

Seismic Performance Evaluation of a U-Girder Highway Bridge: A Comparative Study Under Two Ground Motion Intensity Levels Using CSiBridge

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ABSTRACT

Highway bridges are vital links in transportation networks, and large earthquakes around the world have repeatedly exposed their seismic vulnerability. This paper assesses the seismic response of the Al-Yusufiyya U-girder highway bridge using a detailed finite element analysis realized in the CSiBridge environment. The ground motion intensity the paper studied contains two cases as well: a low-to-moderate seismic case (Case 1: $S_s = 0.3$ g, $S_1 = 0.1$ g, $PGA = 0.2$ g), which is the ordinary seismic hazard condition, and a high-intensity seismic case (Case 2: $S_s = 2.29$ g, $S_1 = 0.869$ g, $PGA = 0.954$ g), which is the near-fault severe earthquake scenario. This analytical framework brings together three synergistic methodologies, namely response spectrum analysis for D/C evaluation, eigenvalue modal analysis to characterize key dynamic properties (e.g. natural frequencies and mode shapes), and nonlinear static pushover analysis following ATC-40 provisions for capacity assessment and performance point definition. Results show a striking difference of structural performance between the two loading configurations. For all three bents considered in Seismic Case 1, the D/C ratios are well below unity, confirming sufficient reserve capacity. In contrast, Seismic Case 2 produces D/C ratios well in excess of 1.0 in both transverse and longitudinal directions at all bents, showing possible failure in the event of a strong seismic excitation. Push over analysis indicates that the hybrid bridge retains a capacity of nonlinear deformation under moderate loading, but that peak lateral forces and corresponding displacements increase significantly under the high-intensity scenario. Capacity-spectrum evaluation of the performance point yields confirmation that in the moderate case the structure behaves elastically, while in the severe case demand may be at or near inelastic limits. These findings highlight the important need for seismic hazard assessment and bridge multi-level performance studies at the site-specific level of seismically active regions.

Keywords: U-girder bridge; seismic performance; CSiBridge; demand-to-capacity ratio; response spectrum analysis; nonlinear pushover analysis; ATC-40; modal analysis; performance point; ground motion intensity

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1. Introduction

Highway bridges provide important continuity of transportation networks between urban zones, across rivers, and through complex landscapes in support of mobility, economic interaction, and social connectedness (Naser, 2022; Waseem & Saleem, 2019). As such, they therefore are a vital piece of infrastructure whose ability to function in the long term following extreme events has a direct impact on the efficiency with which the emergency response can continue and the overall resilience of our communities (Naser et al., 2021; Naser, 2021; Uenaga et al., 2023; Shekhar et al., 2023). Earthquakes are one of the most severe threats to bridge structures among natural hazards due to the ground motions manifested by earthquakes generating dynamic actions much larger than the normal effects of service and gravity loads (Yanan et al., 2021). Extensive previous studies, have demonstrated that certain characteristics of earthquakes such as long-duration shaking, long-period components, and pulse-like near-fault motions impart a substantial increase in the overall structural response and thus the likelihood of incurring seismic damage to bridge systems (Shekhar et al., 2023; Yun & Liu, 2025; Pan et al., 2025).

Piers are one of the basic elements in the bridge substructure, controlling lateral stiffness, seismic force transfer, and displacement response under earthquake excitation (Naser et al., 2021; Siddique & Hossain, 2020; Turchetti et al., 2023). Bridge system with variation in pier geometry, specifically pier height can significantly change the dynamic characteristics of bridges and the proportion of seismic forces between spans and supports and will therefore influence the overall structure stability and seismic safety of the system (Wu et al., 2024; Liang et al., 2024). In relatively tall or slender piers, higher vibration modes will become affective and the demand will be increased on displacements and forces as well in other configurations provided that to be compared with short and/or stiff ones (Yanan et al., 2021; Liang et al., 2024; Vaidya & Nichat, 2020). Also, rocking mechanisms and flexibility of the foundation soil may contribute to another level of complexity in explaining the seismic response of these structures if the support elevations are irregular or soft soils build the supporting bases (Sieber & Anastasopoulos, 2025; Wang et al., 2023; Gao & Lin, 2018).

There is typically a combination of dynamic characterization techniques and nonlinear performance-based evaluation methods used to achieve a realistic understanding of bridge seismic performance (Vaidya & Nichat, 2020; Liu et al., 2024). Modal analysis is still one of the most commonly implemented methods to identify the most appropriate response spectrum at which to characterize seismic demand (Vaidya & Nichat, 2020; Neamaha & Naser, 2025a). Also, nonlinear static pushover analysis is also an appropriate capacity-level analysis for evaluating stiffness degradation and inelastic response of structures due to lateral earthquake loading (Kuria & Kegyes-Brassai, 2024; Tarafder & Prasad, 2020).

The pushover procedure has been increasingly incorporated into performance-oriented bridge assessment studies, particularly for various bridge systems such as prestressed girder and box-girder bridges in order to identify key demand–capacity relationships and trends in seismic capacity (Wu et al., 2024; Tarafder & Prasad, 2020; Farhan & Tasleem, 2020). Nonlinear assessment approaches are typically accompanied by demand-to-capacity (D/C) ratios, which are often employed as pragmatic measures to assess whether structural demand is within the bounds of acceptable safety limits due to seismic excitation (Naser et al., 2021). From the results of earlier research, it has been recognized that seismic performance indices such as D/C ratios or yielding shapes coming out from force–displacement curves are sensitive pretty much to the



pier dimensions and the structural configuration (Siddique & Hossain, 2020). Consequently, the combination of pushover analysis and D/C-based evaluation procedures provides a consistent basis for practicing interpreting seismic performance of bridges and establishing fundamental weaknesses of structures (Neamaha & Naser, 2025a; Neamaha & Naser, 2025b).

Although extensive research has been carried out on high piers, innovative seismic control devices, and specialized bridge pier systems, many past studies were restricted to isolated component behavior or specific excitation conditions (e.g., near-fault) (Yun & Liu, 2025; Pan et al., 2025; Shen et al., 2022). While some studies explored connection systems, precast bridge technologies, and confinement improvement techniques at the component level, relatively few investigations have considered the combined effect of varying pier geometries using an integrated, bridge-level numerical framework that simultaneously incorporates modal analysis, pushover assessment, and D/C evaluation techniques (Zhang et al., 2024; Long et al., 2024; Yu et al., 2023; Jaaz et al., 2021). Hence, controlled numerical investigations of geometry-relevant parameters, using consistent modeling assumptions, are still necessary for better understanding of bridge seismic response and minimizing modeling choice ambiguities (Waseem & Saleem, 2019; Rahman et al., 2018).

Ground shaking assessment methods improving seismic risk-based bridge design is also at the forefront of recent research in bridge earthquake engineering. With its estimated ability to provide the probabilistic vulnerability of bridge systems employed under various structural configurations and seismic scenarios, fragility analysis has been widely applied (Zhou et al., 2024; Otárola et al., 2022). Likewise, soil structure interaction (SSI) effects have been identified as parameters that can change bridge dynamic behavior, specifically for curved and irregular bridge systems (Lu et al., 2025). In addition, probabilistic seismic hazard analysis (PSHA) and performance-based earthquake engineering (PBEE) methods and frameworks are increasingly used for a more complete characterization of seismic demand and bridge structural performance (Kia et al., 2022).

This study focuses on the Seismic behavior of a Reinforced Concrete U-girder highway bridge evaluated against two ground motions of different intensities using the software CSiBridge. The approach employed in the study combines modal analysis, response spectrum analysis, nonlinear static pushover analysis, and demand-to-capacity evaluation to investigate the effects of system response, displacement demand, and seismic performance of the bridge system under different seismic intensities.

2. Bridge description and numerical modeling

The bridge is a prestressed concrete skewed U-girder bridge. It is a simply supported bridge, passing Al-Daura-Yusufiyah expressway. The overall bridge has a span of 70m and a width of 14m, There are four spans in it. Both the first and fourth spans measure 10.8m in length. The superstructure is composed of six double precast prestressed concrete fitting to U/geometrical profile, with 20cm of concrete slab, and 8 cm of flexible pavement, while the second and third spans are 24.20m long. And the substructure of the bridge has six piers in the traverse direction across the pile foundation. Figure 1 view of prestressed concrete U-type bridge and Figure 2 cross section of bridge from original drawing.



a) Side view of bridge

b) Transition direction

FIGURE 1. Model of the rectangular pier concrete bridge.

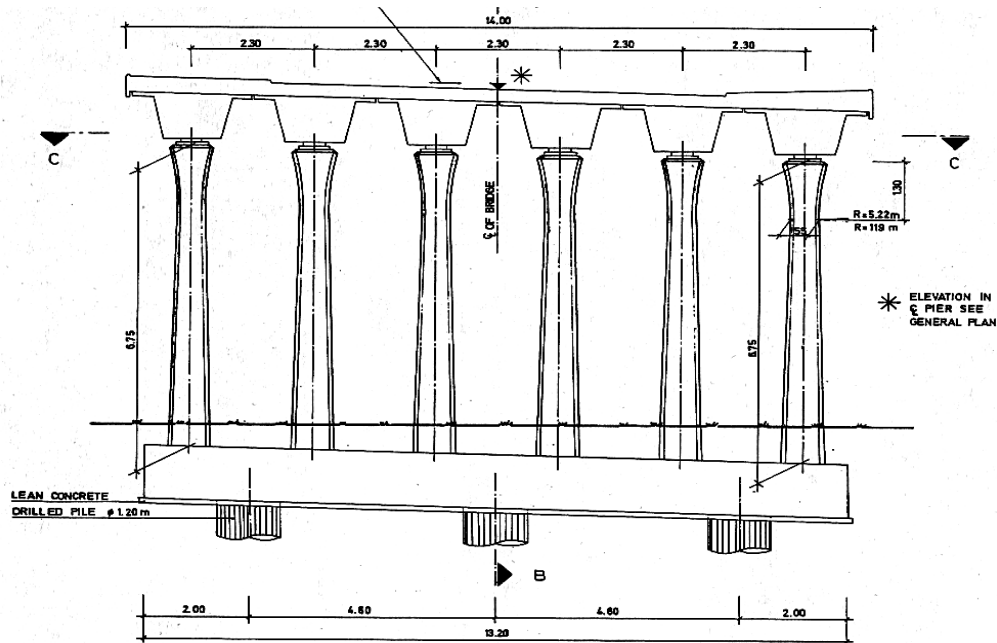


FIGURE 2. Cross section of rectangular pier from general plan.

3. Seismic input

Two ground motion scenarios were defined to cover a broad range of possible shaking of the bridge site. Seismic Case 1 is a moderate seismic hazard case with $S_s = 0.3$ g at short periods, $S_1 = 0.1$ g at a 1 s period and PGA 0.2 g seismic hazard parameters (from the original planning of the bridge).

Seismic Case 2 represents the maximum considered earthquake (MCE) level or the fault-proximate event that produces strongly pulse-like ground motions in the near field, characterized by high intensity with $S_s = 2.29$ g, $S_1 = 0.869$ g, and PGA = 0.954 g (Neamaha & Naser, 2025a) as indicated in the table 1. Design response spectra for both cases were created according to the American Association of State Highway and Transportation Officials (AASHTO) LRFD, with site class effects accounted for via site coefficients F_a and F_v . These spectra were subsequently converted into multi-directional excitation functions applied along the transverse and longitudinal bridge axes in CSiBridge (two dimensional seismic loading) to capture the most critical demand combinations at once.

4. Finite element model of the u-girder bridge

The assembled CSiBridge model faithfully replicates the geometric configuration of the Al-Yusufiyya bridge, including the horizontal alignment, skew angle at supports, and the elevation profile of the bents. Pier heights were set at 7 m in the baseline model discussed herein. Mass was distributed throughout the model based on self-weight and superimposed dead loads including wearing surface, barriers, and utilities. Figure 3 shows the model of the bridge. The concrete was modeled as an isotropic material with a compressive strength of $f'_c = 27.6$ MPa, a modulus of elasticity $E_c = 24.9$ GPa, a Poisson's ratio of 0.2, and a unit weight of 25 kN/m^3 . The reinforcing steel was assigned a yield strength of $f_y = 420$ MPa and a modulus of elasticity $E_s = 200$ GPa. The piers and cap beams were modeled with frame elements, and the deck with the U-girder section properties defined in CSiBridge. Each bent consists of six columns, 6.75 m high and spaced at 2.30 m, which are rigidly connected to a common pile cap. The column bases were assumed fixed at the foundation level, so the flexibility of the piles and soil was not modeled. The girders rest on bearings that restrain the three translations and the rotation about the longitudinal axis, and allow rotation about the other two axes. The abutment supports were modeled as fixed. The same boundary conditions were used in both seismic cases. The seismic mass was calculated from the self-weight of the structure and the superimposed dead loads, namely the 8 cm wearing surface. A damping ratio of 5% was adopted in the response-spectrum functions of both seismic cases. The nonlinear behavior was concentrated in plastic hinges assigned automatically to the bent columns. The pushover analysis was performed using a modal load pattern in the transverse and longitudinal directions, and the performance points were obtained with the ATC-40 capacity-spectrum method.

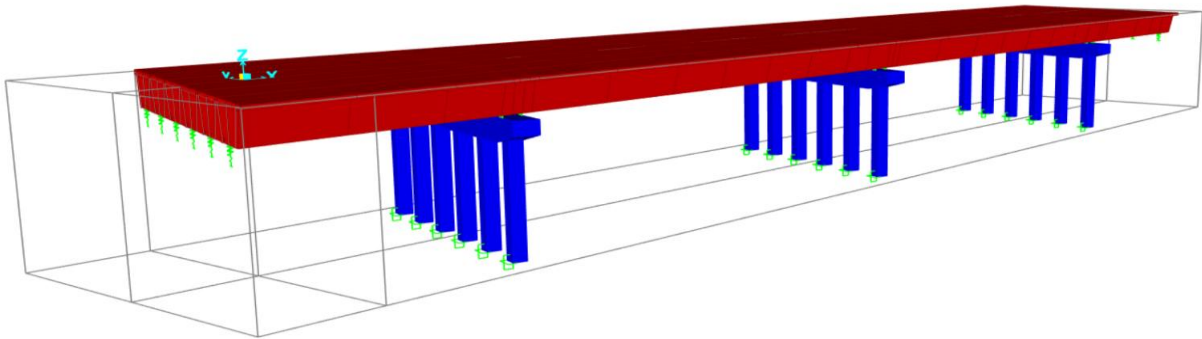


FIGURE 3. Bridge model of the rectangular pier.

5. Seismic resistance analysis

5.1 Demand to capacity ratio (D/C):

The demand-to-capacity (d/c) ratio is one of the most basic parameters used in seismic assessment to determine if the seismic force demand exceeds the structural capacity. With D/C 1.0 providing evidence for potential yielding or failure. Presented in Table 1 are the D/C ratios calculated for each bent in both the transverse (perpendicular to the bridge axis) and longitudinal (parallel to the bridge axis) directions.

TABLE 1. Experimental static load testing results.

Bent no.	D/C ratio (Seismic Case 1: $S_s=0.3$, $S_1=0.1$, $PGA=0.2$)	D/C ratio (Seismic Case 2: $S_s=2.29$, $S_1=0.869$, $PGA=0.954$)
Trans.	0.4443	1.3937



Bent 1	Long.	0.2712	2.3577
Bent 2	Trans.	0.2492	2.1913
Bent 2	Long.	0.2681	2.7985
Bent 3	Trans.	0.2497	1.1221
Bent 3	Long.	0.2331	1.7906

The data in table 1 show a strong seismic intensity dependence of D/C ratios. Under Seismic Case 1 all bents are $D/C < 1.0$, with the largest at bent 1 in the transverse direction ($D/C = 0.4443$), however, this D/C ratio notes that the bridge still has plenty of reserve capacity and will not reach a failure due to seismic loads, even if a loss of strength during earthquake loading occurs. The longitudinal D/C ratios are consistently less than their transverse comparisons at each of the three bents, likely due to the stiffer longitudinal direction mobilized through abutment participation and bent frame geometry.

The transition to Seismic Case 2 produces a dramatic escalation in demand. D/C ratios in the transverse direction reach 1.3937, 2.1913, and 1.1221 at bents 1, 2, and 3, respectively, while longitudinal ratios climb to 2.3577, 2.7985, and 1.7906. The exceedance of $D/C = 1.0$ at all bents and in both principal directions under this high-intensity scenario implies that the structural capacity of the bents is insufficient to accommodate the imposed seismic demand without some degree of inelastic response or potential failure. Bent 2 emerges as the most critically loaded element, exhibiting the largest longitudinal D/C ratio (2.7985). These findings underscore the necessity of detailed nonlinear analysis to assess the actual inelastic deformation demands and the associated performance levels.

5.2 Seismic modal analysis and mode shapes:

Modal analysis helps you to observe the basic dynamic behaviors of the bridge by evaluating the natural periods (or frequencies) of vibration and modal shape associated with government seismic behaviors of the bridge. Table 2 presents the first six periods and frequencies obtained from the eigenvalue analysis carried out with the CSiBridge software in both seismic cases. The relationship between mode number, natural period, and frequency for both cases has been graphically depicted in Figures 4, 5, and 6.

TABLE 2. Natural periods and seismic frequencies for the first six modes under both seismic intensity scenarios.

Mode No.	(Seismic Case 1: $S_s=0.3$, $S_1=0.1$, PGA=0.2)		(Seismic Case 2: $S_s=2.29$, $S_1=0.869$, PGA=0.954)	
	Time (sec)	Seismic Frequency (Hz)	Time (sec)	Seismic Frequency (Hz)
1	0.99067	1.009418015	2.220923	0.450263155
2	0.321879	3.106757066	2.151367	0.464820643
3	0.27158	3.68215874	1.548927	0.64560801
4	0.196318	5.093785385	0.637085	1.56965013
5	0.161188	6.203929357	0.287186	3.482065521
6	0.132545	7.544622866	0.195479	5.11562602

A notable increase in the natural periods for Seismic Case 2 was obtained in contrast to Case 1 in the results of Table 2 while the seismic frequencies showed a significant opposite behavior.

This behavior suggests that at the higher seismic intensity level, the effective structural stiffness is less. The first mode experienced the largest frequency increase, which represents that the global bridge vibration dominates the longitudinal response. On the contrary, the higher modes retained shorter periods at increased frequencies as local structural vibration modes resulted in higher modal stiffness.

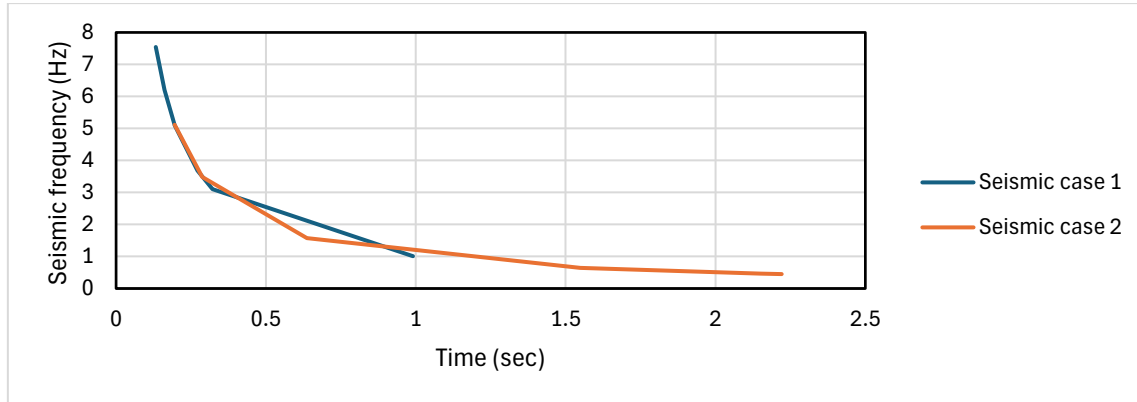


FIGURE 4. The relationship between time and seismic natural frequency for Seismic Cases 1 and 2.

In both seismic cases, Figure 4 shows that as the natural period increases, the seismic frequency decreases progressively. The second seismic case moved to a more flexible structural behavior under increased seismic excitation from shorter periods and higher frequencies of structural folding to larger periods and lower frequency structural modes. It further confirms that the period has an inverse relationship to frequency and that modes with shorter periods have higher frequency vibration and with higher stiffness characteristics.

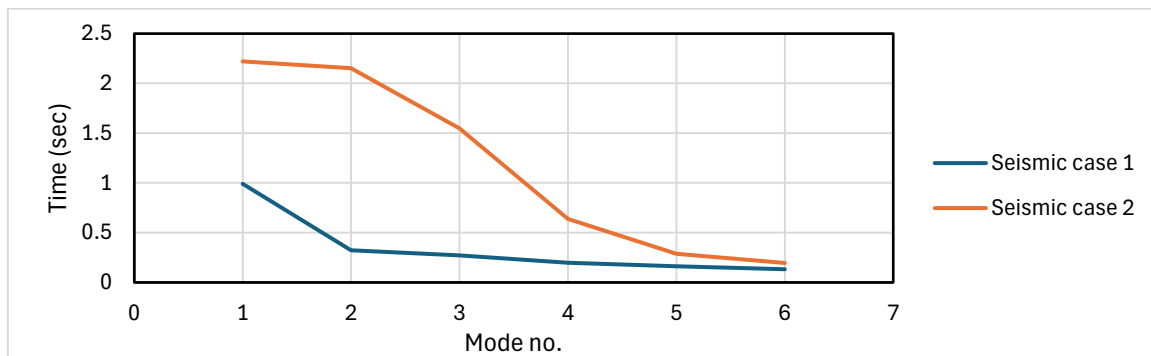


FIGURE 5. The relation between mode numbers and time (sec) for both seismic scenarios. The natural period slowly declines as mode number goes up for each seismic case (Figure 5). The first mode appeared to have the longest periods because they display the global vibration response of the bridge. For Seismic Case 2, the periods were still higher than those of Seismic Case 1 periods, which corroborates that the dynamic response of the bridge for the higher seismic level should be characterized as softer.

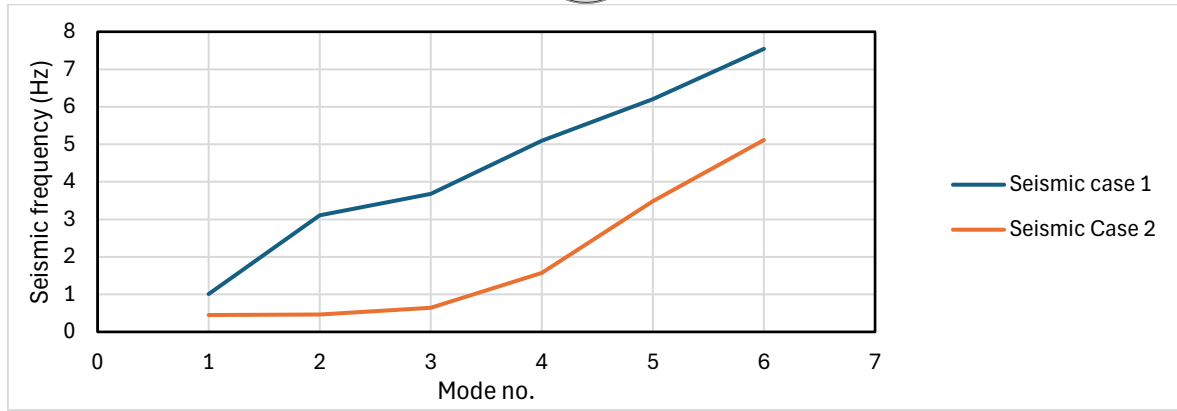


FIGURE 6. The mode numbers and seismic natural frequency for the six extracted modes. The determination of seismic frequencies are as represented in Figure 6, larger mode numbers generated seismic frequencies which are consistent with the change from global structural motion to higher local vibration modes. This seems to follow the fact that Seismic Case 1 used constantly higher frequencies than the Seismic Case 2, indicating the higher effective stiffness relating to this lower level of seismic demand. This is shown to be manifestly more evident at the lower modes where global bridge behaviour is dominant.

In order to study the modal deformation response of the bridge to Seismic Case 1, the maximum and minimum deformation value extraction along the longitudinal (X), transverse (Y), and vertical (Z) directions is shown in table 3 for the first six vibration modes.

TABLE 3. The mode numbers and deformation values in longitudinal and transverse direction for seismic case 1

Mode No.	Min. negative deformation (X) (m)	Max. positive deformation (X) (m)	Min. negative deformation (Y) (m)	Max. positive Deformation (Y) (m)	Min. negative deformation (Z) (m)	Max. positive Deformation (Z) (m)
1	-0.006	0.006	-5.152E-04	0.046	-0.001	0.001
2	-0.022	0.014	-0.045	0.038	-0.009	0.007
3	-0.03	0.002	-0.008	0.007	-0.024	0.003
4	-0.023	0.03	-0.008	0.006	-0.05	0.028
5	-0.038	0.008	-0.006	0.012	-0.01	0.044
6	-0.028	0.028	-0.038	0.018	-0.047	0.055

The vibration is the most complex of the three anisotropic deformation response indicated by the corresponding vibration modes and in three principal directions shown in Table 3. In the higher modes, the transverse and vertical directions generally accounted for greater positive deformation values, while the longitudinal direction presented a relatively stable deformation behavior. The larger magnitude of deformation in some of the higher modes indicates more modal participation and higher complexity of spatial vibration behavior within the bridge structure, with the three eigendirections being represented for Seismic Case 1 based on the extracted modal responses in the third direction, as shown in Figure 7.

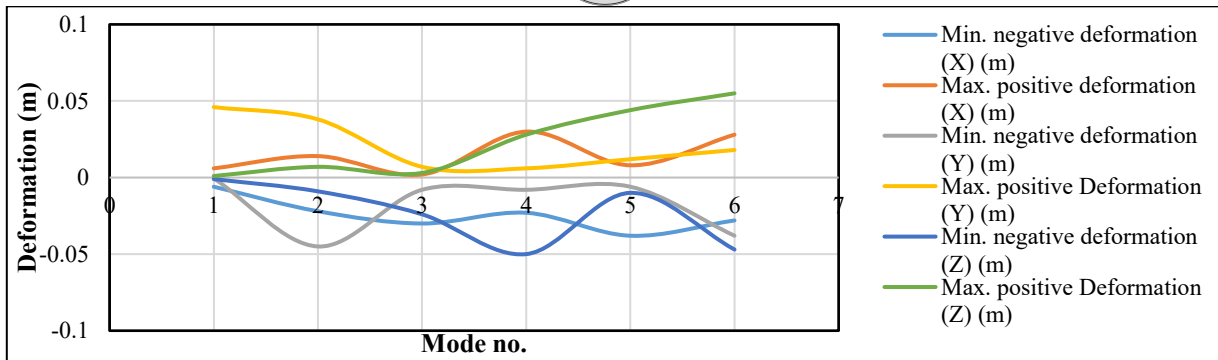


FIGURE 7. Deformation patterns along the three principal directions.

As shown in Fig. 7, due to the coupling dynamic behavior of the bridge system, the mode and direction of deformation response can be seen to irregularly change. In the vertical direction, here was a clear growth of the deformation on the higher modes, while the transverse direction showed strong oscillation between positive and negative answers. From the above analysis, the longitudinal deformation remained more moderate, which means that, under the first seismic scenario, the bridge stiffness in the X-direction remains relatively stable.

In order to study the impact of higher seismic intensity level on the deformation behavior of the bridge, the modal deformation values for Seismic Case 2 in longitudinal, transverse, and vertical directions in table 4 have been extracted.

TABLE 4. The modes numbers and deformation values in longitudinal and transverse direction for seismic case 2.

Mod e No.	Min. negative deformatio n (X) (m)	Max. positive deformatio n (X) (m)	Min. negative deformatio n (Y) (m)	Max. positive Deformatio n (Y) (m)	Min. negative deformatio n (Z) (m)	Max. positive Deformatio n (Z) (m)
1	-0.04	1.648E-04	-0.001	0.005	-0.004	0.004
2	-0.002	0.039	-4.066E-04	0.003	-0.003	0.004
3	-0.002	0.056	-0.002	0.004	-0.005	0.005
4	-0.011	0.013	-0.002	0.043	-0.003	0.002
5	-0.034	0.027	-0.053	0.025	-0.004	0.008
6	-0.019	0.03	-0.009	0.006	-0.041	0.039

Table 4 indicates that Seismic Case 2 produced higher deformation responses in more vibration modes than Seismic Case 1, especially in the transverse and longitudinal directions. Maximum positive deformation in the X-direction increased significantly in the intermediate modes, while the Y-direction showed large negative deformation values in Mode 5. This reflects the stronger modal interaction alternative and less structural flexibility at a higher level of seismic excitation. The spatial deformation reactions of the bridge in the X, Y and Z direction is presented in Figure 8 for Seismic Case 2.

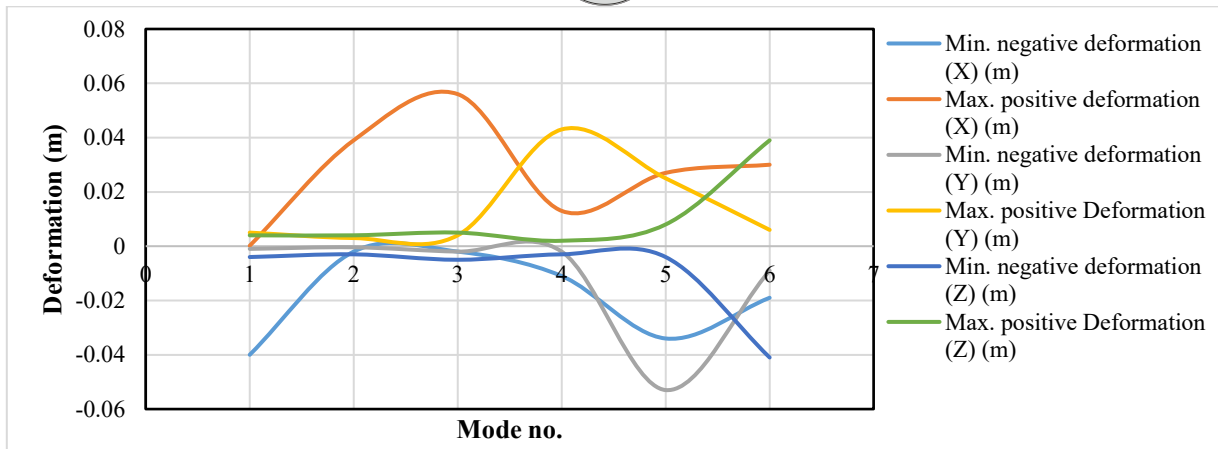


FIGURE 8. Spatial deformation (X-Y-Z directions).

5.3 Non-linear pushover capacity of girder:

This method is based on nonlinear static pushover analysis, and was conducted using the capacity-spectrum method (ATC-40) to describe the elastic-plastic force-deformation relationships of each bent and calculate ultimate balance lateral forces and displacements for individual bending configurations. Pushovers were done in the transverse and longitudinal direction by incrementally applying lateral loads till the target displacement or the formation of a full mechanism was reached. In Table 5, Maximum base shear forces and maximum displacements at the top of each bent are shown in each seismic case while pushover curves are plotted in Figures 9 to 12.

TABLE 5. Max. Force and Displacement value in transverse and longitudinal direction.

Bent no.		(Seismic Case 1: $S_s=0.3$, $S_1=0.1$, PGA=0.2)		(Seismic Case 2: $S_s=2.29$, $S_1=0.869$, PGA=0.954)	
		Max force (KN)	Max displacement (m)	Max force (KN)	Max displacement (m)
Bent 1	Trans.	1276.35	0.056365	1565.092	0.059906
	Long.	445.026	0.08028	558.796	0.129736
Bent 2	Trans.	1457.234	0.061048	1566.354	0.068611
	Long.	241.593	0.042171	665.162	0.123409
Bent 3	Trans.	536.657	0.023226	1194.576	0.059126
	Long.	241.593	0.042171	665.25	0.108191

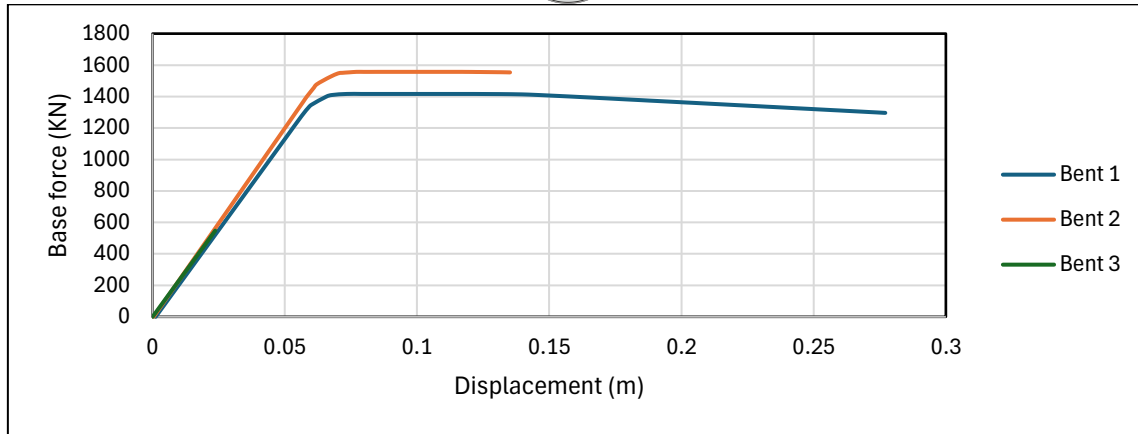


FIGURE 9. Displacement and base force relationship of transverse direction for seismic case 1.

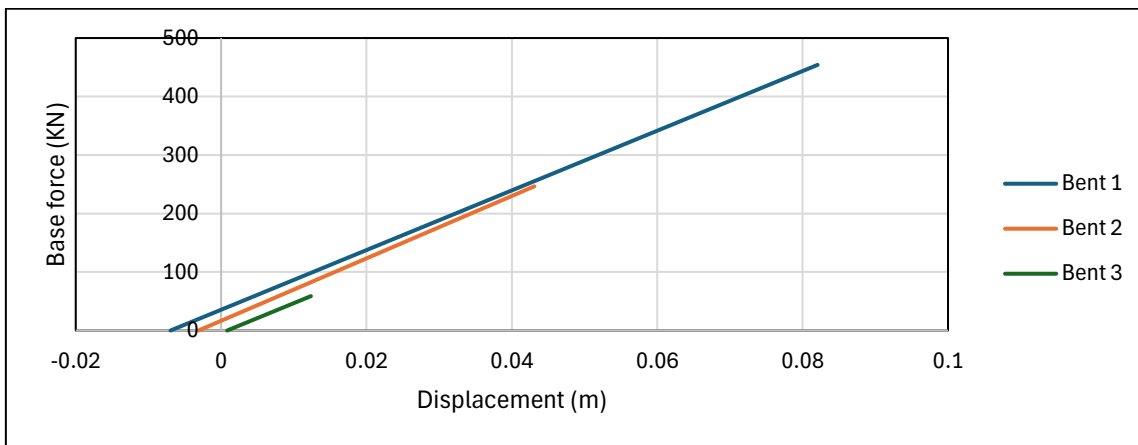


FIGURE 10. Displacement and base force relationship of longitudinal direction for seismic case 1.

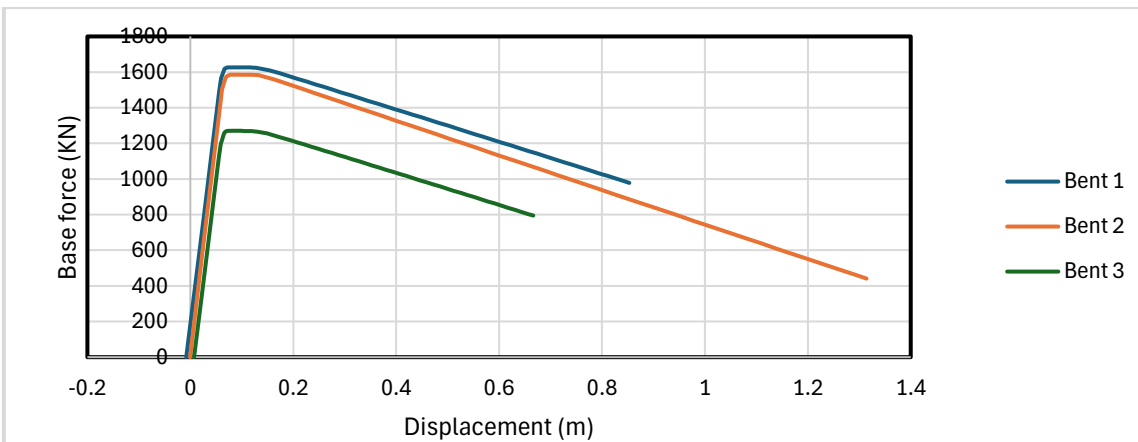


FIGURE 11. Displacement and base force relationship of transition direction for seismic case 2.

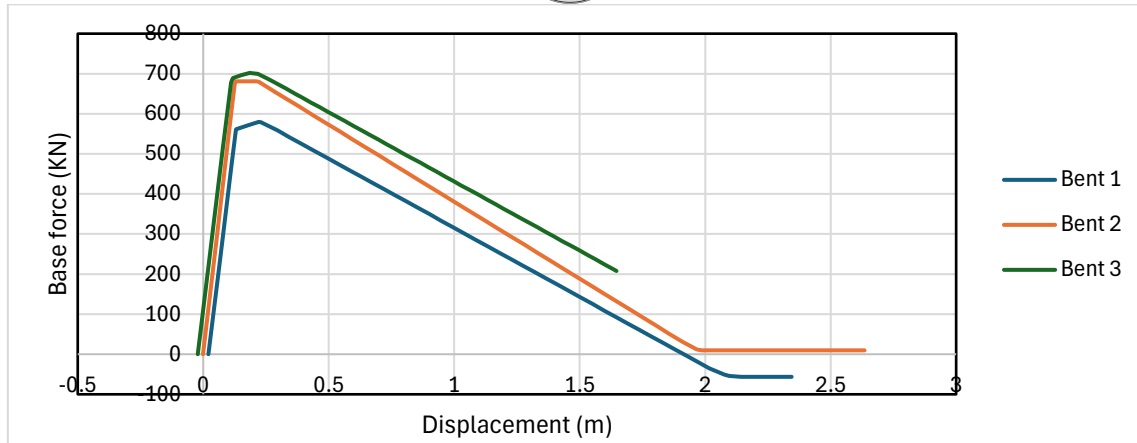


FIGURE 12. Displacement and base force relationship of longitudinal direction for seismic case 2.

Some important behavioral trends from the pushover results in Table 3 are as follows. Under Seismic Case 1 in the transverse direction, Bent 2 has the maximum peak force of 1,457.23 kN and displacement of 0.061 m followed by Bent 1 (1,276.35 kN, 0.056 m) and Bent 3 (536.66 kN, 0.023 m). The minor capacity increase of Bent 3 compared to Bents 1 and 2 could be due to a difference in column section, reinforcement ratio, or tributary mass distribution at that support. Peak forces are substantially smaller in the longitudinal direction (241.59 kN–445.03 kN), while displacements are correspondingly larger, also consistent with the softer longitudinal response seen in the modal analysis.

Seismic demand increases significantly with Seismic Case 2 due to the lateral forces we apply. Transverse forces appearing at Bents 1 and 2 elevate to 1,565–1,566 kN, while Bent 3 experiences a more significant spike to 1,194.58 kN a 122% increase compared to its Case 1 value. Longitudinal forces also show a great increase, with Bent 2 at 665.16 kN (176% of Case 1). It is observed that the maximum displacements associated with Case 2 (a high intensity scenario) are consistently larger in the longitudinal direction with 0.130 m at Bent 1, 0.123 m at Bent 2 and 0.108 m at Bent 3, meaning that the longitudinal deformation mechanism is more ductility-demanding. Overall, these results confirm that although the bridge has adequate inelastic deformation capacity, the high-intensity scenario creates demands on the bridge that far exceed the elastic design envelope.

5.4 Performance points analysis:

The performance point for ATC–40 corresponds to the intersection where the capacity spectrum intersects with the demand spectrum, adjusted for expected inelastic damping; hence it can be used to evaluate expected actual displacement demand at which the building will be subjected during the design earthquake. Tables 6 and 7 summarize the performance point parameters for each bent in both seismic scenarios, and in Figures 13 to 16 the results are portrayed as graphical intersections in the Sa-Sd space.

TABLE 6. Bents performance points under effect of earthquake action for seismic case 1.

Bent no.	Direction	1	2	3
Shear Force (V)	Trans.	555.343	555.33	N.A

	Long.	333.582	N.A	N.A
Displacement (D)	Trans.	0.025	0.023	N.A
	Long.	0.058	N.A	N.A
Spectral acceleration (Sa)	Trans.	1	1	N.A
	Long.	0.642	N.A	N.A
Spectral displacement (Sd)	Trans.	0.022	0.021	N.A
	Long.	0.062	N.A	N.A
Effective period (Teff)	Trans.	0.301	0.294	N.A
	Long.	0.623	N.A	N.A
Effective damping (Beff)	Trans.	0.05	0.05	N.A
	Long.	0.05	N.A	N.A

TABLE 7. Bents performance points under effect of earthquake action for seismic case 2.

Bent no.	Direction	1	2	3
Shear Force (V)	Trans.	555.342	555.329	555.34
	Long.	334.248	341.859	334.241
Displacement (D)	Trans.	0.016	0.022	0.031
	Long.	0.086	0.063	0.044
Spectral acceleration (Sa)	Trans.	1	1	1
	Long.	0.644	0.658	0.643
Spectral displacement (Sd)	Trans.	0.022	0.021	0.022
	Long.	0.062	0.06	0.062
Effective period (Teff)	Trans.	0.3	0.293	0.3
	Long.	0.622	0.608	0.622
Effective damping (Beff)	Trans.	0.05	0.05	0.05
	Long.	0.05	0.05	0.05

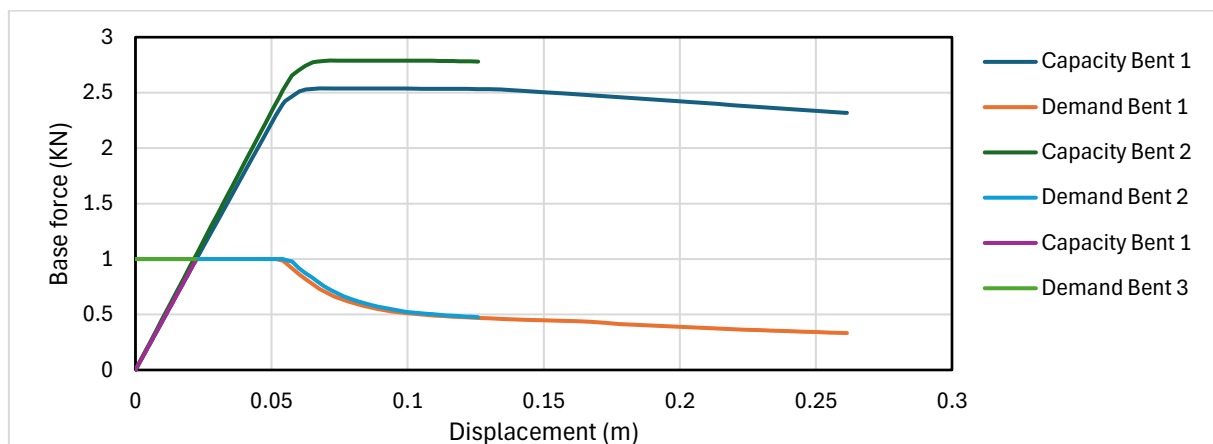


FIGURE 13. Performance point (Sa, Sd) in transverse direction for seismic case 1.

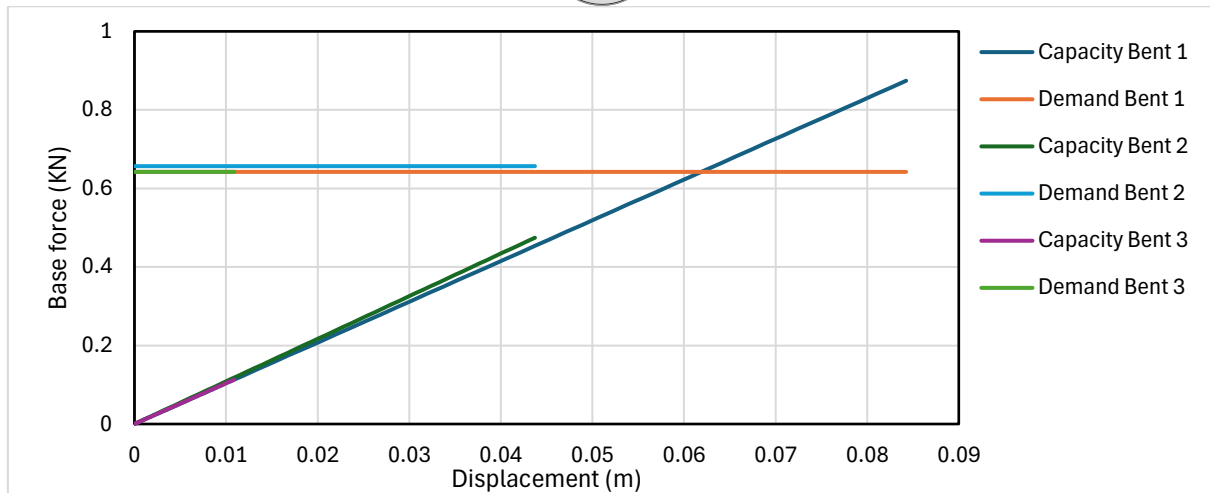


FIGURE 14. Performance point (S_a , S_d) in longitudinal direction for seismic case 1.

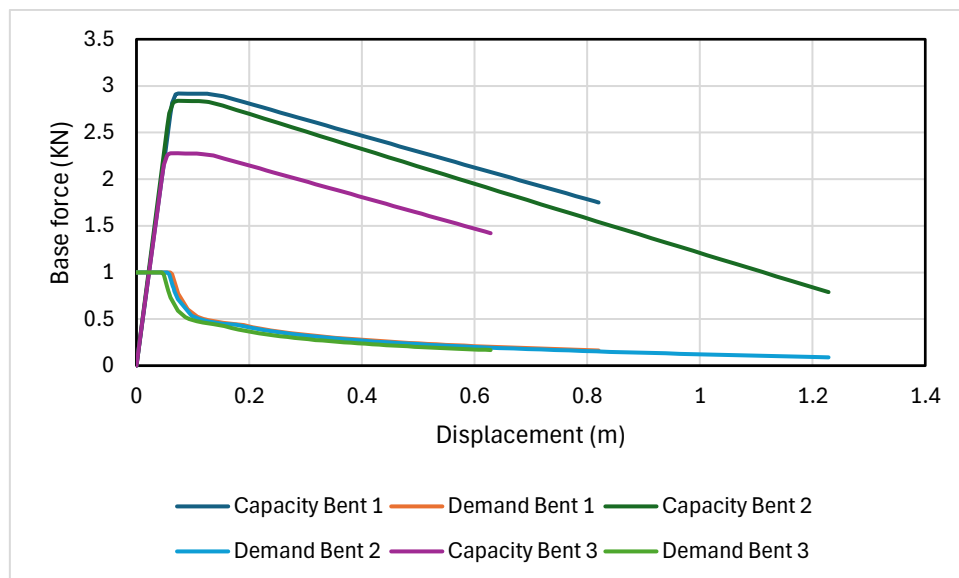


FIGURE 15. Performance point (S_a , S_d) in transverse direction for seismic case 2.

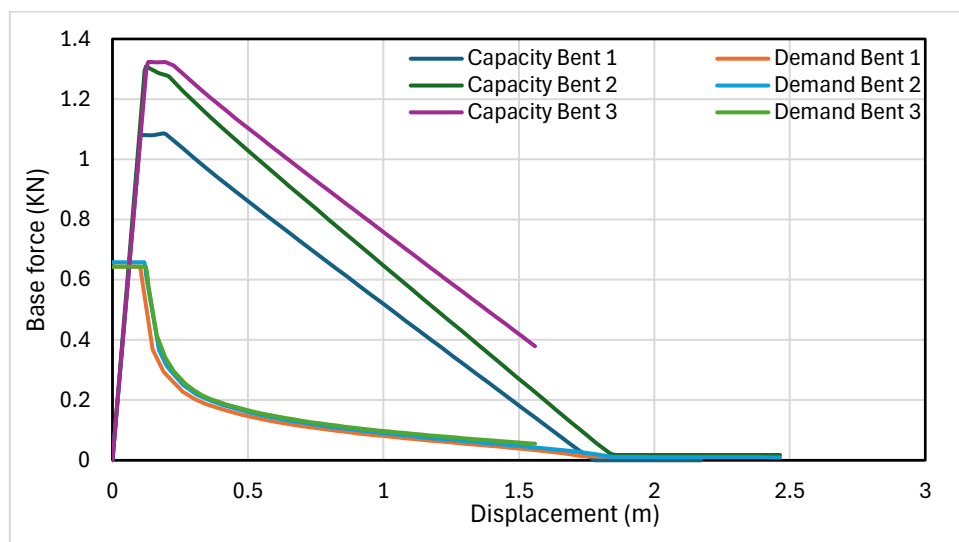


FIGURE 16. Performance point (S_a , S_d) in longitudinal direction for seismic case 2.

Many significant insights can be gained from the performance point data in Tables 4 and 5. In Seismic Case 1, no performance points could be determined for Bent 3 in both directions, and for Bent 2 in the longitudinal direction (noted as N/A), indicating that the seismic demand in those cases was insufficient to drive the structure into inelastic action essentially, the demand spectrum and capacity spectrum did not intersect in the inelastic portion of the spectrum. Performance points at shear forces of approximately 555.3 kN, spectral displacements of 0.022–0.021 m, and effective periods of about 0.294–0.301 seconds are obtained for Bents 1 and 2 in transverse directions. These performance points confirm nearly elastic response with an effective damping ratio (B_{eff}) of 0.05.

For Seismic Case 2, a performance point solution is obtained for all three bents, in all directions, due to the extreme levels of demand imposed by the high intensity ground motion. The shear forces at the performance points in the transverse direction converge to approximately 555.3–555.4 kN for all bents, while the longitudinal values range from 334.24 kN to 341.86 kN. Spectral displacements are steady at $S_d \approx 0.021$ –0.022 m (transverse) and 0.060–0.062 m (longitudinal) and effective periods are uniformly at approx. 0.293–0.300 s (transverse) and 0.608–0.622 s (longitudinal). The fact that the effective damping in case 2 is kept equal to 0.05 at all bents indicates that at the performance point level the structure has not yet entered a grossly inelastic stage, despite the fact that the D/C values discussed in Section 5.1 indicate that force demands are greatly exceeding capacities. This seeming contradiction can be accounted for by recognizing that frequency performance point captures demand on one spectral coordinate while D/C exceedance relates to maximum envelope in force demands across the entire range of the response spectrum. Combined, these two indicators give a full evaluation of the structural vulnerability the building would experience under a strong seismic event.

Conclusion

This study compares the performance of the Al-Yusufiyya U-girder highway bridge under two different levels of ground motion intensity through a multi-method analytical approach in CSiBridge. Here are the main lessons that the investigation yielded.

The performance of the bridge in the first three analytical methods is acceptable under moderate seismic loading (Seismic Case 1, $PGA = 0.2$ g). Demand-to-capacity ratios were found to be far less than 1.0 at all bents and in both main directions, validating that the existing structure has adequate resistance against benign seismic hazard. To these effects, analysis through modes displays primary periods whatsoever between 0.13–0.99 s, matching the stiffness regime of the intermediate-height bent configurations. Pushover analysis shows that the bridge retains significant lateral force capacity, and performance point assessments show that the response is well-within or near the elastic range.

At the latter end of the spectrum, the structural response reverses under the high-intensity seismic scenario (Seismic Case 2, $PGA = 0.954$ g). In the transverse and longitudinal directions, the D/C ratios are higher than 1 at all bents, and the maximum ratios (2.7985 for Bent 2 in the longitudinal direction) indicate that insufficient lateral load resistance capacity exists in the bent. Eigenvalue analysis for Case 2 shows longer fundamental periods, ranging from 0.973 to 2.221 s, which surface much of the near-fault spectral behaviour and with the view into the extreme excitation of the structure, the configuration corresponds to a fundamentally different dynamic system. The longitudinal displacements at Bent 1, 0.130 m, are therefore more than 60% greater



than those reported under Case 1; pushover results confirm the trends of increasing lateral force and displacement demands.

For Case 2, performance point analyses for all bents converge with effective periods of 0.30 s (transverse) and 0.61 s (longitudinal) and effective damping ratios of 5%, suggesting that the D/C exceedance appears to be more of an a priori indication that the structure is, as yet, undeformed and therefore has not fully transitioned from the linear elastic, energy elastic state to the post-yield energy-dissipating regime at the performance point displacement. This indicates that under the high-intensity scenario, force-controlled failure mechanisms are more critical than ductility-controlled mechanisms, show that the bent columns should be adequately designed for shear capacity.

Overall, from an engineering standpoint, we demonstrate that seismic hazard characterization is key in determining bridge safety results. It is necessary to understand that a bridge that is considered acceptable under a design-level moderate earthquake might fail and perhaps be greatly overstressed in a maximum considered event, especially in near-fault zones. The results justify the inclusion of multi-level performance objectives in seismic evaluation of existing bridge infrastructure and highlight the benefits of advanced analytical tools like CSiBridge to capture the influence of the interaction of superstructure, substructure and characteristics of the ground motion.

Future directions for research include time-history analysis using record compatible ground motion for validation of the response spectra results, parametric studies on the effects of pier height and column reinforcement ratio on seismic performance, and fragility analysis quantifying the probability of exceeding defined damage states for a spectrum of ground motion intensities. Future studies should investigate retrofit strategies such as jacketing of the columns, base isolation and addition of damping devices aimed at improving the post-earthquake base shear capacity of the bridge and resilience to extreme seismic loading.

Disclosure

The author reports no conflicts of interest in this work

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