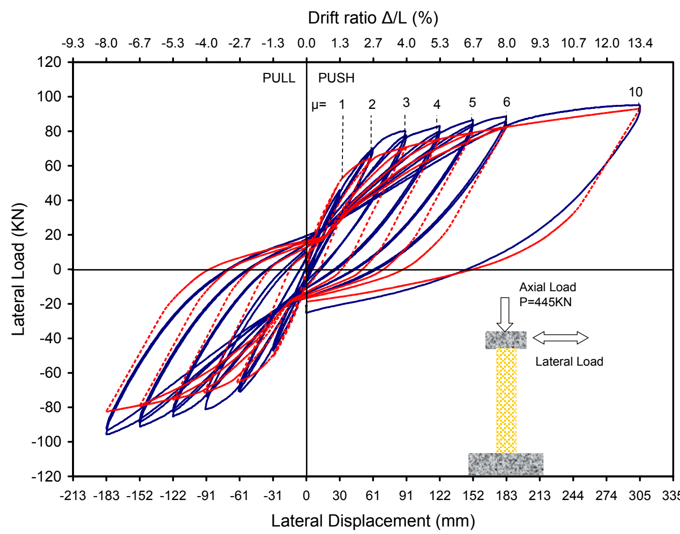

Figure 3. Validation of OpenSees Model with Cast-in-Place CFFT Column.

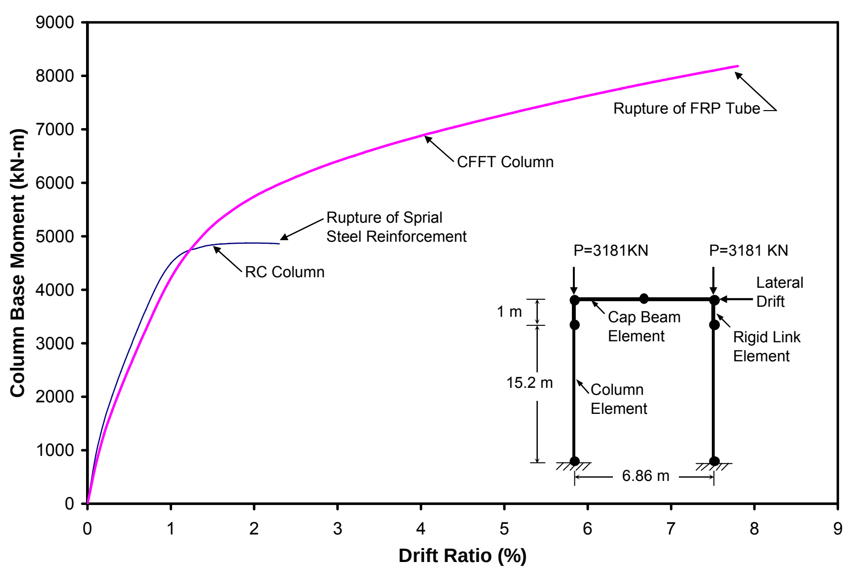

Figure 4. Comparison of CFFT and RC Frames in Pushover Analysis.

Subsequent to the pushover analysis, an entire bridge made of RC or CFFT substructure was subjected to simulated ground accelerations. The study indicated superior performance of CFFT bridge substructure in withstanding large earthquake forces. Figure 5 shows a comparison of damage index for the RC and CFFT columns in a five-span bridge (shown in the figure inset) subjected to Tabas earthquake ground accelerations records. The earthquake in Tabas, Iran was measured between 7.5 and 7.9 on the Richter scale, with an estimated casualty of 25,000 people.

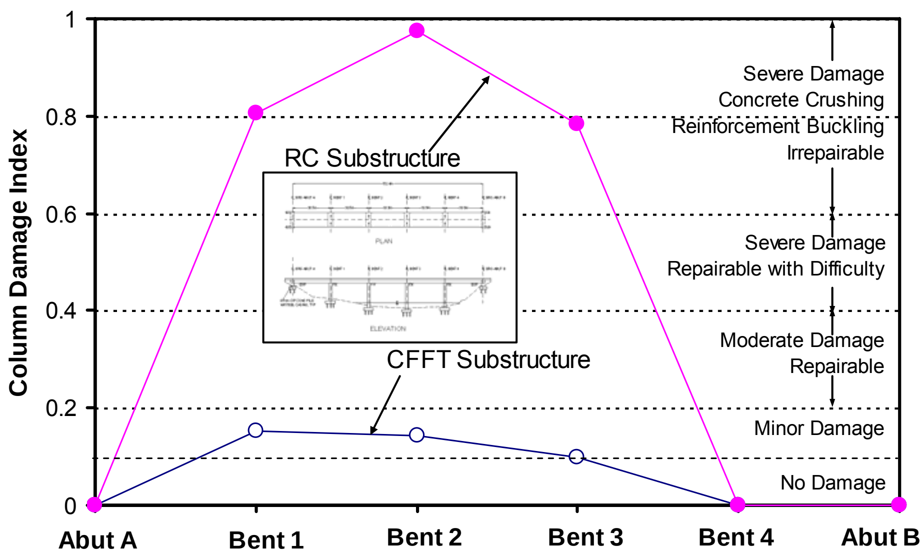

Figure 5. Comparison of Damage Index for RC and CFFT Substructures

Challenges of Using CFFT for Seismic Applications

While previous studies have clearly demonstrated both the feasibility and the advantages of CFFT systems for seismic applications, there remain a number of challenges that need to be met, if the proposed system is going to be viable for bridge applications in earthquake-prone regions. Some the challenges are addressed below:

1. There has not been a direct comparison of carbon and glass fibers for the FRP tubes. Some studies suggest that carbon provides a higher strength, while others have preferred glass for the improved ductility. It has also been suggested that a hybrid lay-up with carbon fibers in the axial direction and glass fibers in the hoop direction may help combined the advantages of both types of fibers.
2. The most recent tests by the research group have focused on flexure-dominant long CFFT columns, whereas shear-dominant short columns may result in different performance for the tubes.
3. Although CFFT pier frames have been tested by the research group under gravity loads, pushover tests on such bents have not been carried out. The design implications at the connections as well as the pier cap beam have not been addressed either.
4. To date, a holistic system-wide performance study of CFFT bridge substructure has not been done except for analytical simulation. Unless a shake table study on multi-span bridge with such substructure is made, system-wide performance may not be clear.

Ongoing Research Program

A three-phase study is underway as part of a multi-university project, which is funded by the NSF-

Network of Earthquake Engineering Simulation Research (NEESR) program. The study addresses the four challenges described in the previous section. The three phases are as follows:

1. Single Column Test Program: A total of six (6) specimens are prepared in this stage with one control RC column and five CFFT columns. The CFFT columns consist of four short columns made of carbon, hybrid, glass, and filament-wound FRP tubes, and one long carbon FPR tube. The formwork for the RC control specimen consisted of a 305 mm (12 in) diameter sonotube, while the other five CFFT columns used FRP tubes as the cast-in-place formwork. Figures 6 and 7 illustrate a glass and a carbon FRP tube, made in the laboratory by wrapping fabric over a sonotube.

Figure 6. Glass FRP Tube.

Figure 7. Carbon FRP Tube.

The glass FRP tube was made with 3 layers of bi-directional glass fiber sheets. The cured laminate thickness per layer is 0.33 mm (0.013 in). Two carbon FRP tubes with shear spans of 1,295 mm (51 in) and 2,210 mm (87 in), respectively, were made with 2 layers of bi-directional carbon fiber sheets. The cured laminate thickness per layer is 0.25 mm (0.01 in). In the hybrid FRP tube, 3 layers of unidirectional glass fiber sheets were wrapped in the transverse direction over the two layers of unidirectional carbon fabric. It may be of interest to note that the layer numbers were designed based on equivalence comparison of the tensile, compressive and shear strengths of cured laminate properties of glass and carbon fabrics. Apart from the CFFT with carbon tube of 2,210 mm (51 in) shear span (hence long column), all the other five columns have the same 1,295 mm (51 in) shear spans (hence short columns). Figure 8 shows the five short specimens, while Figure 9 shows the preparation of long carbon FRP Tube. Table 1 shows the test matrix of single column experiments.

Figure 8.Five Short Column Specimens.

Figure 9.Preparation of Long Carbon Specimen.

Table 1.Test Matrix of Single Column.

Specimen Name	Shear Span mm (in)	Core Diameter mm (in)	f'c MPa (ksi)	Longitudinal FRP	Transverse FRP
RC (Control)	1,295 (51)	310 (12.2)	44.8 (6.5)	None	
Y (Yellow tube)	1,295 (51)	315 (12.4)	44.8 (6.5)	17 Layers of +55o E-Glass	
G (Glass)	1,295 (51)	318 (12.5)	44.8 (6.5)	3 Layers of Bi-directional Glass	
CS (Carbon, Short Column)	1,295 (51)	318 (12.5)	31.0 (4.5)	2 Layers of Bi-directional Carbon	
				2 Layers of	3 Layers of
H (Hybrid)	1,295 (51)	318 (12.5)	31.0 (4.5)	Unidirectional	Unidirectiona l
				Carbon	Glass
CL (Carbon, Long Column)	2,210 (87)	318 (12.5)	31.0 (4.5)	2 Layers of Bi-directional Carbon	

All six columns had the same longitudinal reinforcement of sixteen 10 mm (No. 3) steel bars of Grade 414 MPa (60 ksi) along the entire length of the columns, with adequate embedment into the footing and the column head. The reinforcement ratio is 1.5%. The RC column used a spiral reinforcement with 5.3 mm (0.207 in) steel wire of Grade 414 MPa (60 ksi) with 279 mm (11 in) outside diameter placed at a pitch of 32 mm (1.25 in). CFFT columns had no transverse steel reinforcement, except for four or five 279 mm (11 in) diameter hoops placed at a spacing of 305mm (12 in) only to hold the longitudinal reinforcement cage together during casting concrete. Figure 10 shows the column reinforcements of the RC and the long carbon columns.

The FRP tubes were embedded 305 mm (12 in) into the footings to provide sufficient development length, while the embedment into the column heads was only 152 mm (6 in). Considering the horizontal testing configuration of the columns, the specimens were cast horizontally in two batches with 44.8 MPa (6.5 ksi) and 31.0 MPa (4.5 ksi) 28-day compressive strength. The ready-mix concrete achieved a slump of 254 mm (10 in), equivalent of a pump mix, to ensure proper placement of concrete along the entire length of the tubes.

A pedestal was designed to accommodate the height of the reaction frame and the actuator, and also to span over the tie-down pattern. Each specimen is placed on top of the pedestal, and post-tensioned with eight threaded rods to the strong floor through the pedestal and four threaded rods in the middle to the pedestal. All threaded rods are 25.4 mm (1 in) diameter. Figure 11 depicts the test set-up of specimen CL, i.e., the long column with carbon FRP tube. Tests are currently in progress at FIU.

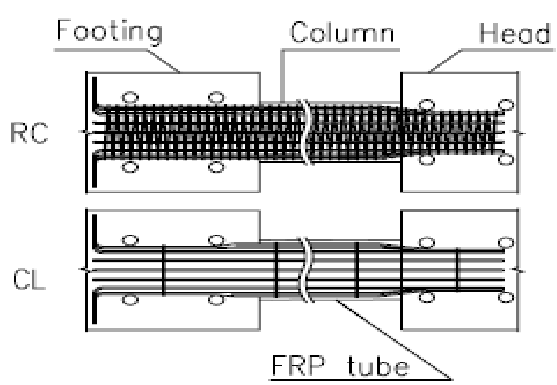

Figure 10. Column Reinforcement for RC and CL.

Figure 11. Test Set-up for Long Carbon Tube.

2. Two-Column Bent Test Program: A total of four (4) bent specimens are prepared with three CFFT and one control RC. The formwork of RC column is by using 203 mm (8 in) sonotube and two of the FRP tubes is made by the industry using filament winding of 17 layers of +55o E-glass fibers and epoxy resin, with an inside diameter of 203 mm (8 in) and a wall thickness of 5.1 mm (0.2 in). The other four FRP tubes are carbon and hybrid FPR tubes and prepared in the SCL at FIU. The process of making the FPR tubes and material properties are same as described in single column test above. The layers of the FRP material is kept as same as that in the single column tests. Table 2 shows the test matrix of two-column bent experiment.

Table 2.Test Matrix of Two-Column Bent Experiments.

Specimens	Columns	Pier Cap Beam
RCF (Reinforced Concrete Frame)	RC	RC
CFF (Carbon FRP-Concrete Frame)	Carbon FRP	Carbon FRP
GFF (Glass FRP-Concrete Frame)	Glass FRP	Carbon FRP
HFF (Hybrid FRP-Concrete Frame)	Hybrid FRP	Carbon FRP

In the bent specimens, the columns are 203 mm (8 in) diameter, while the reinforcement of the column and pier cap beam are scaled down accordingly. All eight columns had the same longitudinal reinforcement of eight 10mm (No.3) steel bars of Grade 414 MPa (60 Ksi) along the entire length of the columns, with adequate embedment into the footing and the pier cap beam. The pier cap beams for all CFFT bents are cast into carbon FRP formwork, which is made by wrapping bi-directional carbon FRP fabric onto a wooden mold. Figure 12 shows the wooden mold for carbon FRP formwork of the pier cap beams. The lateral load is transferred to the pier cap beam through an adapter and four 25.4 mm (1 in) rods outside of the pier cap beam. Two 25.4 mm (1 in) threaded rods are used to simulate the vertical load of the bridge through two 1.25 inch PVC ducts by prestressing. Two Enerpac jacks are interconnected to apply same level of force. Pressure relief valve and hand pump are used to make adjustments during the test to make sure the vertical load is maintained at the same level. Figure 13 shows the elevation view of the two-column bent specimen. The test layout is shown in Figures 14 and 15.The height and span of the two columns in the specimen is adjusted to 1,550 mm (61 in) and 1,270 mm (50 in) based on the large-scale columns in the third phase discussed below. At the bottom of each specimen, four 25.4 mm (1 in) threaded rods are placed inside the footing, and are then connected to the frame with a W-shape steel column which provides lateral resistance during the loading. Two HSS 14×6 steel beams are placed on top of the footing at each end to provide vertical resistance during the tests. Two concrete blocks are used on each side of the specimen to prevent lateral rotations. This phase is currently at the stage of specimen preparation.

Figure 12. Wooden mold Pier Cap Beam.

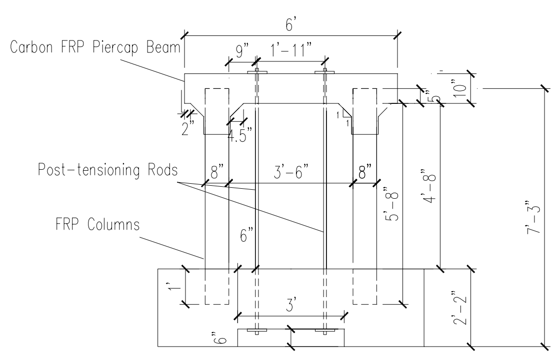

Figure 13. Two-Column Bent Specimen.

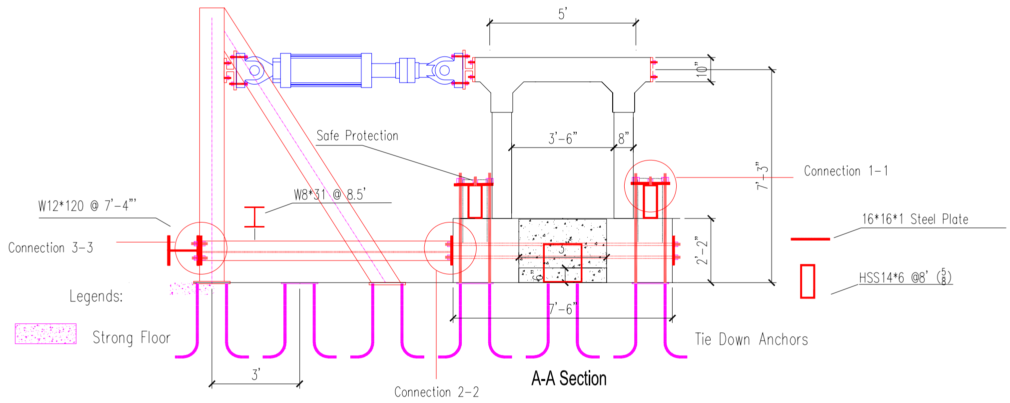

Figure 14. Elevation View of Two-Column Test Layout.

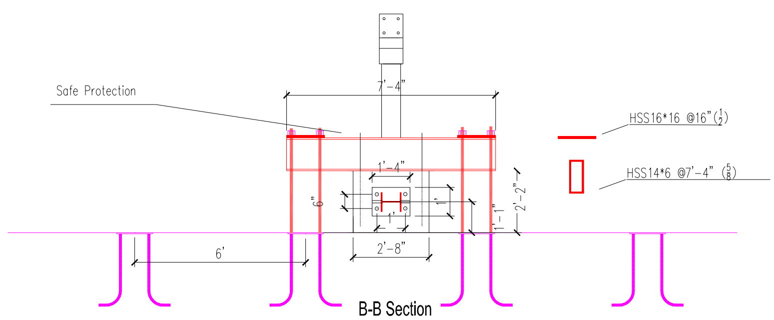

Figure 15. Cross-Section View of Two-Column Test Layout.

3. Four-span Bridge Model Test Program: A large-scaled four-span bridge model will be tested on three bi-directional shake tables at the University of Nevada, Reno to assess the holistic performance of the proposed system. Figure 16 shows the tentative layout of the shake table test of the bridge model. Using the data gained from single column and two-column bent tests, the bridge columns and pier cap beams will be designed and tested on the shake tables to verify and improve the design.

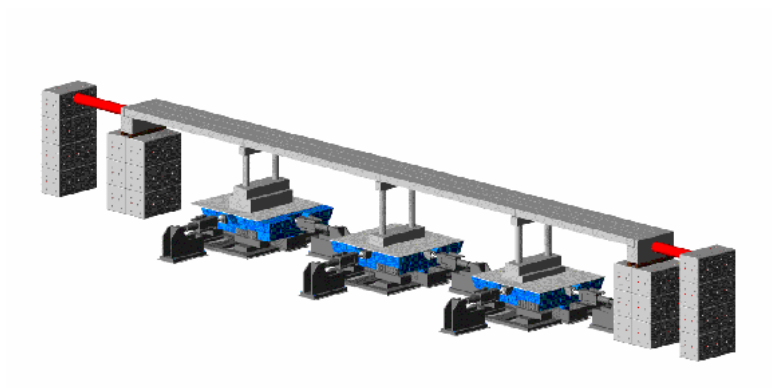

Figure 16. Tentative Layout of Shake Table Test of the Bridge Model.

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