

Proceeding Paper

Seismic Assessment of the Existing Arched Bridge by Application of the Pushover Method [†]

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Abstract

Many concrete arch bridges in Croatia are located in seismically active areas. The last two strong earthquakes that struck central Croatia prompted consideration of the seismic resistance of several dozen arch bridges situated on important traffic routes. Seismic assessment of existing arch bridges presents a significant challenge. Most of these structures were designed and built without seismic guidelines, and their resistance decreases over time due to deterioration. Meanwhile, new regulations increase construction requirements but do not provide a clear procedure for seismic assessment of such structures. Pushover analysis is a nonlinear static procedure used in structural engineering to estimate the seismic performance of bridges by subjecting them to monotonically increasing lateral loads. This method serves as a computationally efficient alternative to complex nonlinear dynamic time-history analysis, while offering insights into structural behaviour beyond the elastic limit. Pushover analysis yields good results for bridges with uniform stiffness in the substructure elements and a dominant horizontal direction of earthquake action. This is generally not the case for arch bridges; therefore, it is necessary to adapt this method. This paper presents the application of pushover analysis in the seismic assessment of an existing arch bridge according to the requirements of current seismic regulations and guidelines. The bridge was built in the 1960s in accordance with the regulations and materials valid at that time, which are no longer in use today.

Keywords: concrete arch bridge; seismic assessment; pushover analysis; plastic hinge rotation



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1. Introduction

Croatia is a seismically active country, making the adequate consideration of earthquake effects a crucial aspect of managing existing transportation infrastructure. Bridges act as key elements in this network, and their usability immediately following an earthquake is of extreme importance for rescue operations and evacuation purposes. However, many existing Croatian bridges were built before the enforcement of modern Eurocodes. These structures were often designed according to older standards that lacked provisions for structural details facilitating ductile behavior, and a considerable number were designed with no regard to seismic actions at all. Over time, changes in the requirements of new

standards, coupled with structural deficiencies and material degradation during years of service, have resulted in varying levels of reliability for these existing structures. Consequently, determining the current condition and capacity of existing bridges in terms of their seismic safety serves as the first and most important step in the strategy for seismic infrastructure protection [1].

Reinforced concrete arch bridges represent a highly specific type of structure characterized by significant robustness. Because of this inherent robustness, the arches themselves generally respond linearly to seismic events, meaning their performance under seismic design conditions is often found to be acceptable. However, the dynamic specificity of arch bridges lies in the flexibility of the arch acting as support for the spandrel columns, combined with a large amount of the bridge's mass being concentrated in the middle. During the inelastic response of an arch bridge to an initial seismic shock, the shortest spandrel columns—particularly those located near the arch crown—are subjected to the greatest deformation demands [2]. This excessive demand leads to heavy cracking and a subsequent reduction in stiffness, which eventually shifts the deformation demands from the crown outwards to the coastal columns, causing their deterioration as well [2].

Despite these well-documented vulnerabilities, current European seismic codes, such as EN 1998-3 for buildings [3] and EN 1998-2 for new bridges [4], do not offer a dedicated procedure specifically for the seismic assessment of existing arch bridges [5]. Furthermore, relying on modern regulations like the Eurocode results in significantly higher seismic internal forces compared to older codes, highlighting the necessity for advanced evaluation to ensure an adequate level of seismic safety [5]. To correctly assess these older structures, it is first necessary to establish an appropriate knowledge level regarding the existing bridge based on its importance, which involves systematically collecting data on geometry, structural details, and material properties in conjunction with appropriate confidence factors [3,5].

To properly address the complexities of arch bridge behavior, a multi-level seismic assessment procedure is highly recommended [5,6]. The first level of assessment utilizes a linear multimodal spectral analysis, incorporating the effective stiffness of the columns, which is typically sufficient for evaluating the arch. However, because spandrel columns exhibit inelastic responses, they require a transition to a second level of assessment based on non-linear static pushover analysis [6]. The pushover method is significantly simplified compared to non-linear dynamic time-history analysis because it is performed on a simplified model and avoids the difficulties and uncertainties associated with selecting appropriate accelerograms. By applying a monotonically increasing lateral load, this method yields capacity curves (force–displacement curves) that can be directly compared with the current seismic demands mandated by the Eurocode [6].

Following this established framework, this paper presents the seismic assessment of an existing arch bridge by applying, consequently, linear multimodal spectral analysis and the non-linear static pushover method. The objective is to visualize the global seismic performance of the bridge by comparing its structural capacity curves with current seismic demands, thereby identifying critical structural deficiencies and, further on, proposing adequate retrofitting measures.

2. Seismic Assessment of Existing Bridges

The resistance of structures changes over time due to weathering, exposure to extreme events, and the neglect of regular maintenance. Simultaneously, structural demands may increase due to changes in loading, an extended required lifespan, or the modernization of building codes and standards. Existing structures frequently fail to meet modern reliability requirements, particularly regarding seismic resistance, because earthquake loads were

either ignored during original design or earlier codes mandated lower seismic loads than those required today [1].

Historically, earthquakes have caused significant damage and collapses, prompting detailed studies that led to a better understanding of seismic actions on structures. A new design philosophy was introduced based on the probability of failure, calculating earthquake actions for a return period of 475 years, or a 10% probability of exceedance in 50 years [3]. While conservative load and resistance models are used in new designs to cover uncertainties without significantly increasing total costs, applying these same conservative measures to evaluate existing structures can lead to unjustified and excessively high expenses. Therefore, it is essential to collect specific data, perform appropriate calculations, and accurately determine the actual resistance and reliability of an existing structure to handle current and future loads [1,5].

Evaluating existing structures for seismic loads can involve lateral force methods, multi-modal response spectra, non-linear static (pushover) analysis, non-linear time-history analysis, or the q-factor approach [3,5,6].

While non-linear dynamic time-history analysis is the most accurate, it is highly demanding. It requires specific ground motion records tailored to the site, complex software, and specialized expertise. Consequently, static calculations are frequently used. Non-linear static methods (like pushover analysis) are particularly valuable as they reveal critical zones prone to large inelastic deformations, stiffness irregularities, forces in potentially brittle elements, and the exact yielding sequence of the structure [6].

2.1. Seismic Assessment of RC Arch Bridges

Many existing reinforced concrete (RC) arch bridges were designed without considering seismic loads [2]. Because current European norms lack a specific procedure for seismically assessing existing bridges, engineers must combine the norms for assessing buildings with the norms for designing new bridges [3–5]. This customized procedure relies heavily on a combination of linear spectral analysis and non-linear pushover analysis [5,6].

The first step is establishing the required Knowledge Level (KL) based on the bridge's importance to correctly model the structure. KL2 is required for average bridges and requires testing 20% of critical cross-sections (or 40% if original documentation is missing). KL3 is mandated for critical infrastructure, requiring testing on 20% of cross-sections with existing documentation, or up to 60% if documentation is absent. These knowledge levels inform the Confidence Factors (CF) used to scale material properties in calculations [3,5].

If the remaining lifespan of the bridge t_L is less than 50 years, seismic loads can be calculated using a reduced peak ground acceleration $a_{g,red}$ for a shorter return period $T_{R,red}$. This is defined by the following relationships, where $a_{g,R}$ is the reference peak ground acceleration and k is a seismicity exponent (typically 0.30 to 0.40) [5]:

$$T_{R,red} = \frac{1}{1 - (1 - p)^{1/t_L}} \quad (1)$$

$$a_{g,red} = a_{g,R} \cdot \left(\frac{T_{R,red}}{T_{NCR}} \right)^k \quad (2)$$

For existing bridges lacking specific seismic detailing, an overall behavior factor of $q = 1.0$ is proposed [4,5].

Because the main arches are highly robust, they generally satisfy structural requirements during the first assessment level. However, spandrel columns—due to their inelastic response—typically require a second assessment level [2,6].

2.1.1. First Level: Linear Multi-Modal Spectral Analysis

This level uses a structural model built from inspections and applies a linear multi-modal spectral analysis in both horizontal and vertical directions. For arch bridges, securing 80% effective mass participation is deemed sufficient, as arch springings are incredibly heavy and difficult to activate. The following checks are strictly performed Table 1 [3,4].

Table 1. First level checks.

Check	Equation	Description
1.1	$d_{allow} \geq d_E$	Ensures seismic displacements (d_E) do not exceed allowed limits (d_{allow}) to prevent deck pounding against abutments.
1.2	$f(N_{Rd}, M_{Rd}) \geq f(N_E, M_E)$	Verifies that the design resistance to axial forces and bending moments exceeds the seismic demand.
1.3	$V_{Bd,1} = \frac{V_{Rd}}{\gamma_{Bd,1}} \geq V_E$	Confirms shear resistance (incorporating a safety factor against brittle failure, $\gamma_{Bd,1} = 1.25$) exceeds the seismic shear demand.

2.1.2. Second Level: Nonlinear Static Analysis

If the first level fails (usually for the spandrel columns), the assessment shifts to non-linear static analysis [5,6]. The N2 single-mode method is applied if a dominant mode captures over 75% of the total mass; otherwise, Modal Pushover Analysis (MPA) is used for all modes contributing over 1% up to an 80% mass threshold. Load distributions can be uniform (stressing coastal columns) or proportional to mode shapes (stressing crown columns). The following structural checks are performed at the target displacement Table 2.

Table 2. Second-level checks.

Check	Equation	Description
2.1	$\theta_{ls} \geq \theta_{p,E}$	Ensures the plastic hinge rotation demand ($\theta_{p,E}$) remains below the limit state chord rotation capacity (θ_{ls}) [3].
2.2.a	$\frac{f_{cm}}{CF \cdot \gamma_{c,acc}} \geq \sigma_{c,E}$	Confirm stresses in unconfined concrete (outside plastic hinges) do not exceed factored mean strengths.
2.2.b	$\frac{f_{cm,c}}{CF \cdot \gamma_{c,acc}} \geq \sigma_{c,E}^{pl.hinge}$	Confirms stresses in confined concrete (within plastic hinges) remain within allowable limits.
2.3	$\frac{f_{ym}}{CF \cdot \gamma_{s,acc}} \geq \sigma_{y,E}$	Verifies reinforcement steel stresses do not exceed factored yield strengths.
2.4	$V_{Bd,1} = \frac{V_{Rd}}{\gamma_{Bd,1}} \geq V_E$	Re-verifies shear capacity in plastic hinge regions against non-ductile failure modes.
2.5	$\frac{A_{t,built}}{S_{T,built}} \geq \min\left(\frac{A_t}{S_T}\right) = \frac{\Sigma A_s}{1.6} \times \frac{f_{ys}}{f_{yt}}$	Checks the spacing of transverse ties to prevent the outward buckling of longitudinal compression reinforcement [3,4].

3. Seismic Assessment of the Arch Bridge over the Slunjčica River

At the picturesque location of Rastoke in Slunj, where the Croatian state road D1 crosses the Slunjčica River, Prof. Kruno Tonković designed an aesthetically pleasing arch bridge [7]. Despite the highly demanding terrain, the bridge represents a fine example of harmony between the landscape and the structure.

The bridge was originally designed according to the PTP 2, 3, 5, and 11 regulations from the 1960s [7]. The original design accounted for the permanent loads of the bridge, traffic loads, shrinkage and temperature effects, and wind loads. The static analysis was performed using the classical theory of elasticity. However, seismic loads were not considered, as the national technical standards of the 1960s classified the bridge's location as a low-seismicity zone [7].

According to the current Croatian seismological map, the peak ground acceleration at the bridge site is 0.12 g [8]. Therefore, a seismic assessment of the bridge was conducted in accordance with current European regulations to inform the decision of whether the bridge requires rehabilitation, replacement, or if the risk can be accepted, leaving the bridge in its existing state [9].

3.1. Basic Information on the Computational Model

The bridge was modelled using the software Sofistik, utilizing line elements for all bridge components: the arches, the columns, and the deck [10].

The arches are fixed at both foundation ends, and the cross-tie at the ends of the arch was not modelled. The columns are rigidly connected to both the arch and the deck, with the exception of the central columns (P6, P7, and P8), which have their rotation around the y and z axes released. At the abutment locations, bearings were modelled to allow longitudinal displacements; additionally, one of the bearings at abutment A13 allows for transverse displacement.

3.2. Materials

The materials from the original main design do not comply with current regulations [7]. Therefore, updated material properties were used for the condition assessment of the bridge.

Material Specifications:

- Deck Concrete: Original design specified M-300; updated to C20/25.
- Column Concrete: Original design specified M-300; updated to C20/25.
- Arch Concrete: Original design specified M-450; updated to C25/30.
- Reinforcement: Original design specified St. 37; updated to B220B.

3.3. Longitudinal Elevation

The span of the arch axis is $l_{ax} = 72.61$ m Figure 1. The rise in the arch, defined by the roadway position, is $f_{in} = 10$ m for the intrados, $f_{ax} = 9.91$ m for the arch axis, and $f_{ex} = 9.82$ m for the extrados. The arch shape is a circular segment with an intrados radius of $R_{in} = 69.8$ m, an axis radius of $R_{ax} = 71.47$ m, and an extrados radius of $R_{ex} = 73.18$ m. The road alignment on the bridge is straight in the floor plan, while the vertical profile has a constant upward slope of 4% from abutment A1 to A13. The deck is supported by vertical columns (P2 to P12) spaced at a longitudinal distance of 9.8 m. The heights of the columns vary longitudinally from left to right: 5.65; 8.76; 7.54; 3.62; 1.28; 0.34; 0.74; 2.46; 5.58; 10.28; and 10.99 m Figure 1.

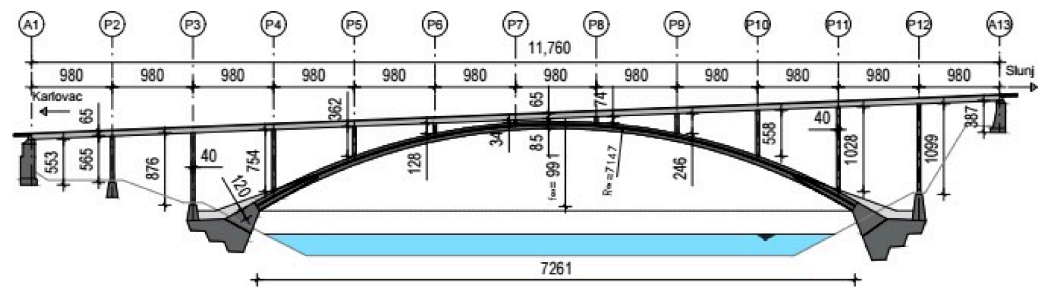


Figure 1. Longitudinal layout.

3.4. Cross-Sections

The bridge features the following specific cross-sections which were modeled for the condition assessment:

- The main arches possess a solid, variable-depth cross-section: 120 cm thick at the arch springing (abutment) and 85 cm thick at the arch crown Figure 2.
- The bridge deck is a standard reinforced concrete slab with a constant thickness of 60.9 cm in the central section (which has a width of 6.05 m) and thins out toward the transverse edges Figure 2.
- The column bents consist of two columns, each 200 cm wide with a dovetail-shaped cross-section. The clear transverse distance between the twin columns varies along the arch, measuring 180 cm at the crown and 120 cm at the arch abutments Figure 2.

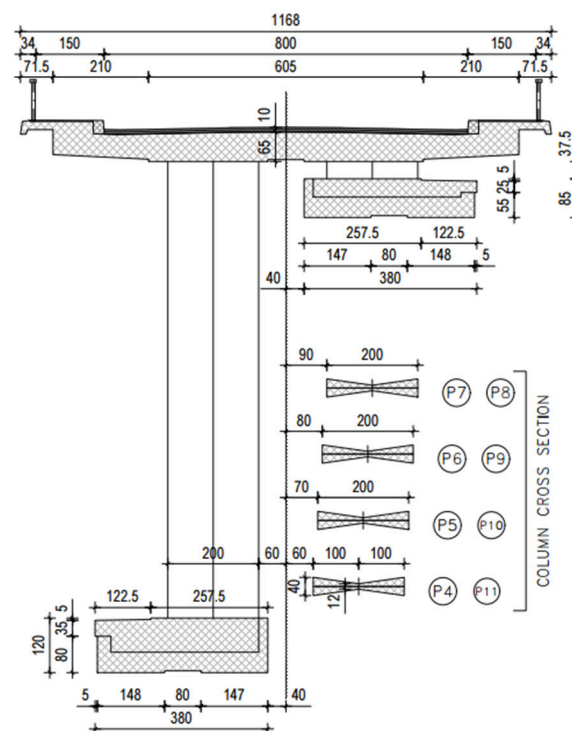


Figure 2. Transverse layout.

3.5. Effective Stiffness of Spandrel Columns

When modeling the bridge, the effective stiffness of the spandrel columns must be accounted for in all columns [2,6]. This reduction in the stiffness of the concrete cross-section is necessary due to the cracking that occurs in the areas of potential plastic hinges at the top and bottom of the columns. During the initial seismic shock and the subsequent inelastic response of the arch bridge, the largest displacement demands are placed on the columns near the arch crown, resulting in their excessive cracking. Following this cracking

and the associated stiffness reduction, the displacement demands shift toward the coastal columns, causing their degradation as well [2].

The effective stiffness of the spandrel columns is determined using the “Plastic Hinge” command in Sofistik, which provides a set of idealized Moment-Rotation laws in the plastic region for various levels of axial force [10]. These laws are established based on cross-sectional analysis and a subsequent idealization method. The axial force values used to read the effective stiffness factors are calculated as the sum of the self-weight, additional permanent loads, and the vertical reaction from the seismic load for the dominant mode of vibration. This effective stiffness is applied exclusively at the boundaries of the columns where plastic hinges form. Because the axial forces from the seismic load differ depending on whether the earthquake predominantly acts in the x or y direction, the corresponding effective stiffnesses vary accordingly Figure 3.

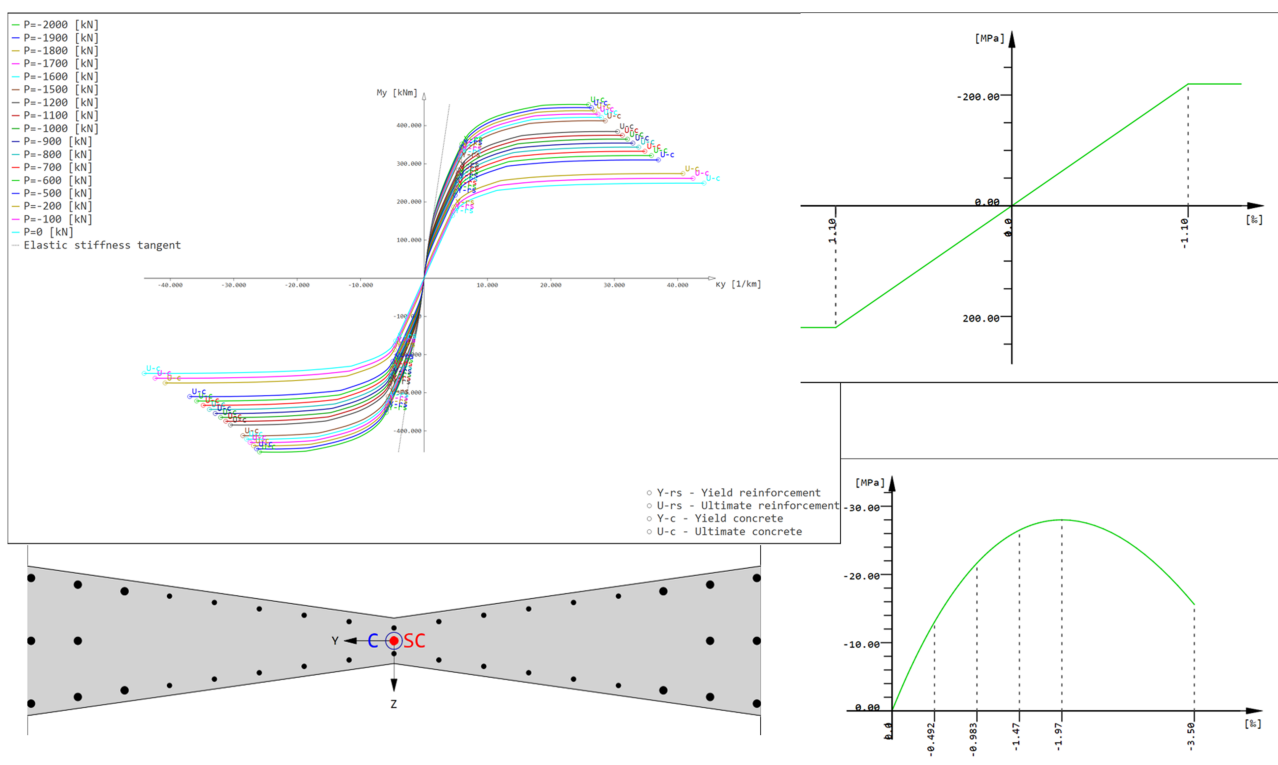


Figure 3. Parameters for defining the effective stiffness of pier S9.

The plastic hinge length is determined according to Equation (3) from EN 1998-2 [4]:

$$L_p = 0.10 \cdot L + 0.015 \cdot f_{yk} \cdot d_{bL} \quad (3)$$

where

- L : The distance from the plastic hinge section to the section of zero moment under seismic action.
- f_{yk} : The characteristic yield strength of the longitudinal reinforcement.
- d_{bL} : The diameter of the longitudinal reinforcement bars.

3.6. Loads on the Computational Model

Seismic Loads for Linear Analysis: Instead of the standard peak ground acceleration of 0.12 g [8], the assessment utilizes a reduced value of $a_{g,red} = 0.1$ g, which reflects an assumed remaining service life of 30 years [5]. A behavior factor of $q = 1.0$ is applied because the existing structure lacks modern seismic detailing [4,5]. Because the massive

weight of the arch springings makes it difficult to achieve the standard target for effective mass participation, the model was adjusted by removing the self-weight from the arch segments closest to the abutments. The load is applied using EC8 Type 1 response spectra combined across three directions (with a factor of 1.0 for the leading direction and 0.3 for the accompanying directions) [4].

Seismic Loads for Non-Linear Pushover Analysis: Two distinct seismic load cases were defined for the longitudinal (x) and transverse (y) directions [6]. This separation is required because the effective stiffness of the columns varies depending on which direction the seismic forces are acting upon.

4. Summary of Seismic Analysis Results

The seismic assessment of the reinforced concrete arch bridge over the Slunjšica River was conducted through a two-level procedure: a Level 1 assessment using linear multi-modal spectral analysis, and a Level 2 assessment utilizing non-linear static pushover analysis [11].

4.1. Level 1 Assessment: Linear Multi-Modal Spectral Analysis

The initial assessment level verifies global displacements and structural member resistances against seismic demands:

- **Check 1.1 (Displacement Demand):** The maximum longitudinal displacement at the abutments was calculated as $d_{E,x} = 20.147$ mm. The allowable displacement limit was estimated at 0.7 mm per meter of the bridge length (totaling an allowable displacement of $d_{allow} = 41.16$ mm). Because the seismic displacement is lower than the allowable limit, this check is satisfied. Transverse displacements are constrained by abutment bearings, thus also fulfilling the criteria.
- **Check 1.2 (Bending Moment and Axial Force Resistance):** This check compares the provided structural capacity against the combined bending moment and axial force seismic demands.
- **Arch:** The arch bases and the arch crown do not have sufficient resistance, meaning this check is not satisfied. It is assumed this failure is partly due to the absence of the arch cross-tie in the numerical model, but a Level 2 assessment is required.
- **Spandrel Columns:** Only the centrally hinged columns (P6, P7, P8) satisfied this check. All other columns failed to meet the required capacity.
- **Deck (Superstructure):** Spans I and II passed the check, but Span III and the sections over bearings IV, V, and VI failed.
- **Check 1.3 (Shear Force Resistance):** The shear capacity was verified against the seismic shear demands. The arches, all columns, and all sections of the bridge deck possess sufficient shear resistance. Therefore, this check is satisfied across the entire structure.

Conclusion of Level 1: Because the arch, the majority of the spandrel columns, and portions of the deck did not satisfy the bending and axial resistance requirements (Check 1.2), it is mandatory to proceed to the Level 2 assessment based on non-linear pushover analysis.

4.2. Level 2 Assessment: Non-Linear Static Pushover Analysis

The non-linear static pushover method (N2 method) was applied in both the longitudinal (x) and transverse (y) directions. The analysis evaluates the bridge's capacity to reach specific target displacements: $d_{Tx} = 28.319$ mm for the longitudinal direction and $d_{Ty} = 16.671$ mm for the transverse direction. Target displacements were adopted from modal response spectrum analysis, provided they are within the limits achievable by the structure, such as the size of expansion joints at abutments, etc.

4.2.1. Global Bridge Capacity

- Longitudinal (X) Direction: The target displacement was not reached under either the modal-proportional or the uniform load distribution patterns. This indicates that the bridge lacks sufficient global capacity in the longitudinal direction under seismic loading Figure 4a.

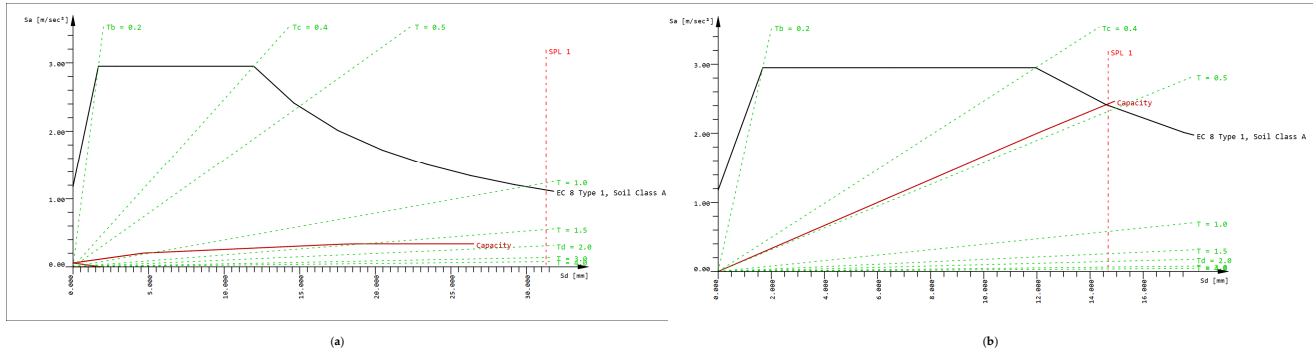


Figure 4. (a) A-D diagram for longitudinal (X) direction; (b) A-D diagram for transverse (Y) direction.

- Transverse (Y) Direction: The target displacement was successfully reached for both load distribution patterns. The bridge exhibits satisfactory global seismic capacity in the transverse direction Figure 4b.

4.2.2. Component Deformation and Stress Checks

Detailed structural checks were performed at the target displacements (or at the failure point for the longitudinal direction).

Longitudinal Direction Checks:

- Check 2.1 (Plastic Hinge Rotation): The rotation demands ($\theta_{p,E}$) were compared against rotation capacities (θ_{ls}). The columns P3, P4, P11, and P12 successfully satisfied this requirement. However, the shorter columns—P2, P5, P9, and P10—exceeded their rotation capacities, meaning they failed this check.
- Checks 2.2a, 2.2b, and 2.3 (Material Stresses): Stresses in unconfined concrete, confined concrete (in plastic hinge regions), and reinforcement steel were checked against their factored strength limits. All evaluated elements successfully satisfied these stress checks.
- Check 2.4 (Shear Resistance in Hinge Regions): All evaluated elements possessed sufficient shear capacity and satisfied the check.
- Check 2.5 (Buckling/Tie Spacing): The spacing of transverse ties was checked to ensure it prevents the buckling of longitudinal compression reinforcement. All evaluated columns failed this geometric requirement, meaning this check is not satisfied.

Transverse Direction Checks:

- Check 2.1 (Plastic Hinge Rotation): The rotation demands for the transverse direction were significantly lower than capacities. All evaluated columns successfully satisfied this check.
- Checks 2.2a, 2.2b, and 2.3 (Material Stresses): All evaluated elements (arches, columns, and deck) remained within allowable stress limits, thereby satisfying these checks.
- Check 2.4 (Shear Resistance): Shear capacities were sufficient across all evaluated elements (arches, columns, and deck), meaning the check is satisfied.
- Check 2.5 (Buckling/Tie Spacing): Consistent with the longitudinal direction, the transverse tie spacing is inadequate across all columns, meaning this check is not satisfied.

5. Conclusions

This study presented a comprehensive seismic assessment of an existing reinforced concrete (RC) arch bridge designed and constructed in the 1960s, prior to the implementation of modern seismic design codes. Because the bridge is situated in a region currently classified under moderate seismicity, its structural safety was re-evaluated using a two-level assessment framework comprising a linear multimodal response spectrum analysis (Level 1) and a nonlinear static pushover analysis via the N2 method (Level 2).

The Level 1 linear assessment confirmed that shear capacities across all structural members and longitudinal displacements at the abutments satisfied code requirements. However, combined axial force and bending moment (N-M) interaction demand-to-capacity ratios were significantly exceeded for the arch, the deck superstructure, and the fixed spandrel columns (P2–P5 and P9–P12), with only the crown-hinged columns (P6–P8) satisfying linear strength checks.

To obtain a more realistic and less conservative evaluation of the structural response, a Level 2 nonlinear static pushover analysis was conducted in both orthogonal directions. In the transverse (Y) direction, the target displacement was successfully attained, confirming adequate global transverse ductility and capacity. Conversely, in the longitudinal (X) direction, the target displacement could not be reached, demonstrating insufficient overall longitudinal seismic capacity. Component-level deformation demand evaluations revealed that while the arch and deck superstructure possessed sufficient inelastic deformation capacity in both directions, critical vulnerabilities were identified in the spandrel columns. Specifically, spandrel columns P2, P5, P9, and P10 exhibited plastic rotational demands exceeding their chord rotation capacities during longitudinal response. Furthermore, excessive transverse tie spacing relative to contemporary ductile detailing standards increases the risk of longitudinal reinforcement buckling within potential plastic hinge zones.

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