E-43
Kongres 2026
Congress 2026

doi: 10.67563/ASESCONF2026019N

Novak Novaković[1], Milan Pandrc[2]

DESIGN OF NEW REINFORCED CONCRETE ARCH BRIDGE OVER THE ĐETINJA RIVER

Summary: This paper presents the design of a reinforced concrete arch bridge over the Đetinja River canyon, part of the Užice bypass route in Serbia. The 292 m long structure features a main arch span of 162 m with a rise of 23,6 m (rise-to-span ratio 1/6,86), placing the deck approximately 90 m above river level. The rationale for selecting a relatively shallow arch, thrust line optimization by iterative polygon-of-forces construction, and the structural behaviour under permanent, variable, thermal, and seismic actions are discussed. Variable arch cross-section dimensions and their structural and aesthetic roles are explained. Particular attention is given to concrete Freyssinet hinges used at the short pendel piers. The cantilever construction method with temporary towers and cable stays is outlined.

Keywords: arch bridge, shallow arch, thrust line, concrete hinges, cantilever construction

PROJEKTOVANJE NOVOG ARMIRANO-BETONSKOG LUČNOG MOSTA PREKO REKE ĐETINJE

Rezime: U radu je prikazan projekat armirano-betonskog lučnog mosta preko kanjona reke Đetinje, koji je deo obilaznice oko Užica u Srbiji. Konstrukcija dužine 292 m ima luk raspona 162 m i strelom 23,60 m (odnos strela/raspon = 1/6,86), a nivo kolovoza je oko 90 m iznad vodene površine. Obrazlažu se razlozi odabira relativno plitkog luka, optimizacija potpornje linije iterativnom metodom poligona sila i analiza ponašanja konstrukcije pod stalnim, promenljivim, temperaturnim i seizmičkim dejstvima. Prikazana je uloga promenljivog poprečnog preseka luka i istaknute su tehničke i estetske prednosti. Posebna pažnja posvećena je betonskim zglobovima tipa Frejsine, koji se koriste kod kratkih pendel stubova. Opisan je metod konzolne gradnje luka uz pomoć privremenih tornjeva i zategnutih sajli.

Ključne reči: lučni most, plitki luk, potpornja linija, betonski zglobovi, konzolna gradnja

[1] DEL ING d.o.o., Belgrade, Serbia, novak.novakovic@deling.rs
[2] DEL ING d.o.o., Belgrade, Serbia

1. INTRODUCTION

The construction of the Užice bypass (State Road IB-28, M-19.1, Užice–Kadinjača–Bajina Bašta) required a major river crossing of the Đetinja River canyon, situated approximately 200 m upstream of the Velika Brana (Large Dam). The bypass section begins at the grade-separated "Surduk" interchange on the Zlatibor road and extends approximately 4,8 km to Sinjevac and Volujac.

The bridge site is characterised by a deeply incised limestone canyon. The water level (reservoir surface) at the bridge axis lies at approximately elevation 442 m a.s.l. The road profile, descending at a constant 3% gradient, crosses the canyon at elevations 536–537 m on the southern bank and 528–529 m on the northern bank, establishing a clearance of approximately 90 m above the water surface. Total structure length along the road axis is 294,2 m. The alignment is straight between stations 0+427.37 and 0+590.70, then passes through a clothoid transition (A = 111.13) and a circular curve of radius R = 190 m to the northern abutment at station 0+719.37.

The design was developed in two successive phases – Schematic Design (IDP, 2023) and Detailed Design for Building Permit (PGD, 2025) – both by DEL ING d.o.o., Belgrade. The structural analysis was performed using the SOFiSTiK software suite. Serbian regulations for building structures and related Eurocodes formed the normative framework. The relevant standards applied per action are: EN 1990 [7] for the basis of design and combinations of actions; EN 1991-1-5 [8] for thermal actions; EN 1991-2 [9] for traffic loads on bridges; EN 1998-1 [11] and EN 1998-2 [12] for seismic actions; and EN 1992-1-1 [10] and EN 1992-2 [13] for the design of the reinforced and prestressed concrete members. The investor is “Putevi Srbije" (Roads of Serbia).

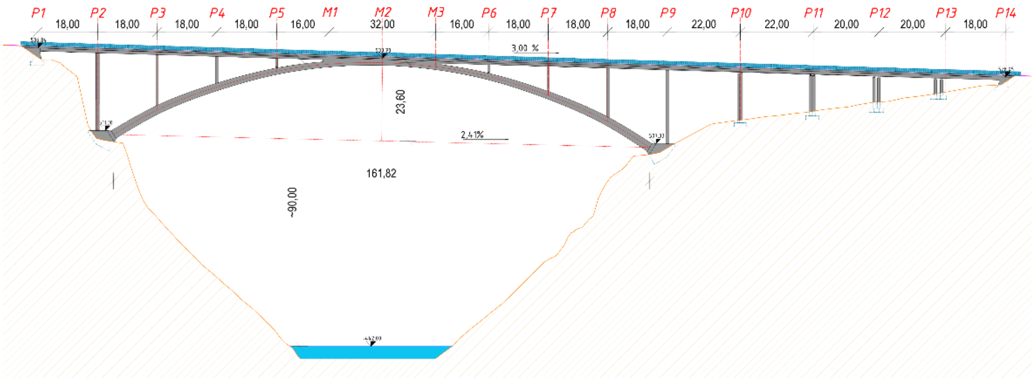

Figure 1: Longitudinal section - view of the arch bridge showing arch axis geometry, variable arch cross-section depth, and terrain profile with slopes

2. SITE CONDITIONS AND STRUCTURAL TYPE SELECTION

2.1. CANYON MORPHOLOGY AND FOUNDATION CONDITIONS

Both canyon walls consist of compact limestone rock. The lower portions of each bank are steeply inclined at approximately 60° to the horizontal. The left bank exhibits a pronounced terrain break – the slope transitions from approximately 13° to 54° measured in the road alignment direction, making access to the canyon floor and lower wall elevations extremely difficult. The right bank shows a similarly steep ridge of almost 60°. Laboratory testing of core samples from a borehole near the southern abutment established an allowable compressive bearing pressure of 20,4 MPa; design contact stresses at the arch foundations under ULS conditions reach 2,09 MPa (right bank) and 1,98 MPa (left bank), well within this capacity.

2.2. SELECTION OF BRIDGE TYPE

A reinforced concrete arch bridge was selected over competing alternatives: prestressed concrete girder viaduct (multiple spans), a steel or composite girder bridge, or a cable-stayed bridge. The governing reasons were:

(1) Canyon geometry: The steep, competent limestone rock faces on both banks are ideal for receiving the large horizontal thrust of an arch, obviating the need for massive spread-footing or piled abutments.

(2) Span range: At 162 m, a reinforced concrete arch is among the most economical structural solutions; the span-to-weight efficiency of arch action surpasses that of equivalent girder or cable-stayed systems at this crossing height.

(3) Durability and maintenance: A monolithic reinforced concrete arch requires no bearings (apart from the concrete hinges discussed in Section 5), no cable replacements, and no expansion joints along the arch itself, minimizing lifetime maintenance costs.

(4) Aesthetics: An open-spandrel arch bridge is visually coherent with the canyon landscape; the form reflects the structural logic, echoing the existing nearby railway arch also in reinforced concrete.

Table 1: Comparative assessment of structural options for the Đetinja bridge

Criterion	RC Arch	PC Girder Viaduct	Cable-Stayed	Steel Box Girder
Foundation suitability	★★★	★★	★★	★★
Economy at 162 m span	★★★	★★	★	★
Durability / maintenance	★★★	★★	★	★★
Construction complexity	★	★★★	★★	★★★
Aesthetic integration	★★★	★★	★★	★★
OVERALL	★★★	★★	★★	★★

3. ARCH GEOMETRY AND THRUST LINE OPTIMISATION

3.1. RISE-TO-SPAN RATIO

The arch has an axial span of L = 161,82 m and a rise of f = 23,60 m (measured between the chord connecting the two spring points and the arch crown), yielding a rise-to-span (flatness) ratio of f/L = 1/6,86. Conventional design recommendations for circular or parabolic arch axes suggest a flatness ratio $L/f \leq 6$, to limit horizontal thrust and bending sensitivity. The adopted ratio marginally exceeds this limit, imposed primarily by the contractor's requirement (documented in the Detailed Design phase) to position the arch springs as high as possible on the canyon walls, improving access during construction.

A shallower arch introduces three mutually reinforcing structural challenges:

(a) Increased horizontal thrust: For a uniformly loaded arch of span L and rise f, the horizontal thrust is $H \approx qL^2/(8f)$. As f decreases, H rises sharply, imposing proportionally higher compressive stresses on the arch cross-section and larger forces on the foundations.

(b) Sensitivity to asymmetric loading: For a fixed arch on rigid supports, bending moments due to asymmetric live load are proportional to the thrust H times the deviation of the arch axis from the pressure line. A shallow arch has a larger H and is geometrically less tolerant of pressure-line deviation.

(c) Temperature sensitivity: Thermal elongation or shortening of the arch generates horizontal displacements at the abutments, causing large bending moments proportional to $3EI \cdot \alpha \Delta T / f^2$ (approximate, for a fixed parabolic arch), which increase rapidly as f diminishes.

The spring points are deliberately placed at different elevations: the chord connecting them falls at approximately 2% gradient, roughly half the road gradient of 3%. This asymmetry was introduced to follow the terrain and to position the arch axis perpendicular to the superstructure centreline, maximising the effective rise available.

3.2. SPAN LAYOUT AND ARCH–DECK INTERACTION

The arch stiffness is highly driven by the stiffness of the girder.

Because the arch crown and the deck are monolithically connected over the central 36 m, the arch and the superstructure act as a coupled system: in this merged zone the effective bending stiffness is the combined contribution of both members, so the deck girder participates directly in the stiffness of the arch. Once the arch axis had been established, the next design step was to determine the most economical, yet aesthetically acceptable, number and spacing of spandrel columns. The spans were selected so that the hogging and sagging bending moments in the continuous deck girder are as uniformly distributed as possible, while respecting the rigidity (stiffness) of the supporting arch. Balancing the support and span moments in this way allows a single, near-uniform reinforcement and prestressing layout to be carried along the deck, which is both economical and constructionally simpler. This span optimisation is closely coupled with, and feeds directly into, the iterative thrust line construction described in Section 3.4, since the column positions define where the deck reactions (point loads) are applied to the arch.

Choice of a prestressed deck. Although the adopted span lengths (around 20 m) would be suitable for an ordinary reinforced concrete deck, the girder was deliberately designed as prestressed. Two benefits follow from this decision. First, in combination with an arch that remains fully in compression, prestressing of the deck provides a reliable guarantee that the sections remain uncracked in the serviceability limit state (SLS), preserving full gross-section stiffness and durability. Second, keeping both the arch and the central deck uncracked limits the time-dependent redistribution of stresses from the arch to the deck in the central monolithic zone caused by creep, since redistribution is driven by differential stiffness loss, which cracking would otherwise highlight.

A minimal structural depth was chosen for the deck girder. With spans of about 20 m, a very deep girder is neither structurally necessary nor aesthetically desirable — a heavy, high deck riding above a slender arch reads poorly against the canyon. Instead, the global stiffness of the structure was intentionally provided by the arch rather than by the deck. This is why the variable-height box section (and, equivalently, the twin-rib / TT girder solution) was adopted: it keeps the deck visually light while the arch, through its variable box cross-section, supplies the in-plane stiffness the system requires.

3.3. ARCH CROSS-SECTION

The arch has a hollow box cross-section throughout its length, a decision taken in the Detailed Design phase to replace the partially solid sections (with trapped formwork) originally proposed in the Schematic Design. The switch was motivated by: (i) increased stiffness to reduce deformation sensitivity during cantilever construction; (ii) the ability to use telescopic formwork for hollow casting; and (iii) uniformity of behaviour.

Cross-section dimensions vary linearly along the arch (Figure 1). At the abutments (pier axes P2 and P9) the cross-section is 700 cm wide × 270 cm deep (notation width/height including 10 cm bottom grooves on the external face). At the crown zone, dimensions reduce to 600 cm wide × 150 cm deep. The constant-width zone of 600 cm extends symmetrically for 66 m around the arch crown. Flange plate thickness is 30÷34 cm and web thickness is 60 cm throughout.

The structural rationale for the variable depth is threefold. First, the larger moment of inertia at the abutments accommodates the peak bending moments arising from asymmetric traffic loading and thermal effects – loads that a shallow arch is particularly susceptible to. Second, reducing the depth toward the crown decreases the arch weight in the compression-critical crown zone, lowering the risk of in-plane buckling (the governing stability mode for shallow arches). Third, from an aesthetic standpoint, a visually tapering arch communicates the structural logic of the form: carrying more where the demands are greater.

The in-plane buckling safety of the arch was verified by second-order analysis including geometric and material nonlinearity, confirming adequate stability margin under all design load combinations.

3.4. ITERATIVE THRUST LINE CONSTRUCTION

Since no single load configuration produces a unique pressure line, the arch axis was optimised for minimal bending moments under the governing permanent load state at both $t = 0$ (initial) and $t = \infty$ (after creep and shrinkage redistribution). The process, carried out iteratively between the structural analysis model (SOFiSTiK) and manual graphical verification, proceeded as follows:

Step 1: An initial circular arc of radius $R \approx 150$ m was assumed as the arch axis.

Step 2: A full spatial FEM analysis yielded pier reactions and arch self-weight distribution.

Step 3: A thrust line (pressure line) was constructed graphically using the polygon of forces (funicular polygon), connecting the successive resultant forces at each pier and distributing the arch weight as a series of point loads.

Step 4: Selected nodal points of the arch axis were shifted to reduce eccentricity between the arch centroid and the thrust line.

Step 5: The analysis was repeated. Convergence was declared when no tensile stresses under the frequent combination (partial traffic + temperature change ΔT) remained, and all tensile stresses under characteristic combination are below allowable limits throughout the arch length.

This iterative optimisation, described originally in the literature by Salonga & Gauvreau [1] as fundamental to efficient arch design, is particularly important for shallow arches because even modest misalignment between arch axis and pressure line generates significant bending moments through the amplifying effect of the large horizontal thrust.

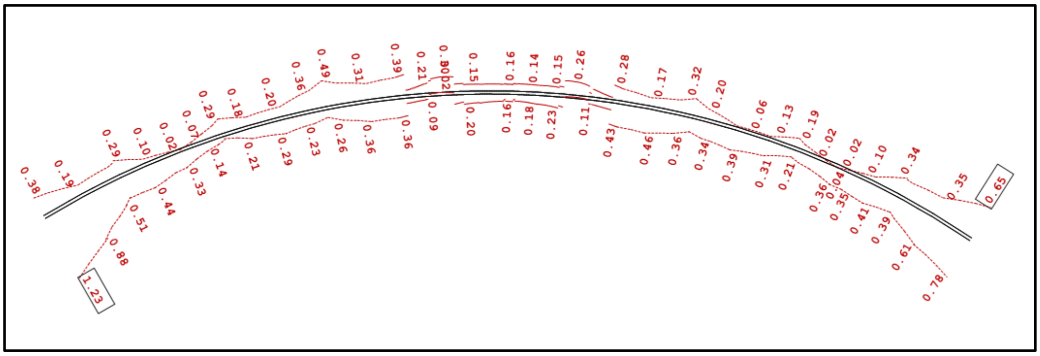

Figure 2: Thrust line due characteristic comb. (from SOFiSTiK): initial circular arc axis

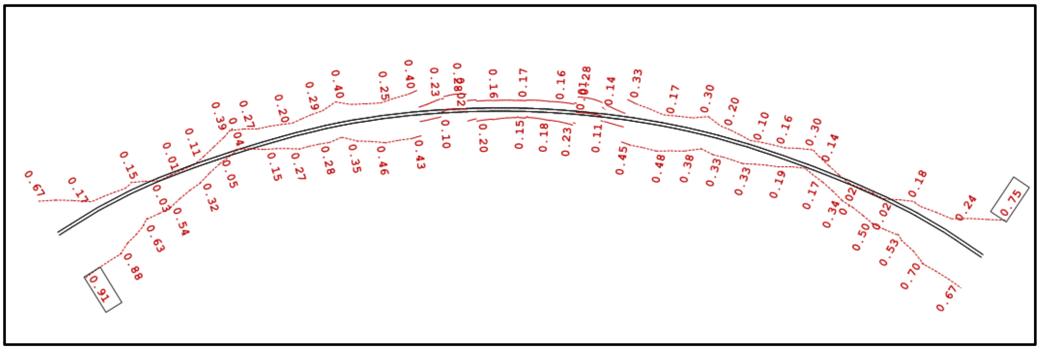

Figure 3: Thrust line due characteristic comb. (from SOFiSTiK): optimized arc axis

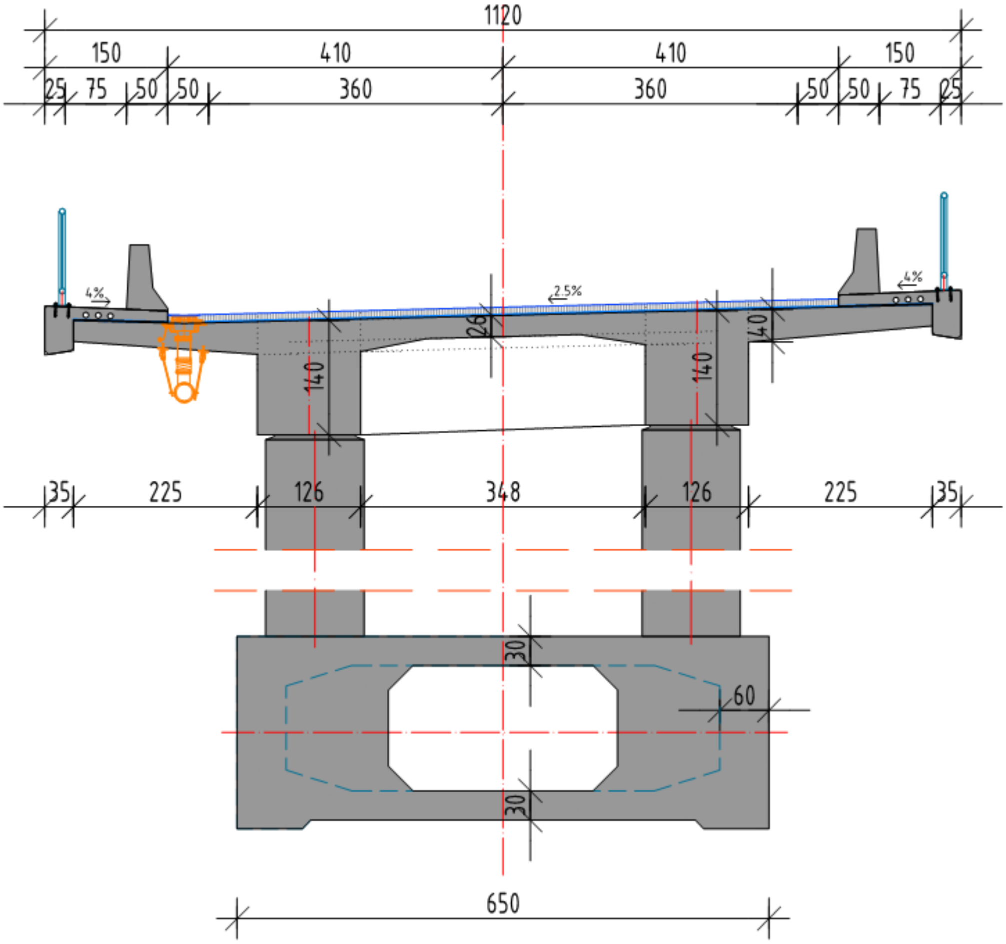

Figure 4: Detail cross-sections showing; variable arch box cross-section from spring 700/270 cm to crown 600/150 cm; concrete hinge detail at arch piers. (Source: DEL ING d.o.o., PGD drawings, 2025)

3.5. SUPERSTRUCTURE CROSS-SECTION

The superstructure is a continuous prestressed concrete twin-rib (TT) girder of constant depth 140 cm, spanning between pier tops at intervals of 14 m (crown zone, fields 5 and 6) and 18–22 m (outer fields). The rib width was increased from 120 cm (Schematic Design) to 126 cm (Detailed Design) to accommodate the prestressing cable ducts and coupler anchors.

The total deck width is 11,20 m, comprising: two 3,60 m traffic lanes (7,20 m), two 50 cm buffer zones to safety barriers (1,00 m), two 50 cm concrete safety barriers (1,00 m), two 75 cm inspection walkways (1,50 m), and two 25 cm walkway guard rails (0,50 m). On the transition curve and circular arc portion, the left lane widens progressively to 4,10 m, resulting in a total cross-section width of approximately 11,70 m at the bridge end.

A 36 m central length of the superstructure is monolithically merged with the arch at the crown, a measure that maximises the effective arch rise while also providing significant composite stiffness at the most vulnerable section for snap-through and asymmetric buckling.

4. STRUCTURAL BEHAVIOUR UNDER DESIGN ACTIONS

4.1. PERMANENT LOADS AND TIME-DEPENDENT EFFECTS

Under permanent loading (self-weight and superimposed dead load), the arch is essentially in a state of pure compression once the arch axis has been aligned with the pressure line. The analysis at time $t = \infty$ must account for creep and shrinkage redistribution: creep reduces the elastic modulus of the concrete over time, altering the stiffness distribution between arch and superstructure. The redistribution shifts some permanent-load moment from the stiffer arch to the superstructure, a favourable effect that was incorporated into the design envelope.

4.2. VARIABLE (TRAFFIC) LOADS

The bridge is classified as a road bridge of Category Ib (motorway standard, carriageway width B ≥ 6.00 m). Traffic loading was applied according to the Load Model LM1 (tandem systems + uniformly distributed load) of the applicable Serbian regulations. The governing effect of variable loading on an arch bridge is the half-span (asymmetric) loading case: loading one half-span while leaving the other unloaded maximises the bending moment in the arch rib. For a shallow arch, this bending moment is amplified because the thrust H is large and acts as a magnifying factor on any eccentricity of the pressure line from the arch axis.

The arch's variable cross-section contributes positively here: the enlarged cross-sections near the springs provide additional bending resistance exactly where asymmetric loading bending moments peak. The intermediate arch piers rigidly connected to the arch also contribute stiffness against in-plane deformation.

4.3. THERMAL ACTIONS

Temperature actions are among the most critical for a shallow arch. A uniform temperature change ΔT of the arch produces a horizontal elongation or shortening of the arch chord, effectively shifting the abutments horizontally. Since the abutments are embedded in rock (practically rigid), this displacement induces bending.

A differential temperature gradient (top warmer than bottom in summer, or vice versa in winter) induces additional sagging or hogging in the arch. For the Đetinja bridge, the temperature difference between arch top surface and soffit was taken as $\Delta T,top$ = +8°C ($\Delta T,bot$ = −5°C), and the uniform temperature range was determined from the climatic zone (continental, Užice area). The arch cross-section dimensions and the degree of prestress in the superstructure were verified to ensure that, under the frequent combination including temperature, tensile stresses in the arch concrete do not exceed allowable values. This requirement was the controlling constraint in the thrust line iteration process described in Section 3.4.

4.4. SEISMIC ACTIONS

The structure is located in seismic zone Z3 per Serbian regulations, with a design return period of 475 years on soil category A (rock), the acceleration is $a_{g,R}$ = 0,15g. The arch was analysed as a three-dimensional structural system; the arch's high in-plane stiffness makes it relatively insensitive to horizontal seismic excitation in the arch plane, while out-of-plane seismic behaviour governs the pier and arch rib design. Peak contact stresses at the arch foundations under seismic combinations reach 3,21 MPa (right bank) and 3,60 MPa (left bank), still within the geotechnical allowables.

The seismic analysis followed EN 1998-2 [12]. The bridge was designed to remain essentially within limited ductile / essentially elastic behaviour, adopting a behaviour factor q = 1,2 as recommended by EN 1998-2 (Table 4.1) for arch bridges. No plastic hinge mechanism or intentional energy dissipation was assigned to the arch or the piers; the concrete hinges are not ductile plastic hinges but articulations that release bending while transmitting compression, and were therefore not relied upon for seismic energy dissipation. Consequently, the members were verified to remain in the elastic range under the design seismic action, and no capacity-design over-strength was required.

5. CONCRETE HINGES AT ARCH PIERS

5.1. HISTORICAL DEVELOPMENT AND WORKING PRINCIPLE

Concrete hinges are local neck-reductions in reinforced concrete members, designed to concentrate rotation capacity at a defined point while transmitting large compressive normal forces. The concept was first implemented practically by Augustin Mesnager (France, 1907), who developed a semi-articulation with a pair of crossed steel bars running through the narrow throat, enabling the transfer of both axial forces and shear while limiting bending stiffness [2, 3].

The Freyssinet hinge permits rotation about one axis (in the plane of the structural element) while remaining moment-resistant in the perpendicular direction. Load transfer across the throat relies entirely on the confined concrete; no reinforcement crosses the throat. The throat geometry (width and height) governs both the rotational flexibility and the compressive capacity. Properly designed and executed concrete hinges have demonstrated outstanding long-term performance, documented Freyssinet hinges more than 100 years old show no deterioration [5]. Despite this record, their use diminished in the latter twentieth century primarily because current design codes do not explicitly treat them and the design still relies on semi-empirical guidelines [2, 3, 5].

5.2. APPLICATION IN THE ĐETINJA BRIDGE

In the Đetinja bridge design, concrete hinges are used in two structural contexts:

Short arch piers (pendel system): The two shortest pier pairs on the arch, at heights of 2,25 m and 2,60 m (measured clear between arch intrados and superstructure soffit), are designed as pendel struts – slender compression members with concrete hinges at both top and bottom ends. At these minimal heights, even the modest horizontal displacement of the arch crown under asymmetric loading or thermal expansion on the order of 5–15 mm would generate moment demands in a rigidly connected pier exceeding its capacity. The hinge reduces the bending stiffness of the connection to nearly zero, converting the pier to a pure axial-force (compression) member.

First abutment-adjacent stiff piers: The piers at the arch abutment elevation (P2/P9 in the pier numbering) are rigidly connected to the arch at their base and connected to the superstructure via concrete hinges at their tops. This allows the superstructure to rotate slightly relative to the pier head, accommodating differential deflection without generating large secondary moments. Also, pier P3, P4 and P7, P8 are designed with hinges on top.

5.3. DESIGN PRINCIPLES AND CONSTRAINTS

The design of the concrete hinges followed the recommendations established by Leonhardt and subsequently refined by Schlappal [2, 3], adapted for compatibility with the partial-safety-factor framework of Eurocode 2. The key design parameters and constraints are:

Throat width b_t and height h_t: The throat dimensions are set such that the compressive stress in the throat concrete under the characteristic normal force remains below the effective strength of the triaxially confined throat. The confinement factor for a well-proportioned Freyssinet hinge typically raises the effective compressive strength to approximately 3÷5 times the unconfined cylinder strength f_{ck}.

Sustained loading limit: Based on experimental research [2, 3], permanent compressive normal forces shall not exceed approximately 45% of the ultimate hinge capacity, to avoid time-dependent damage from the interaction of creep and bending-induced cracking.

Rotation capacity: The throat must accommodate all expected rotations arising from thermal movements, differential settlement, and traffic-induced deformations, without exceeding the concrete's ultimate compressive strain at the outer fibre. Required rotations were estimated from the spatial structural model and verified against the assumed throat geometry.

5.4. ADVANTAGES OVER MECHANICAL BEARINGS

Concrete hinges offer several distinct advantages over mechanical bearings (laminated elastomeric, pot, or PTFE slide bearings) that made them the preferred solution for the short pendel piers: zero maintenance, no uplift risk, high durability, economy. For members permanently under high compression (as all arch piers are), the concrete hinge is simpler and more economical to fabricate than a mechanical bearing designed for the same compressive force.

6. CONSTRUCTION METHOD

The arch is constructed by the balanced cantilever method (Figure 5), using a self-propelled traveling form that advances segment by segment from both abutments simultaneously toward the crown. Temporary towers of 50 m height are erected above each arch abutment, immediately behind the spring line. The key elements of the temporary works system are:

Back-stay cables: Anchored into the rock mass behind each abutment via pre-stressed rock anchors (geo-anchors), these cables resist the overturning moment at the base of the temporary tower induced by the cantilever arch half-span weight.

Forward stay cables: Inclined stays extending from the temporary tower tip to successive segments of the growing cantilever arm. Each stay is tensioned as a new segment is cast and released (if necessary) after the subsequent segment is in place, maintaining the arch geometry within specified tolerances.

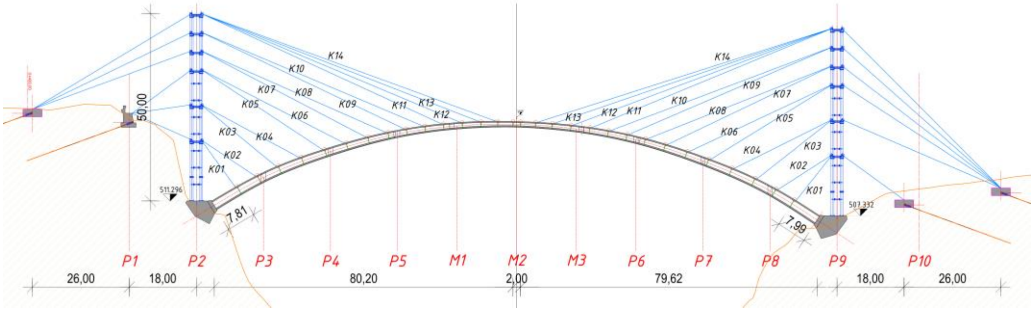

Figure 5: Construction stage diagram showing temporary towers, back-stay anchors, forward stay cables, and travelling form advancing from both abutments toward the crown

Construction stages were modelled in full in SOFiSTiK, covering: traveler advancement CS x(9), segment self-weight CS x(10), stay cable installation without force CS x(12), cable stressing CS x(14), optional cable release CS x(15), and time-dependent creep and shrinkage for each construction period CS x(18). The most critical phases are the late cantilever stages when the full (heaviest) segments near the crown are cast; analyses showed that a stiffer arch (deeper cross-section at the crown, 150 cm vs. 120 cm as in the Schematic Design) was much more controllable during execution.

After arch closure, the temporary stays are progressively destressed and removed, the traveler is dismantled, and the superstructure (continuous TT girder) is cast field-by-field on falsework bearing directly on the completed arch. Prestressing follows a field-by-field sequence starting from both end spans simultaneously (P1→P2 and P14→P9), converging symmetrically toward the crown. Six prestressing cable families (A1–A7) are interlaced so that the bending moment envelopes in all construction and final states remain within allowable limits.

7. CONCLUSIONS

The Đetinja River arch bridge demonstrates the feasibility and structural rationality of a relatively shallow reinforced concrete arch (f/L = 1/6,86) for a deep canyon crossing where site access constraints push the arch springs high on the canyon walls. The key design conclusions are:

1. Iterative thrust line optimisation by the polygon-of-forces method, implemented within a spatial FEM framework, successfully minimises permanent-load bending moments in the shallow arch.

2. A variable-depth hollow box cross-section, deeper at the springs, shallower at the crown – serves dual structural and aesthetic purposes: higher section stiffness at the locations of maximum asymmetric-load and thermal bending moments, and reduced weight at the buckling-critical crown zone.

3. Concrete Freyssinet hinges at the short pendel piers provide a maintenance-free, durable alternative to mechanical bearings, eliminating the bending moments that would otherwise arise from even small horizontal displacements in these squat members.

4. The loads governing the design differ by member and direction, which is characteristic of a relatively shallow arch integral with its deck. Longitudinally, the combination of temperature and traffic is decisive: the arch and the superstructure remain fully in compression under the quasi-permanent and frequent combinations, and even under the characteristic (rare) combination the tensile stresses stay below the limits. Transversely, the out-of-plane seismic action governs the design of the arch, the superstructure, and the piers. For the arch cross-section itself, the enlarged sections at the springs are governed by the peak bending moments from asymmetric (half-span) traffic and thermal actions. The short pendel piers are governed by axial compression combined with the imposed rotations that dictate the concrete-hinge throat geometry.

5. A detailed construction stage analysis was essential for establishing the temporary tower geometry, cable stay sequencing, and the cross-section proportions necessary to achieve the required arch geometry within acceptable tolerance.

REFERENCES

[1] Salonga, J., Gauvreau, P. (2014). Comparative study of the proportions, form, and efficiency of concrete arch bridges. Journal of Bridge Engineering, ASCE, 19-3, 04013010-1.

[2] Thomas Schlappal et al. (2020). Serviceability Limits of Reinforced Concrete Hinges. Engineering Structures.

[3] Thomas Schlappal et al. (2020). Ultimate Limits of Reinforced Concrete Hinges. Engineering Structures. https://doi.org/10.1016/j.engstruct.2020.110889

[4] Guojing, Zhang et al. (2022). Optimal arch shape of long-span open-spandrel arch bridges under vertical permanent loads. Structures, 45 (2022).

[5] Schacht, G., Marx, S. (2015). Concrete hinges in bridge engineering. ice publishing, Engineering History and Heritage, Vol. 168 (EH2).

[6] Geotechnical Report (2025). Geotechnical conditions for construction of Užice bypass (PGD). Institut za puteve A.D., Belgrade.

[7] SRPS EN 1990:2012 Eurocode – Basis of structural design (all EN with Serbian NA).

[8] SRPS EN 1991-1-5:2012. Eurocode 1 – Actions on structures, Part 1-5: General actions – Thermal actions.

[9] SRPS EN 1991-2:2012. Eurocode 1 – Actions on structures, Part 2: Traffic loads on bridges.

[10] SRPS EN 1992-1-1:2015. Eurocode 2 – Design of concrete structures, Part 1-1: General rules and rules for buildings.

[11] SRPS EN 1998-1:2015. Eurocode 8 – Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings.

[12] SRPS EN 1998-2:2012. Eurocode 8 – Design of structures for earthquake resistance, Part 2: Bridges. CEN, Brussels.

[13] SRPS EN 1992-2:2014. Eurocode 2 – Design of concrete structures, Part 2: Concrete bridges – Design and detailing rules.

[14] SOFiSTiK AG. SOFiSTiK Analysis Programs – User Manuals.