Seismic Response of Curved Highway Bridge with Seismic Isolation and Hybrid Protective Systems

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SUMMARY:
This paper presents the results of an experimental investigation conducted on a 0.4-scale model of a highly curved, 3-span, steel girder bridge with two different configurations of protective systems using multiple shake tables. In the first configuration isolators are provided at all supports ("full isolation") while in the second configuration isolators are only provided at the abutments ("hybrid isolation"). The isolators in the first case were designed such that the columns remained elastic under the design earthquake. On the other hand, the isolators in the latter case were designed to not only keep the columns elastic but also substantially reduce the superstructure displacements. It is shown that both techniques are successful at protecting the columns, even during the maximum considered earthquake (150% design earthquake). It is also shown that the hybrid isolation technique reduced the superstructure displacements by a factor of about one-half, but with a two-fold increase in the abutment shear forces.

Keywords: Curved Bridge, Seismic Isolation, Hybrid Isolation, Large-Scale Experiment, Multiple Shake Tables

1. INTRODUCTION

Recent earthquakes have again illustrated that highway bridges are susceptible to earthquake damage (Koacelli 1999; Chi-chi 1999; Sisqually 2001; Sichuan 2008; Chile 2010; Great East Japan 2011). Strengthening schemes for bridge columns and their footings are required but cost-effective solutions are elusive. Innovation is necessary, and seismic isolation offers promise here because it avoids the need to strengthen critical members including foundations. The isolators can be designed such that the no damage is expected in the columns and foundations at a prescribed earthquake level. However, the large displacements associated with isolated bridges can lead to either pounding against the abutment backwall or costly movement joints at the abutments. One proposed technique to reduce the superstructure displacement and still be able to achieve an elastic column, is the use of hybrid isolation where stiff, hysteretic isolators are placed at the abutments and designed to attract the loads away from the columns (Buckle and Wei 2010).

This paper presents the results of the experimental investigations conducted on a 0.4-scale, 3-span curved steel girder bridge with the following configurations:

- Full isolation configuration– isolators are placed between the superstructure and substructure at all support locations (i.e. abutments and piers).
- Hybrid isolation configuration – isolators are placed at the abutments only.

In the full isolation case, the isolators were designed such that the columns remain elastic under the design earthquake. In the hybrid isolation case, the objectives were two-fold – (a) elastic columns and (b) reduced superstructure displacements.

2. HYBRID ISOLATION

In this isolation technique, isolators are only used at the abutments while monolithic or pinned connections remain at the piers. This technique is also known as “partial isolation”. It involves placing hysteretic energy dissipators at the abutments which may be designed to attract load away from the piers and, at the same time, reduce the displacement of the superstructure. Since these dissipators must allow for thermal expansion to occur at the abutments (and other movements such as creep and shrinkage), the most suitable device is an elastomeric bearing with a large lead core, i.e. a lead-rubber isolator. The significant amount of energy that may be dissipated by these devices, particularly those with large lead cores, in combination with the inherent stiffness of most bridge abutments, reduces the superstructure displacement and thus the column forces. The possibility of pounding at the abutment backwall is greatly reduced and the cost of road joints made more reasonable. These reductions can be significant and can materially improve the capacity-demand ratio for critical members that might otherwise need strengthening in a bridge retrofit project.

However, to implement this technique, the abutments must be able to transfer forces from the superstructure to the foundation that are considerably higher than in a conventional bridge or a fully isolated bridge. In the longitudinal direction (or tangential direction in a curved bridge), it is expected that many abutments can provide this capacity by engaging the fill behind the backwall. But in the transverse direction (or radial direction in a curved bridge), the back fill is not effective and the capacity of the abutment piles may be exceeded in this direction. One way of protecting the piles is by limiting the magnitude of the forces transferred from the superstructure, and this may be achieved by providing a yielding component in the transverse load path, such as a ductile end cross-frame. In the hybrid isolation experiment discussed in this paper, buckling restrained braces (BRB) are used as the yielding components.

3. CURVED BRIDGE MODEL

3.1. Description

The 0.4-scale bridge model used in the experimental investigations is a 3-span, steel I-girder bridge with high degree of curvature (subtended angle is 104^{o} (1.8 radians)). The overall geometry of the prototype and the model is summarized in Table 3.1. The superstructure comprises of a reinforced concrete deck that is composite with three steel I-girders. The reinforced concrete deck is 83 mm thick with 19 mm haunch. The girders are built-up sections consisting of 16 mm by 229 mm flange plates and 10 mm by 660 mm web plate. The piers are single columns with a drop cap. Figure 3.1 shows the as-built bridge model inside the Large-Scale Structures Laboratory at University of Nevada, Reno.

(a) view along the deck (b) view underneath the deck

Figure 3.1. Bridge model assembled on the shake table array in the Large-Scale Structures Laboratory

The column diameter in the model is 0.61 m (1.52 m in the prototype) with 1% longitudinal and transverse steel ratios. The specified concrete strength was 38 MPa and the steel reinforcement is A706 steel.

Figure 3.2 shows the plan view of the bridge inside the laboratory. Abutment 1 is on a 6 degree-of-freedom shake table while Piers 2 and 3 and Abutment 4 are on biaxial shake tables. The orientations of tangential and radial axes at supports referred to in the subsequent sections are also shown in Figure 3.2.

The weight of the bridge model is 563 kN (not including the footings) and the added weight used to satisfy similitude requirements is 833 kN. Thus, the total weight of the bridge model is 1,396 kN. The added weight comprised steel plates and lead bricks distributed on the bridge deck and the top of the bent caps. Figure 3.3 shows the bridge model with the added weight distributed on the deck.

Table 3.1. Overall geometry of the curved bridge

	Prototype	Model
Total length (m)	110.5	44.2
Span lengths (m)	32 - 46.5 - 32	12.8 - 18.6 - 12.8
Centerline radius (m)	61	24.4
Total width (m)	9.15	3.66
Girder spacing (m)	3.4	1.37
Column height (m)	6.1	2.44
Column diameter (m)	1.52	0.61

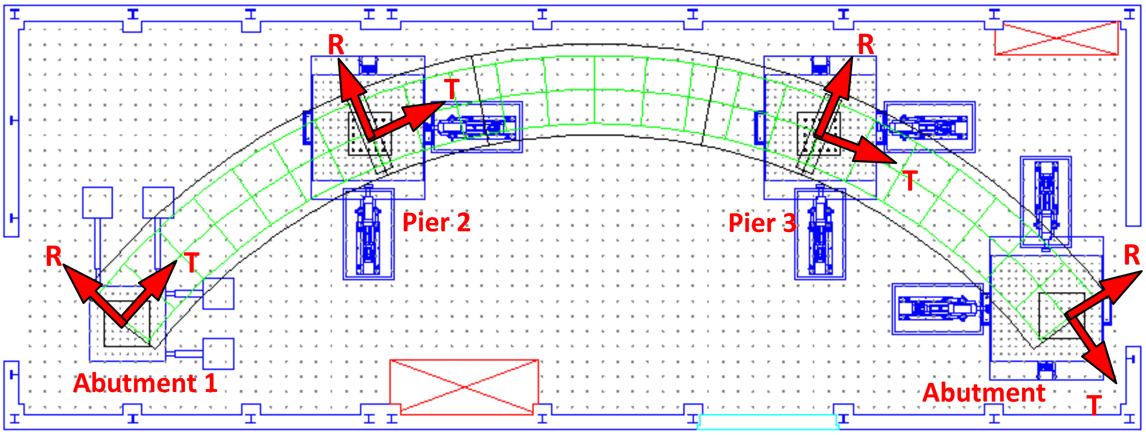


Figure 3.2. Bridge model plan view and support tangential (T) and radial (R) axes

Figure 3.3. Added weight on bridge deck

The curved bridge and its components were designed based on the 2008 AASHTO LRFD Bridge Design Specifications (AASHTO 2008). The design spectrum was based on a rock site in Seismic Zone 3. The peak ground acceleration was 0.47 g, the short-period spectral acceleration (S_S) was 1.135 g, and the 1-second spectral acceleration (S_1) was 0.41 g.

This paper reports the experimental results for full and hybrid isolation, but it is noted that this model was also used to experimentally study the effect of:

- curvature,
- live load,
- rocking columns, and
- abutment-backfill interaction.

The findings of these experimental studies, coupled with analytical investigations, will be used to develop a set of seismic design guidelines for horizontally curved steel bridges.

The results presented herein were obtained using the Sylmar record of the 1994 Northridge Earthquake as input motion. The Sylmar ground motion was scaled by 0.475 such that the spectral acceleration at 1.0 second was equal to the 1-second spectral acceleration of the design spectrum (0.41 g). This scaled motion was considered to be the Design Earthquake (DE) for the purpose of this experiment. It was applied in increments of 10%, 20%, 50% 75%, 100% 125% and 150%, the last case being the Maximum Considered Earthquake (MCE).

The boundary conditions in the conventional bridge referred to in Sections 4.2 and 4.5 are the following:

- At abutments – free translation in the tangential direction and restrained in the radial direction by a failing sacrificial key. The shear key was designed to fail at 75% DE thus the abutment is free in translation in any horizontal direction at earthquake levels greater than 75% DE.
- At piers **-** pin connection between the superstructure and pier cap using pot bearings.

It is noted that the column properties in the conventional, full isolation, and hybrid isolation cases are the same.

3.2. Curved Bridge with Full Isolation

In this configuration, lead-rubber bearings (LRB) were placed between the superstructure and the substructure. These isolators were designed such that the columns remain elastic (no or minimal yielding in the column) at 100% DE. There are two sets of isolators – one at the abutments and one at the piers. The abutment isolators have a bonded diameter of 191 mm with a 32 mm lead core. The pier isolators have a bonded diameter of 229 mm with a 38 mm lead core. Isolator properties are summarized in Table 3.2.

Table 3.2. Properties of LRB Isolators used in the Full Isolation case

Parameters	Abutment Isolators	Pier Isolators
Shear modulus, G (MPa)	0.41	0.41
Modulus of elasticity, E (MPa)	1.24	1.24
Bonded diameter, B (mm)	191	229
Layer thickness, tr (mm)	6.35	6.35
No. of layers, n	11	11
Total rubber thickness, Tr (mm)	70	70
Total Height, H (mm)	178	178
Lead core diameter, dL (mm)	32	38
Bonded Area, Ab (mm2)	27,848	40,053
Stiffness, Kd (N/mm)	165	236
Characteristic strength, Qd (kN)	6.27	9.03

3.3. Curved Bridge with Hybrid Isolation

In this configuration, LRB isolators were only used at the abutments while conventional pot bearings were used at the piers. These isolators were sized such that the column performance would be the same as that in the full isolation case (i.e. elastic behavior) at 100% DE. Thus, the isolators are larger in size and have larger lead cores to increase hysteretic damping. As explained previously, the stiffer isolators attract load away from the piers reducing the demand. The period of the hybrid isolated bridge is about 0.5 sec at 100% DE which is similar to that of a conventional (non-isolated) bridge. The superstructure displacement is therefore less than that of the fully isolated bridge where the period is more than twice as long (about 1.2 sec at 100% DE).

Table 3.3 shows the properties of the LRBs used in the hybrid isolation case. The bonded diameter is 279 mm and the lead plug core is 79 mm in diameter. Material properties are similar to those used in the full isolation case.

As mentioned previously, buckling restrained braces (BRB) were used as diagonal members of the end cross-frames at the abutments. These devices limited the radial shears at the abutments by yielding in tension and compression without buckling. Shear keys were provided to restrain the LRBs in the radial direction and improve the effectiveness of the BRBs. Thus, the LRBs dissipated energy in the tangential direction, while the BRBs dissipated energy in the radial direction.

Table 3.3. Properties of LRBs used in the Hybrid Case

Parameters	Abutment Isolators
Shear modulus, G (MPa)	0.41
Modulus of elasticity, E (MPa)	1.24
Bonded diameter, B (mm)	279
Layer thickness, tr (mm)	6.35
No. of layers, n	8
Total rubber thickness, Tr (mm)	51
Total Height, H (mm)	178
Lead core diameter, d_L (mm)	79
Bonded Area, A_b (mm^2)	56,235
Stiffness, K_d (N/mm)	459
Characteristic strength, Q_d (kN)	39.1

4. EXPERIMENTAL RESULTS

Bridge periods, superstructure displacement, isolator performance, column performance, and base shear are presented in this section for the Sylmar input motion, and the results compared for the full isolation and hybrid isolation cases.

4.1. Bridge Periods

Table 4.1 shows the periods of longitudinal and transverse vibration modes of the conventional, fully isolated, and hybrid isolated bridges. In the conventional case, the longitudinal and transverse periods are 0.50 and 0.53 sec, respectively. These were determined by subjecting the bridge to small-amplitude random vibrations. On the other hand, because isolator stiffness is dependent on its displacement, the vibration periods of full and hybrid isolation cases were evaluated at 100% and 150% DE. It is shown that the full isolation period is about 2 and 3 times the period of the conventional case. In the hybrid case, the periods at 100% DE are the same as those in the conventional case. At 150% DE, its longitudinal vibration period increased to 0.90 sec because of increased isolator displacement but the transverse vibration period remains the same because the bridge is restrained in the radial direction at the abutments.

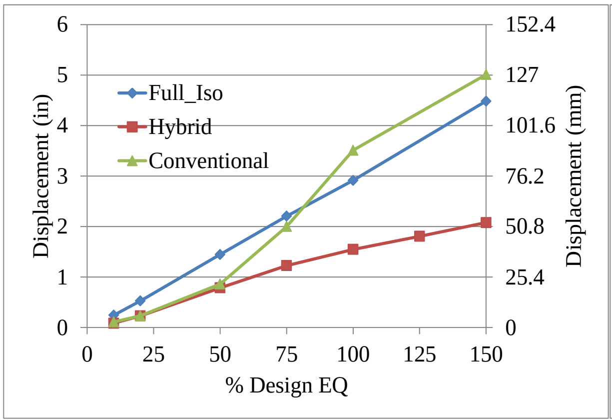
4.2. Superstructure Displacement

Figure 4.1a shows the displacement of the deck at the center of the bridge (i.e. at the mid-span of Span 2) full and hybrid isolation and conventional cases. As expected, the fully isolated case has larger displacements than the hybrid case because it is more flexible. At 100% Design Earthquake (DE), the deck displacement in the fully isolated case was 74 mm and at 150% DE, the deck displacement was 114 mm. In the hybrid case, the deck displacement at 100% DE was 39 mm and at 150% DE it was 53 mm. Thus, the superstructure displacement in the hybrid isolation case is 47.3% and 53.5% less than the superstructure displacement in the full isolation case, for the 100% and 150% DE motions respectively. This reduction depends on the stiffness of the isolators used at the abutments. If the abutment isolators were more flexible, the reduction in deck displacement would be less. If these isolators were stiffer, the reduction in deck displacement would be even greater.

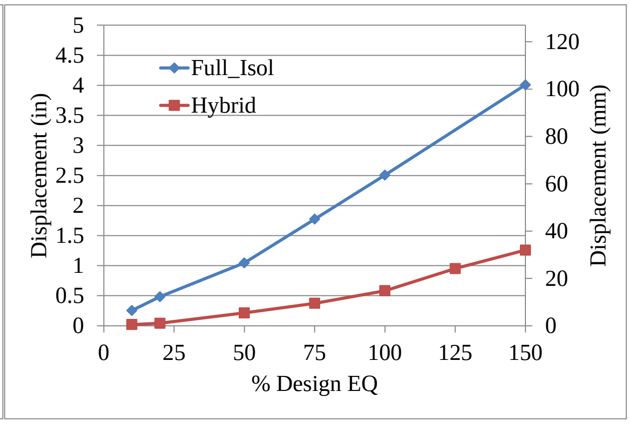
The deck displacements of full isolation case at earthquake levels below 75% DE are larger than those in the conventional case, as expected. However, beyond the 75% DE, the conventional case has more displacement than the full isolation. This is because the shear keys failed at 75% DE and all the seismic forces are taken by the columns at higher earthquake levels. The increased demand caused significant yielding in the columns resulting to significant reduction in the bridge stiffness.

Table 4.1. Comparison of vibration mode periods

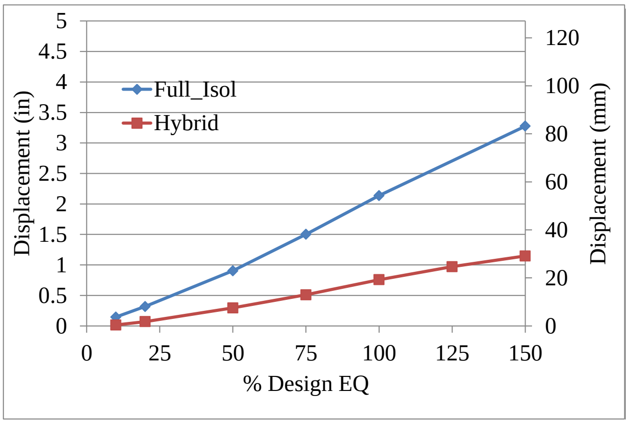
Bridge		Longitudinal Vibration Mode	Transverse Vibartion Mode
Conventional		0.50 sec	0.53 sec
Full Isolation	at 100%DE	1.00 sec	1.00 sec
	at 150% DE	1.43 sec	1.43 sec
Hybrid Isolation	at 100%DE	0.50 sec	0.55 sec
	at 150% DE	0.90 sec	0.55 sec



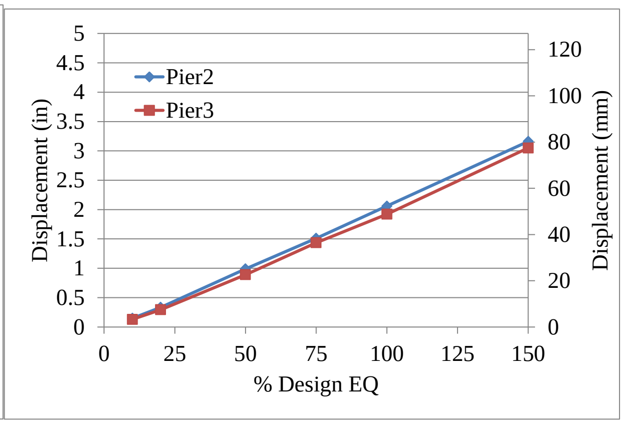
(a) Deck displacement



(b) Isolator displacements at Abutment 1



(c) Isolator displacements at Abutment 4



(d) Isolator displacements at Piers

Figure 4.1. Resultant deck and isolator displacements

4.3. Isolator Displacements

Figures 4.1b and 4.1c show the isolator displacements at the abutments for the full and hybrid isolation cases. Similar to the above observation, the hybrid isolation case has smaller isolator displacements than the full isolation case due to stiffer isolators and stiffer bridge system.

In the full isolation case, the isolator displacements at Abutments 1 and 4 at 100% DE are 64 mm and 54 mm, respectively. At 150% DE, the isolator displacements are 102 mm and 83 mm at Abutments 1 and 4, respectively. The difference in the isolator displacements between Abutments 1 and 4 is attributed to the in-plane torsion in the bridge caused by the curved geometry. This trend was also observed at other levels of earthquake shaking.

The above observation in the isolator displacements at the abutments of the full isolation bridge is also true in the isolator displacements at the piers (Figure 4.1d). Although the difference is smaller compared to the abutments, the isolator displacements at Pier 2 are always greater than the isolator displacements at Pier 3. This is because the piers are closer to the center of stiffness (center of rotation) of the bridge and thus the in-plane torsional effect is less. The average isolator displacements at the piers at 100% DE and 150% DE are 51 mm and 79 mm, respectively. These displacements are smaller than those at the abutments because of the flexibility of columns. The isolators are springs acting in series with the columns and the sum of the isolator and column displacements should be of the same order of magnitude as the abutment isolators.

Also shown in Figure 4.2 are the isolator displacements at the abutments for the hybrid configuration. Unlike the observation made above for the full isolation case, the abutment isolator displacements in the hybrid case are about the same. The effect of in-plane torsion is less because the isolators in the hybrid case are active only in the tangential direction. It is noted that, as mentioned previously, the radial direction at the abutments are restrained by shear keys. Thus, the displacements shown for the hybrid case are tangential isolator displacements.

4.4. Support Shear Forces

4.4.1 Abutments

Figure 4.2 shows the total tangential and radial shear forces at abutments. These forces were taken from the load cells located underneath each isolator. It is noted that the radial shears do not include those taken by the shear keys. However, these shears are limited by the yield capacity of the BRBs to approximately 80 kN.

The total tangential shears in the full isolation case at Abutments 1 and 4 are 97.74 kN and 63.03 kN, respectively, at 150% DE. In the hybrid case, they are 246.33 kN and 236.59 kN, respectively. This corresponds to an increase by a factor of 2.7 at Abutment 1 and 3.8 at Abutment 4.

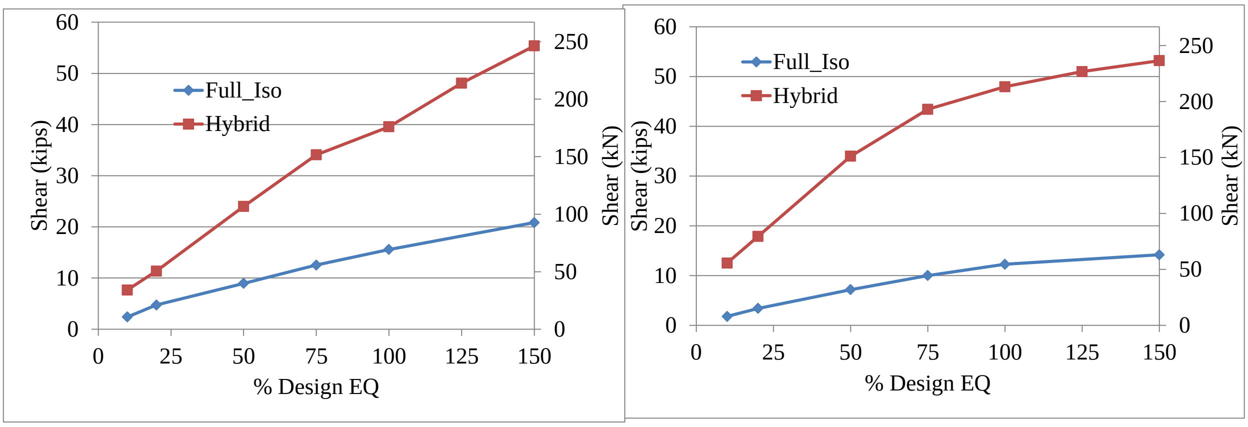
The radial shears in the full isolation case increases linearly with the earthquake level and at 150% DE they are equal to 48.66 kN and 72.32 kN at Abutments 1 and 4, respectively. In the hybrid case, however, the radial shear started to level off at about 60 kN after the 75% DE because the BRB started yielding. Theoretically, if the shear key is perfectly aligned in the radial direction, the radial forces in the isolators should be zero because they should be taken by the shear key. The recorded isolator radial shear could be due to either slight misalignment of the shear key which in turn made the isolator to deform radially or slight misalignment of the load cells or a combination of both. Readings from the displacement transducer show a maximum radial deformation of 3 mm. Although this is small, the corresponding isolator shear forces could be considerable, and as shown in Figures 4.2c and 4.2d they are 55.87 kN at Abutment 1 and 68.10 kN at Abutment 4. However, these are only about half of the total Q_d which is equal to 117.3 kN (the Q_d per isolator is 39.1 kN as shown in Table 3.3).

4.4.2 Piers

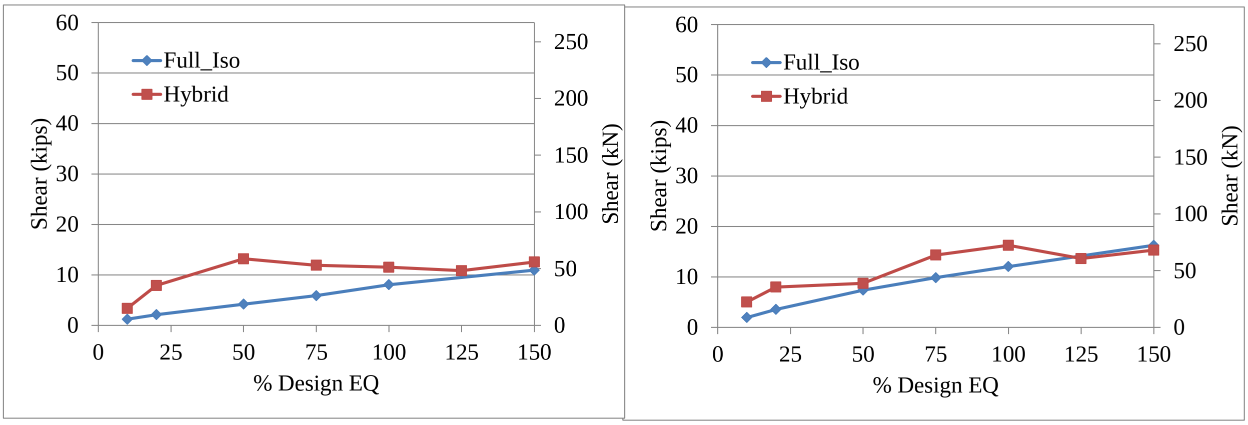
Assuming single curvature behavior, the column shear at first yield (i.e. onset of rebar yielding) is equal to 111.35 kN, while the shear at effective yield is equal to 154.61 kN.

Figure 4.3 shows the resultant shear forces at the piers. These forces were taken from the load cells located underneath each isolator and pot bearings. For the full isolation case, the resultant isolator shear forces at Piers 2 and 3 are 75.72 kN and 88.09 kN, respectively, at 100% DE. At 150% DE these forces are 94.20 kN and 117.48 kN, at Piers 2 and 3 respectively. Thus, the rebar in Pier 3 started yielding during the 150% DE. In fact, readings from the strain gages in the column show that several longitudinal rebars has yielded.

In the hybrid isolation case, the resultant bearing shear forces at Piers 2 and 3 are 94.19 kN and 72.25 kN, respectively, at 100% DE. Therefore, the objective of keeping the column elastic during 100% DE was achieved. The bearing shears at 150% DE at Piers 2 and 3 are 126.52 kN and 102.99 kN, respectively. Readings from the strain gages in the column show that several rebars have yielded. However, at this earthquake level, the column can still be considered as essentially elastic because the shear forces are still below the effective yield shear force.



(a) Tangential shear forces at Abutment 1 (b) Tangential shear forces at Abutment 4



(c) Radial shear forces at Abutment 1 (d) Radial shear forces at Abutment 4

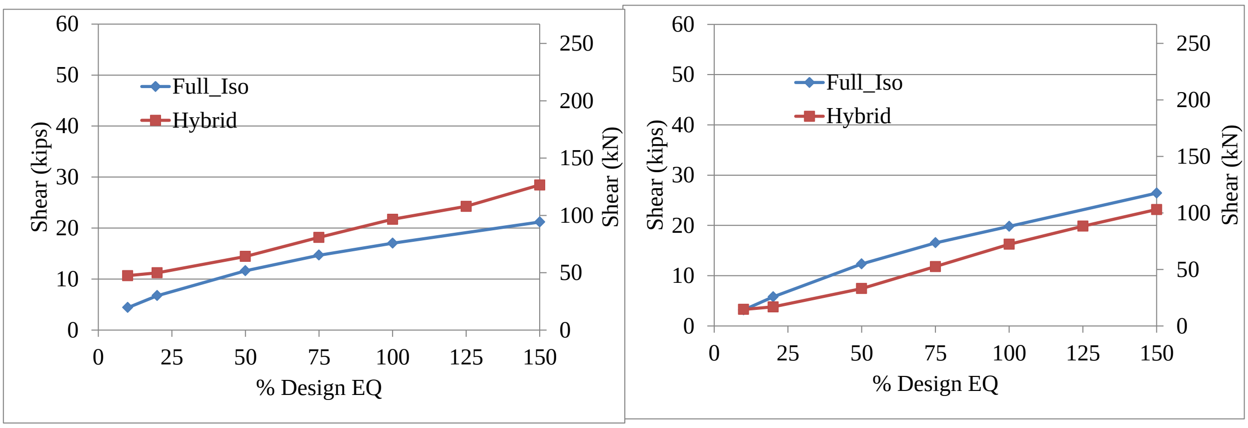
Figure 4.2. Isolator tangential and radial shear forces at abutments

4.5 Column Performance

The damage to the columns during 150% of the Design Earthquake is shown in Figure 4.4 for the conventional and full isolation cases. It is apparent that there was little cracking in the columns of the full isolation case.

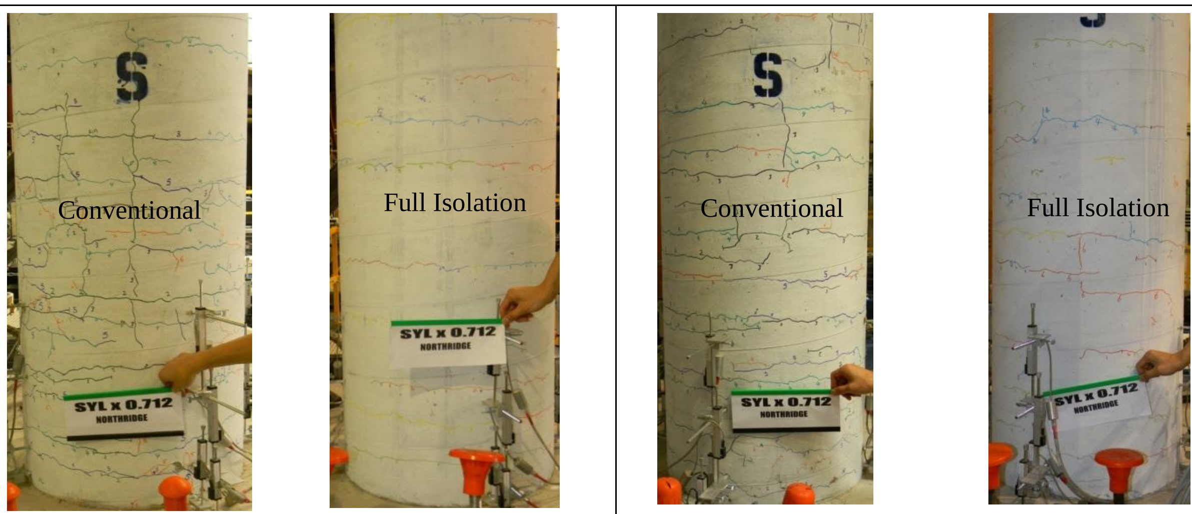
Readings from the strain gages show that all the longitudinal (8 – 16mm) rebars at Piers 2 and 3 have yielded in the conventional case. The maximum strain recorded at Pier 2 was 26,000 $\mu\varepsilon$ while at Pier 3 it was 24,500 $\mu\varepsilon$. In the full isolation case, 3 rebars have yielded at Pier 2, while 2 rebars have yielded at Pier 3. The maximum strain at Pier 2 was 6,000 $\mu\varepsilon$ while at Pier 3 it was 4,000 $\mu\varepsilon$. It is noted that the yield strain of the rebar is 2,400 $\mu\varepsilon$.

Although not shown here, the performance of the columns in the hybrid isolation case is similar to the full isolation case. As shown in Figure 4.3, the support shears in both isolation cases are about the same.



(a) At Pier 2 (b) At Pier 3

Figure 4.3. Resultant shear forces at piers



(a) Column at Pier 2 (b) Column at Pier 3

Figure 4.4. Comparison of damage to columns at 150% Design Earthquake

5. SUMMARY AND CONCLUSIONS

This paper has presented the response of a curved bridge with seismic isolators located at all supports (full isolation configuration) and with isolators only at the abutments (hybrid isolation configuration). The experimental investigations were carried out on a 0.4-scale model of a highly curved, 3-span, steel girder bridge using the NEES shake table array at the University of Nevada Reno. In the full isolation case, the isolators were designed such that the columns remained elastic during the design earthquake. In the hybrid isolation case, the objectives were two-fold: (a) elastic columns in the design earthquake and (b) significantly reduced superstructure displacements.

By comparing the response of the bridge with full and hybrid isolation, the following conclusions were made:

- Hybrid isolation is effective at reducing the superstructure displacement. In this experiment the reduction was about one half, which reduces the possibility of pounding at the abutment back-walls and requires significantly smaller movement joints at the abutments.

- Full isolation is effective at keeping the columns elastic under the design earthquake and essentially elastic under the maximum considered earthquake.
- Hybrid isolation is also effective at keeping columns elastic under the design earthquake and essentially elastic under the maximum considered earthquake.
- Hybrid isolation increases the shear force demand on the abutments. In this experiment the increase was more than a factor of 2 compared to the fully isolated case.

It has been shown that the design objectives of the two isolation techniques were achieved. Although both were effective in keeping the columns elastic during the design earthquake, each technique has its own advantages and disadvantages. The full isolation case has greater superstructure movement but places less demand on the abutments. On the other hand, the hybrid isolation case has less superstructure movement (smaller movement joints) but greater abutment forces.

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