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# The effect of pier and deck connection on the seismic response of U-turn curved bridge

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## ABSTRACT

The maximum seismic response of irregular U-turn curved bridge is significantly related to the pier and deck connection. An irregular U-turn curved bridge with radius of 45m, 8.5m total width and 154m total length was analyzed in this paper. This highly horizontally curved bridge was evaluated by performing nonlinear time history analysis on the representative bridge model with 3 types of connection between pier and deck: (i) rigid connection, (ii) hinge connection, and (iii) lead rubber bearing connection. Nonlinear hinge was modelled at the base and the top of the pier by using fiber hinge. Moreover, the effect of the number of connection between pier and deck was evaluated. The results indicate that bridges with one connection give better performance level than those with two connections; however, bridges with only one connection is more critical to the torsion issue.

## 1. INTRODUCTION

The long distance on the freeway is a major issue for drivers who need to make a U-turn. To solve this major issue, drivers need a U-turn curved bridge on the freeway, so they can make a U-turn. From a civil engineering point of view, this bridge presents many challenges in its planning process. Past studies investigated that the maximum response of each bridge structure component rose due to earthquake loading in different direction (Ni, Chen, Teng, & Jiang, 2015). Past studies also showed that curved bridges are more susceptible to earthquakes than straight bridges, and the seismic response of bridges will be more dangerous as the curvature of the bridge increases (Soleimani, Yang, & DesRoches, 2017). Due to the nature of this U-turn curved bridge, the bridge should not be analyzed like any other straight bridge as it is classified as an irregular bridge under the American Association of State Highway and Transportation Officials (AASHTO) rules.

## 2. SAMPLE STRUCTURES

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The bridge is a ten-span, solid slab superstructure (depth = 950mm), supported on reinforced concrete single pier bents with the rectangular section (1400mm x 1400mm). This bridge has 8.5m total width and 154m total length. The bridge radius is 45m. The super elevation of the bridge is 10% and the slope of the road in longitudinal direction is 5%. The design speed of this U-turn curved bridge is 40 km/h. See Figure 1 for Plan View. Figure 2 shows the section of bridge's superstructure. The location of this study is Jakarta, Indonesia, with soft soil classification, and the structural design follows AASHTO LFRD Bridge Design Specifications using the modification factor, R, as 3.

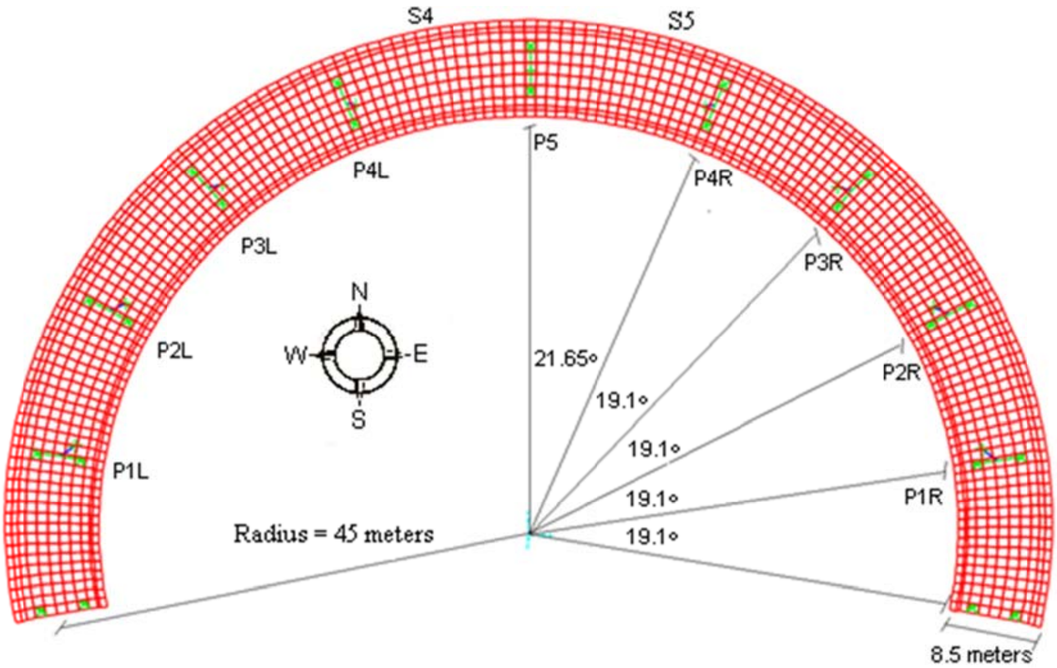


Fig. 1 Plan View

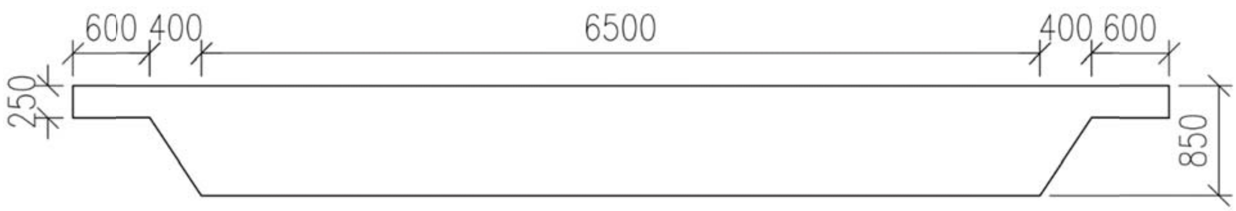


Fig. 2 Section of bridge's superstructure

The investigation and analysis include total six bridge structures, namely Rigid-1, Rigid-2, Hinge-1, Hinge-2, LRB-1, and LRB-2. All bridges have the same geometry as described before. At Rigid-1 and Rigid-2, rigid connections are used between the pier and the deck. At Hinge-1, one hinge connection is used between the pier and the deck. At Hinge-2, two hinge connections are used between the pier and the deck.

At LRB-1 and LRB-2, lead rubber bearings (LRB) are used as seismic isolators and placed between the pier and the deck. The dimensions and properties of isolators used in LRB-1 and LRB-2 are shown in Table 1. The connection between superstructure and the abutment in all analyzed bridges is lead rubber bearing. The abutments would not be included in earthquake resisting systems since the connection between superstructure and abutment is LRB.

Table 1. LRB isolators properties

| **Parameters** | **Abutment** | **LRB-1** | **LRB-2** |
|---|---|---|---|
| Stiffness of elastomer, $K_r$ (kN/mm) | 4.52 | 4.52 | 2.36 |
| Stiffness of lead core, $K_p$ (kN/mm) | 45.16 | 45.16 | 23.64 |
| Yield strength, $F_y$ (kN) | 561.78 | 561.78 | 328.51 |
| Effective Stiffness (kN/mm) | 7.52 | 7.52 | 3.86 |
| Effective Damping (%) | 29.75 | 29.75 | 29.22 |

## 3. BRIDGE MODELLING

The demand of the bridges was analyzed using Acceleration Response Spectrum Analysis (ARSA) and the capacity is analyzed using nonlinear pushover analysis for ultimate displacement. The demand and capacity were then compared to ensure the safety of the bridge. Then, Nonlinear Time History Analysis (NLTHA) was performed to bridges that meet the demand and capacity criteria that were analyzed before. Three groups of time history for NLTHA were selected and matched to represent high seismicity of Jakarta. They were chi-chi (TCU120), landers (MEL), and sitka earthquake record (212V50). Jakarta is classified as a high seismic performance zones (seismic zone 4) in accordance to values of Sds (0.837 g) and Sd1 (0.573 g). Each time history record then shall be modified to be compatible with target response spectrum (7% probability of exceedance in 75 years) using the time-domain procedure. Then, each record was rotated in 3 directions ($0^o$-$90^o$, $45^o$-$135^o$, $90^o$-$0^o$) due to the structure configuration of the bridge. For each suite of ground motion, 100% of the input ground motion in each horizontal directions are given simultaneously to structure. Direct Hilber-Hughes-Taylor was used. NLTHA was performed after nonlinear dead loads analysis. Nominal material properties were used in ARSA, while expected material properties were used in NLTHA. Table 2 shows those properties used in the analysis.

The pier was modeled as a single frame element that starts from the top of the foundation and ends at the bottom of the connection. Rigid links were used to correctly model the structural behavior between the connection and the deck. For the purpose of NLTHA, a plastic hinge was modelled at the base and the top of the pier. The plastic hinge was assigned using fiber hinge. Mander’s uniaxial nonlinear concrete model was used for modelling the stress-strain behavior of unconfined and confined concrete (Mander, Priestley, & Park, 1988). The hysteretic behavior for concrete nonlinear model used Takeda hysteresis model, while the reinforced steel nonlinear model used the kinematic hysteresis model. The length of the hinge at pier was calculated according to AASHTO Guide Specifications for LFRD Seismic Bridge Design and the location of the plastic hinge was assumed at mid-height of the hinge zone (Aviram, Mackie, & Stojadinovic, 2008).

The deck structure was modelled with elastic shell element. The elastic element was selected because nonlinear behavior was not expected in deck structure during earthquake. The super elevation (10%) and the slope of the road in longitudinal direction (5%) was considered in structure modelling. So that the height of each pier varies following the elevation of the bridge deck. The bridge model of the bridge analyzed can be seen in Figures 3 and 4 below.

Table 2. Properties of materials

| **Conrete Properties** | | | |
|---|---|---|---|
| Unconfined compressive strength | $f'_{co}$ | 35 | Mpa |
| Expected strength of $f'_{co}$ | $f'_{ce}$ | 45.5 | Mpa |
| **Reinforcing Steel Properties** | | | |
| Specified yield strength | $f_y$ | 400 | Mpa |
| Specified tensile strength | $f_u$ | 550 | Mpa |
| Expected yield strength | $f_{ye}$ | 465 | Mpa |
| Expected tensile strength | $f_{ue}$ | 655 | Mpa |

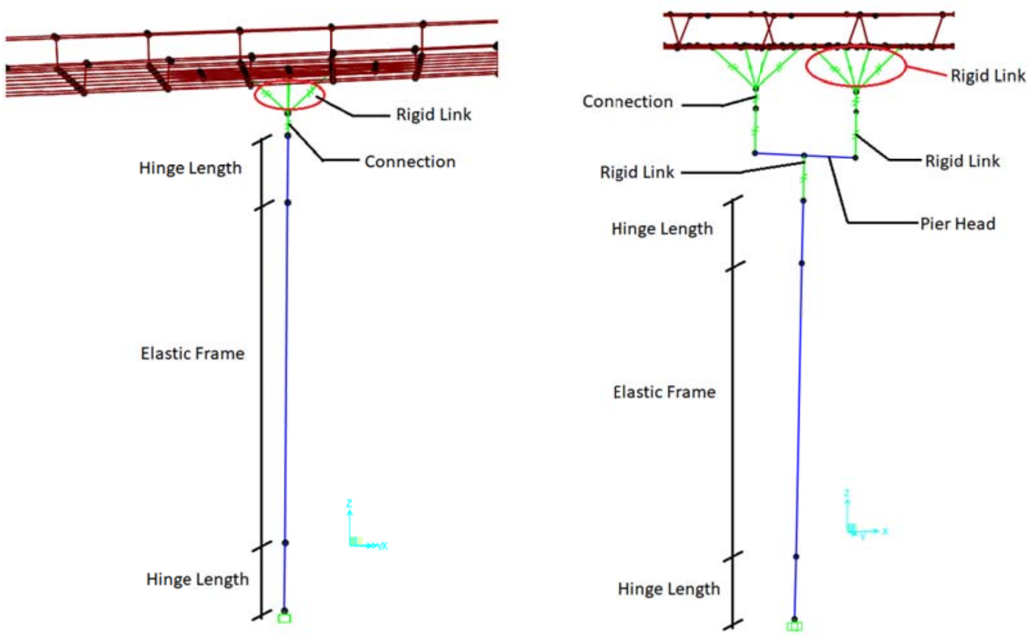


Fig. 3 Pier model with 1 connection and 2 connections

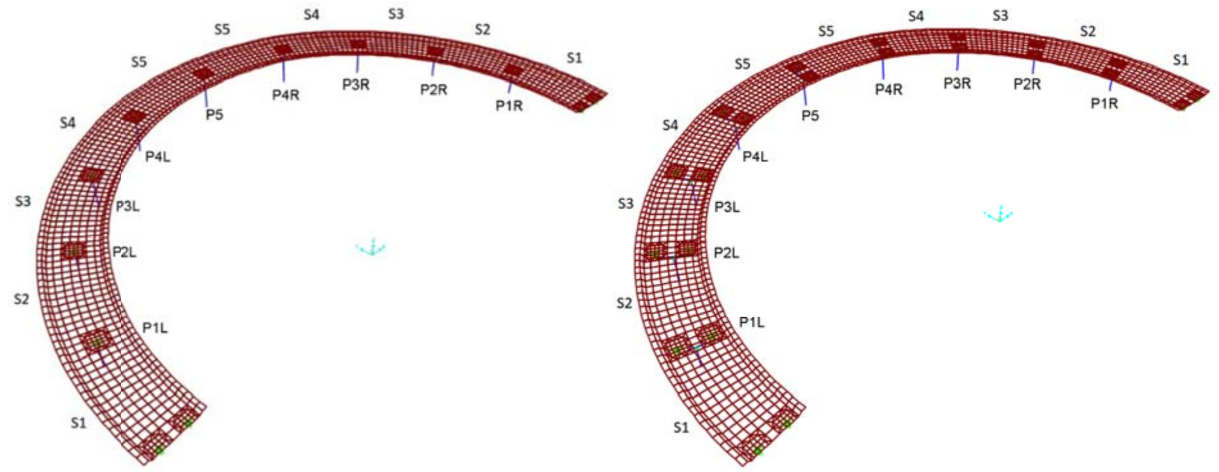


Fig. 4 Bridge model with 1 connection and 2 connections

## 4. RESULTS AND DISCUSSION

*4.1 Free Vibration Analysis*

Modal analysis was conducted and periods and shapes of vibration were obtained for all three types of connection. Values of the fundamental mode of vibration in transverse and longitudinal direction are presented in Table 3. For all of analyzed bridge models, the first fundamental mode was in E-W direction. This result was expected because the bridge was symmetrical in N-S direction, but asymmetrical in E W direction. The influence of the number of connections between column and deck was not very important regarding to their periods.

Table 3 Periods of the first fundamental mode (seconds)

| | 1 Connection | | | 2 Connections | | |
|---|---|---|---|---|---|---|
| First Fundamental Mode | Rigid | Hinge | LRB | Rigid | Hinge | LRB |
| E-W | 0.39 | 0.37 | 0.71 | 0.38 | 0.41 | 0.69 |
| N-S | 0.36 | 0.35 | 0.60 | 0.35 | 0.38 | 0.61 |
| Torsional | 0.28 | 0.30 | 0.51 | 0.27 | 0.31 | 0.52 |

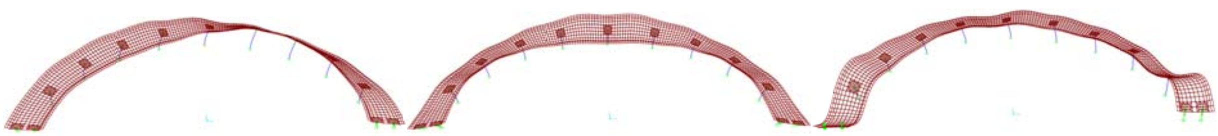

Fig. 5 Mode shapes for curved bridge (E-W, N-S, and Torsional)

### *4.2 Top Pier Displacement*

A brief summary of the maximum seismic response of the pier of all bridge models is shown in Figures 6 and 7. It can be seen that the displacement curvature curves in Figures 6 and 7 are almost similar to the shape curvature of the bridge. The reason is that the pier on the edge (P1L and P1R) is stiffer than the pier in the middle of the bridge (P5) due to its height. It also can be seen that the maximum displacement occurs in the bridge model with two connections. However, there are no significant differences in the seismic response comparing the one connection and two connections in a rigid model.

From Figure 6 and Figure 7, it can be seen that the lowest displacement at the top of pier P5 occurs at the bridge with LRB and rigid connection. The reason is that the bridge with rigid connection tend to have more redundancy than the one with hinge or LRB connection. While the bridge with LRB connection tend to have more energy dissipation than the one with rigid or hinge connection.

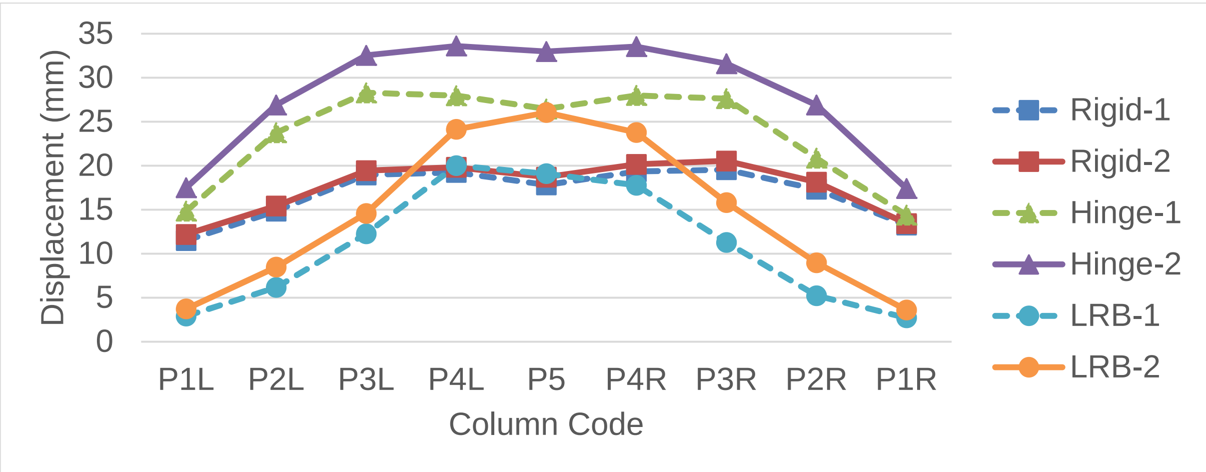


Fig. 6 Top pier displacement (E-W direction)

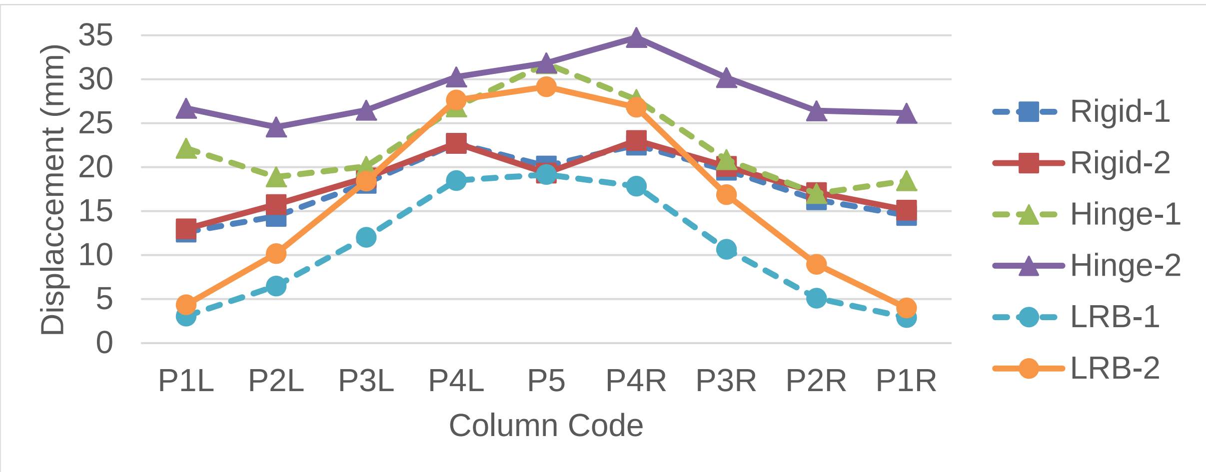


Fig. 7 Top pier displacement (N-S direction)

*4.3 Pier Base Shear*

From Figure 8 and Figure 9, the influence of the number of connections of U-turn curved bridge is not significant regarding to their seismic response, except the one with the hinge model in Figure 8. It can be seen that the Hinge-1 and Hinge-2 curves in Figures 8 and 9 are almost similar to the shape of the curved bridge. Therefore, the seismic response of the bridge with hinge connection depends on the pier stiffness, while the seismic response of the bridge with LRB connection is governed by LRB stiffness. The seismic response of the bridge with rigid connection in E-W direction does not depend on the pier stiffness. Its base shear curve in Figure 8 is almost similar to the response of the straight bridge, while its base shear curve in N-S direction connection similar to the shape of the curved bridge.

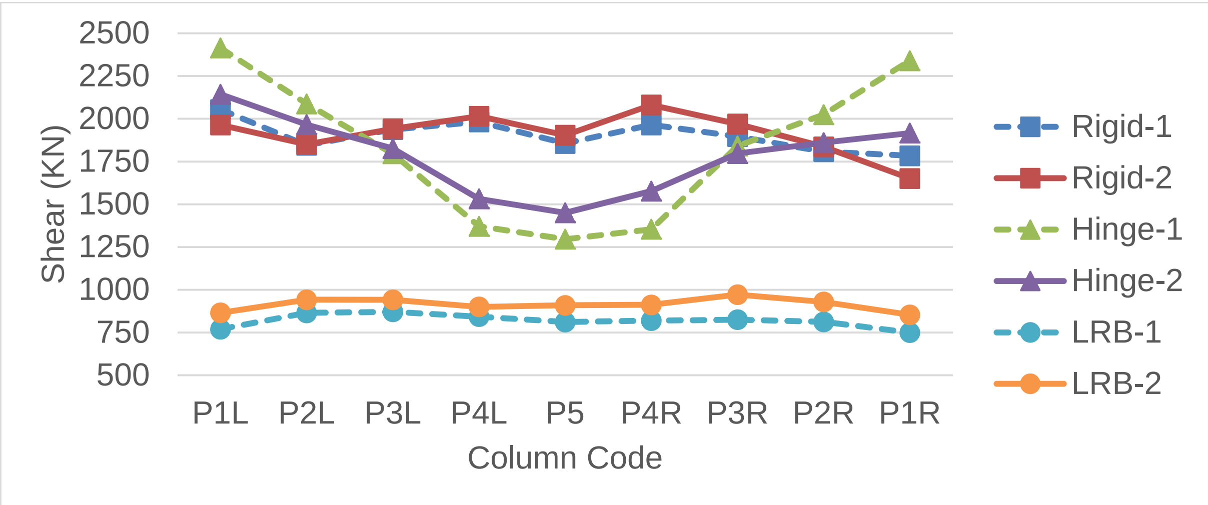


Fig. 8 Base shear in each pier (E-W direction)

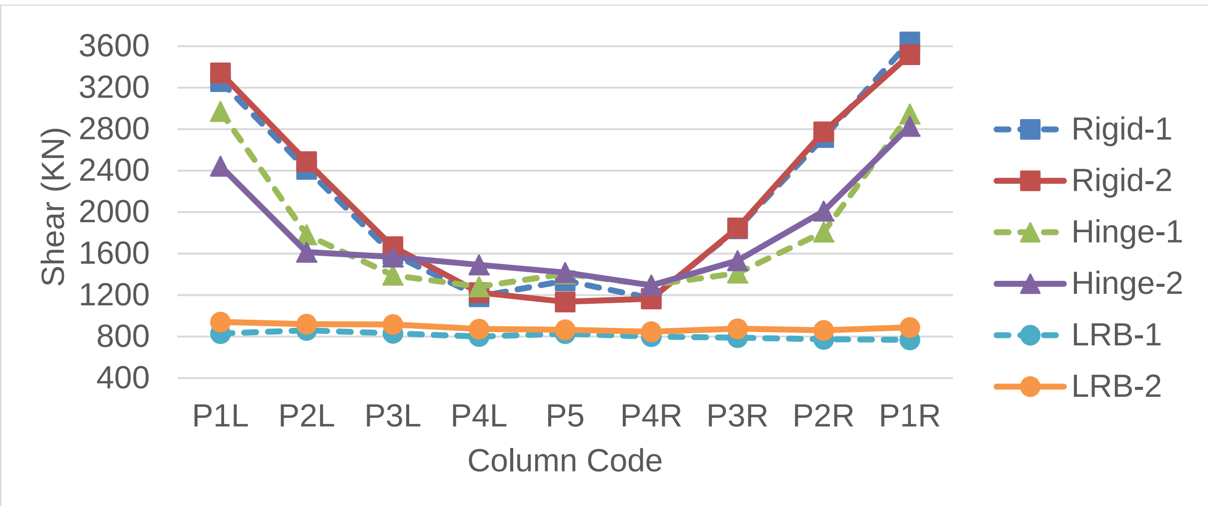


Fig. 9 Base shear in each pier (N-S direction)

*4.4 Deck Longitudinal Rotation*

Figure 10 and 11 shows the longitudinal rotation at deck S4 and S5 (shown in Figure 1). The "B" in Figure 10 and Figure 11 shows the width of the bridge deck which is 8500 mm. It can be seen that the longitudinal rotation of bridge deck with one connection is two times larger than the bridge with two connections, except the bridge with one rigid connection. This observation may alert design engineers that when U-turn curved bridge is designed with only one connection, rigid connection should be used to connect the pier and the deck.

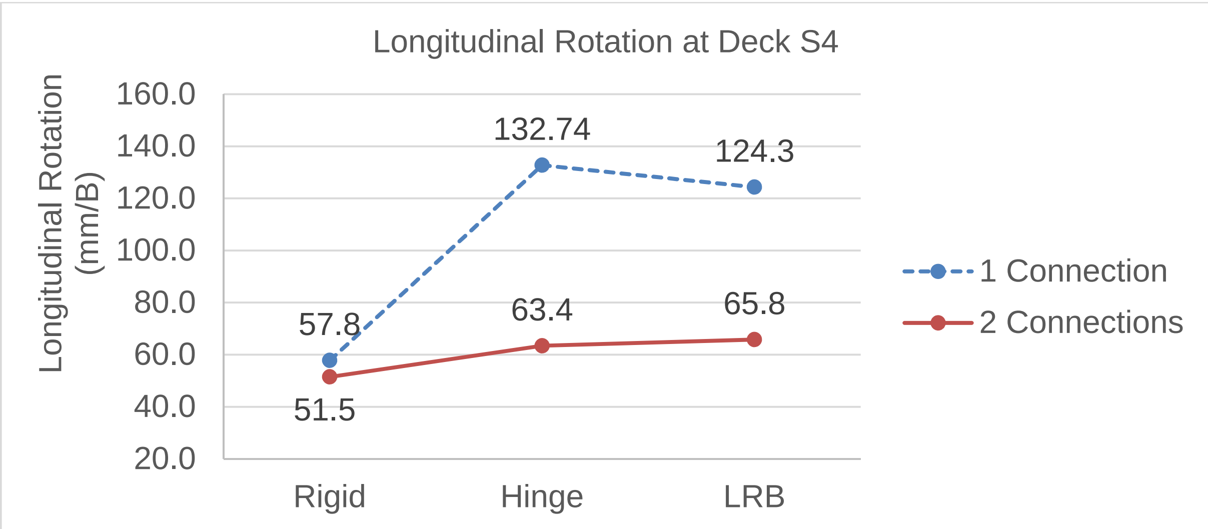


Fig. 10 Longitudinal Rotation at Deck S4

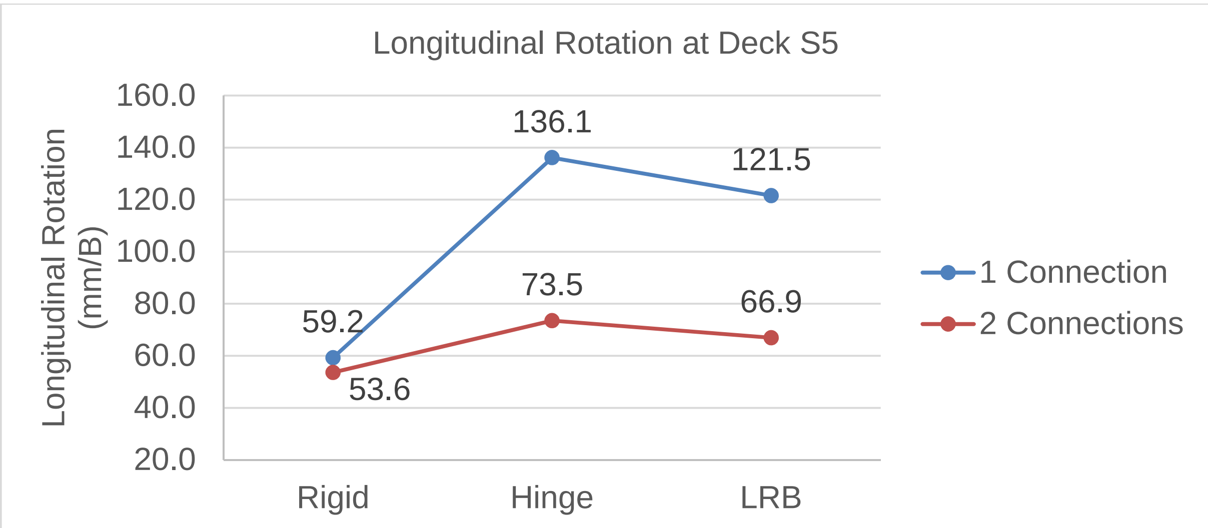


Fig. 11 Longitudinal Rotation at Deck S5

*4.5 Performance Level*

Table 4 shows the maximum rotation at plastic hinge on each pier. The performance level target on preliminary design is life safety according to AASHTO LFRD Seismic Bridge Design. It can be seen that the maximum rotation occurs at Hinge-2 and the bridge with two connections tend to have bigger plastic hinge rotation than the bridge with one connection. The performance level of each pier is calculated based on ASCE 41-13. The result shows that although the bridge was designed for life safety performance, the actual performance of all the bridge was below immediate occupancy except for Hinge-2 bridge which is immediate occupancy to life safety.

Table 4 Maximum rotation at plastic hinge

| Rotation ($x10^{-3}$ rad) | P1L | P2L | P3L | P4L | P5 | P4R | P3R | P2R | P1R | Status |
|---|---|---|---|---|---|---|---|---|---|---|
| Rigid-1 | -2.46 | -2.26 | -2.29 | -2.14 | -1.83 | -1.97 | -2.69 | -2.55 | 3.12 | <IO |
| Rigid-2 | -2.49 | -2.29 | -2.29 | -2.09 | -1.89 | -2.07 | -2.71 | -2.58 | 3.08 | <IO |
| Hinge-1 | -4.42 | -3.61 | -3.34 | -2.94 | -3.34 | -2.92 | -2.86 | 2.93 | 3.53 | <IO |
| Hinge-2 | -5.56 | -4.92 | -4.33 | 3.43 | -3.52 | 3.93 | 4.09 | 3.85 | 5.32 | IO-LS |
| LRB-1 | 0.26 | -0.64 | -1.39 | -2.20 | 2.03 | 2.00 | 1.25 | 0.44 | 0.26 | <IO |
| LRB-2 | 0.35 | -1.00 | -1.89 | -3.02 | 3.29 | 3.33 | 2.33 | 1.18 | -0.31 | <IO |

Table 5 Time history record associated to Table 4

| | P1L | P2L | P3L | P4L | P5 | P4R | P3R | P2R | P1R |
|---|---|---|---|---|---|---|---|---|---|
| Rigid-1 | LD 90 | CC 90 | CC 45 | LD 45 | LD 0 | LD 90 | LD 90 | SK 0 | CC 90 |
| Rigid-2 | LD 90 | CC 0 | CC 45 | LD 45 | LD 0 | SK 0 | LD 90 | SK 45 | CC 90 |
| Hinge-1 | CC 45 | CC 90 | CC 90 | CC 90 | SK 0 | LD 0 | LD 90 | LD 90 | CC 90 |
| Hinge-2 | CC 45 | LD 90 | LD 90 | CC 90 | SK 90 | CC 45 | CC 0 | CC 45 | CC 45 |
| LRB-1 | LD 45 | LD 45 | LD 0 | LD 0 | LD 45 | LD 90 | LD 90 | CC 45 | LD 45 |
| LRB-2 | LD 45 | LD 45 | LD 90 | LD 0 | LD 45 | LD 90 | LD 90 | LD 90 | LD 90 |

Table 5 shows the time history record (chi-chi/CC, landers/LD, or sitka/SK) and the directions ($0^o$-$90^o$, $45^o$-$135^o$, $90^o$-$0^o$) associated with Table 4. It can be seen that the maximum response of each bridge structure component arises due to earthquake loading in different direction. This observation may alert design engineers that when U-turn curved bridge was designed, the time history record or earthquake loading should be rotated to produce maximum response of each pier.

## 5. CONCLUSIONS

The comparative analysis of the seismic response of six curved bridge models by applying the nonlinear time history analysis was carried out with the aim of studying the influence of connection type variations. The conclusions are:

1. Bridges with the hinge connection have the lowest performance level and have the highest pier displacement.
2. Bridge with only one connection give better response and performance than bridge with two connections. When U-turn curved bridge was designed with only one connection, the rigid connection should be used to prevent the higher longitudinal rotation in bridge deck.
3. Even though bridges with rigid connection have the highest pier base shear than bridges with hinge connection, bridges with rigid connection have better performance than bridges with hinge connection.
4. The result showed that although the bridge was designed for life safety performance, the actual performance of almost all bridges were below immediate occupancy.

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