

Croatian Bridges designed by Structural department of Zagreb Civil Engineering Faculty utilizing Sofistik software

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Zusammenfassung:

Einige grössere kroatische Brücken sind beschrieben, die alle mit Sofistik Programmen berechnet wurden. Die Projekte aller diesen Brücken wurden ausgeführt von der Konstruktiven Abteilung der Fakultät für Bauwesen, Universität Zagreb, Kroatien. Die Rijeka Dubrovačka Brücke ist eine Schrägseilbrücke mit Verbundüberbau, die Krka Autobahnbrücke ein Stahlbetonbogen mit Verbundüberbau, die Mirna Brücke eine Balkenbrücke im Verbund und die Jasenovac Brücke eine stählerne Balkenbrücke. Um mit älteren Versionen von Sofistik einfacher umgehen zu können, wurden einige Vorprogramme geschrieben in parametrischer Darstellung, die sich an Sofistik, AutoCad und MS Word Programme anlehnen, sehr ähnlich zu den neuen Sofistik Programmen CSM und SSD.

1 BRIDGE ACROSS RIJEKA DUBROVACKA

1.1 Introduction

The works on the bridge started in 1990, before the war for Croatian independence, when approach roads and west bank abutment were constructed. After the war, the original design was changed and the works restarted in 1999. Works on the bridge construction started again in 1999, based on a modification of the original design.

1.2 Design

The asymmetrical layout with one pylon had to be chosen because the motorway from the west enters the bridge in a sharp curve, so that the more economical three - span cable-stayed structure was not possible. The location of the bridge is in a highly active seismic zone and subject to very strong winds, which strongly influenced the bridge design.



Figure 1.1: Photo of the completed bridge.

The bridge consists of the main cable-stayed bridge with composite superstructure with spans of 244 + 80,7 m, and the 87,4 m box type prestressed concrete approach viaduct with the 60 m cantilever. The overall bridge length between abutment ends is 518.23 m (Fig.1.1).

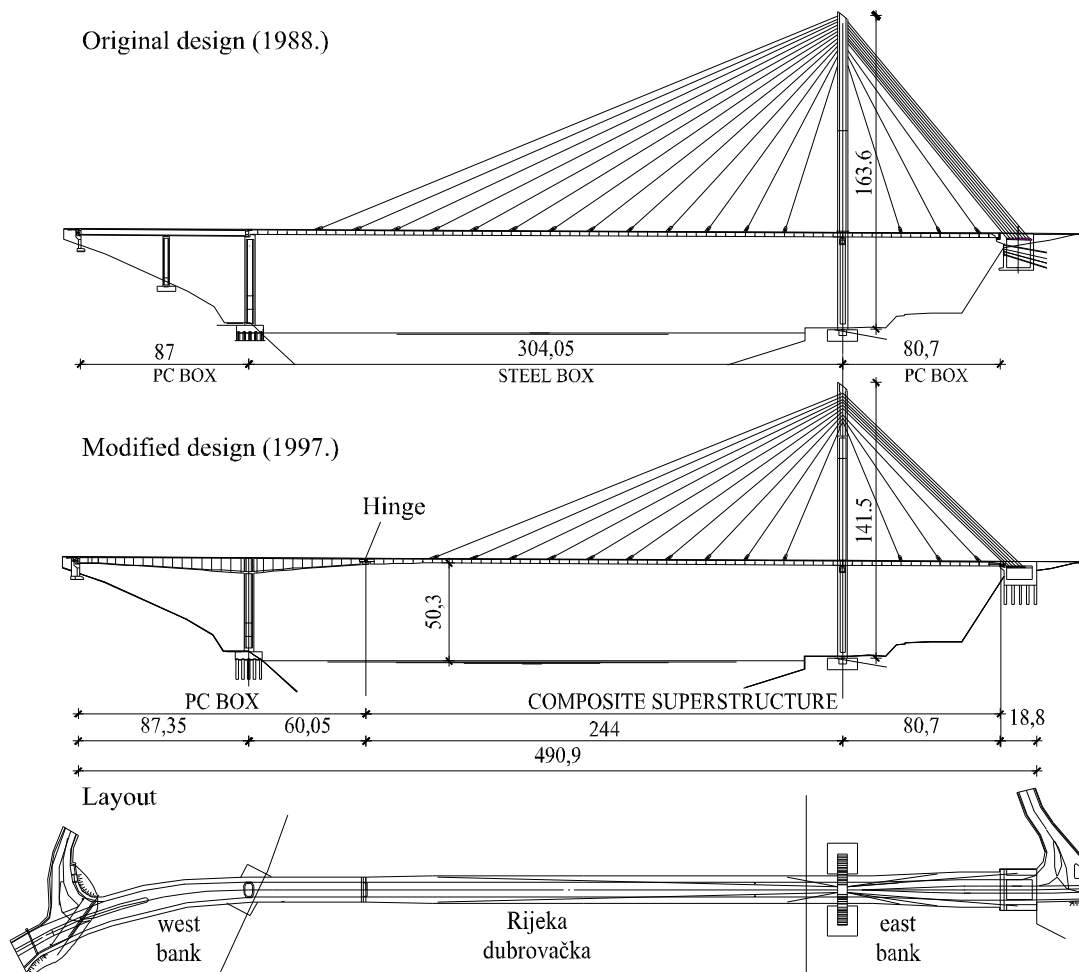


Figure 1.2: Old (top) and new (bottom) design of the bridge across the Rijeka Dubrovačka.

According to the original design, the bridge should have had two separate structures, the skew and curved approach viaduct on the west side in prestressed concrete and the main cable - stayed bridge (Fig. 1.2, on the top). The stiffening girder of the cable - stayed bridge consisted of prestressed concrete beam in side span and steel box type girder in main span, interconnected for complete flexural continuity at the pylon. The ratio of the side span in prestressed concrete to the main span in steel was 0,30. The "A" shaped concrete pylon was 163,6 m high, so that ratio of the height of the pylon to the main span was amounted to 0,37.

The Contractor changed the original design in order to simplify the construction procedure and to reduce the maintenance costs. The following changes have been made:

- a) The prestressed concrete box-type approach viaduct on the right bank has been extended into the main span by 60 m and the intermediate pier eliminated. This led to the reduction of the suspended main span by 60 m.
- b) The superstructure of the main bridge was changed to open composite type, comprising the steel grillage of two main girders 2 m high, cross girders and concrete deck plate of 25 cm thickness (fig. 1.3). Subsequent wind tunnel testing revealed that bottom wind bracing is to be installed, to increase the torsional stiffness of the superstructure cross section.
- c) The height of the pylon has been reduced to 141,5 m.

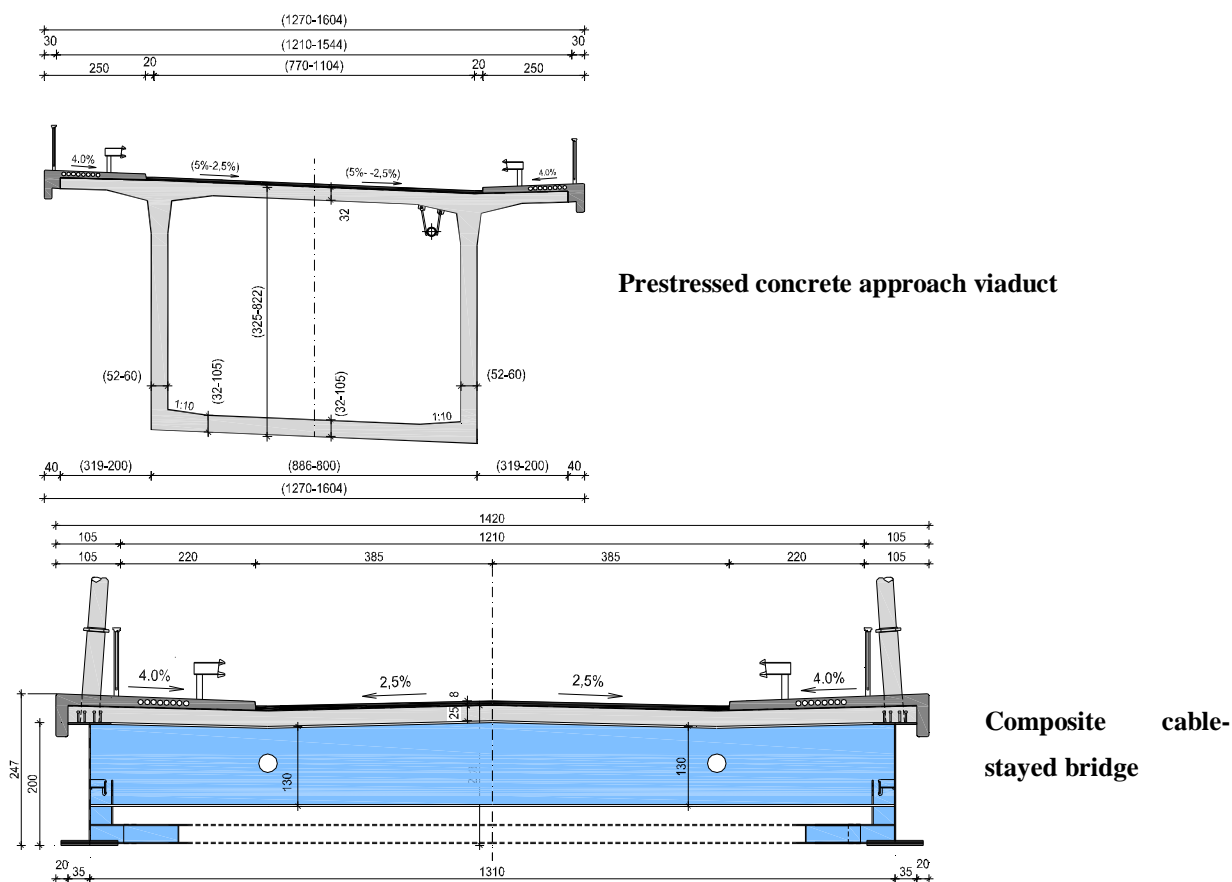


Figure 1.3: Cross sections of the Rijeka Dubrovačka bridge.

The longitudinal layout of the cables is of modified fan type with partial suspension. Cable - stays in two inclined planes are spaced at 20 m.

Main girders are 2 m high. The beam is supported at the end of the approach viaduct cantilever, at the pylon cross-beam and fixed at the east abutment. The steel grade is St 52-3 according to DIN and the concrete deck is C 35/45.

The "A" shaped concrete pylon is of box-type, with walls max. 60 cm thick (Fig. 1.4). Cable stays transfer large forces to the top of the pylon, so that it had to be prestressed longitudinally and transversally. The designer offered an alternative with cable anchorages to the pylon in steel, but it was rejected for economic reasons. The complicated executed detail is highlighted in Fig. 1.4.

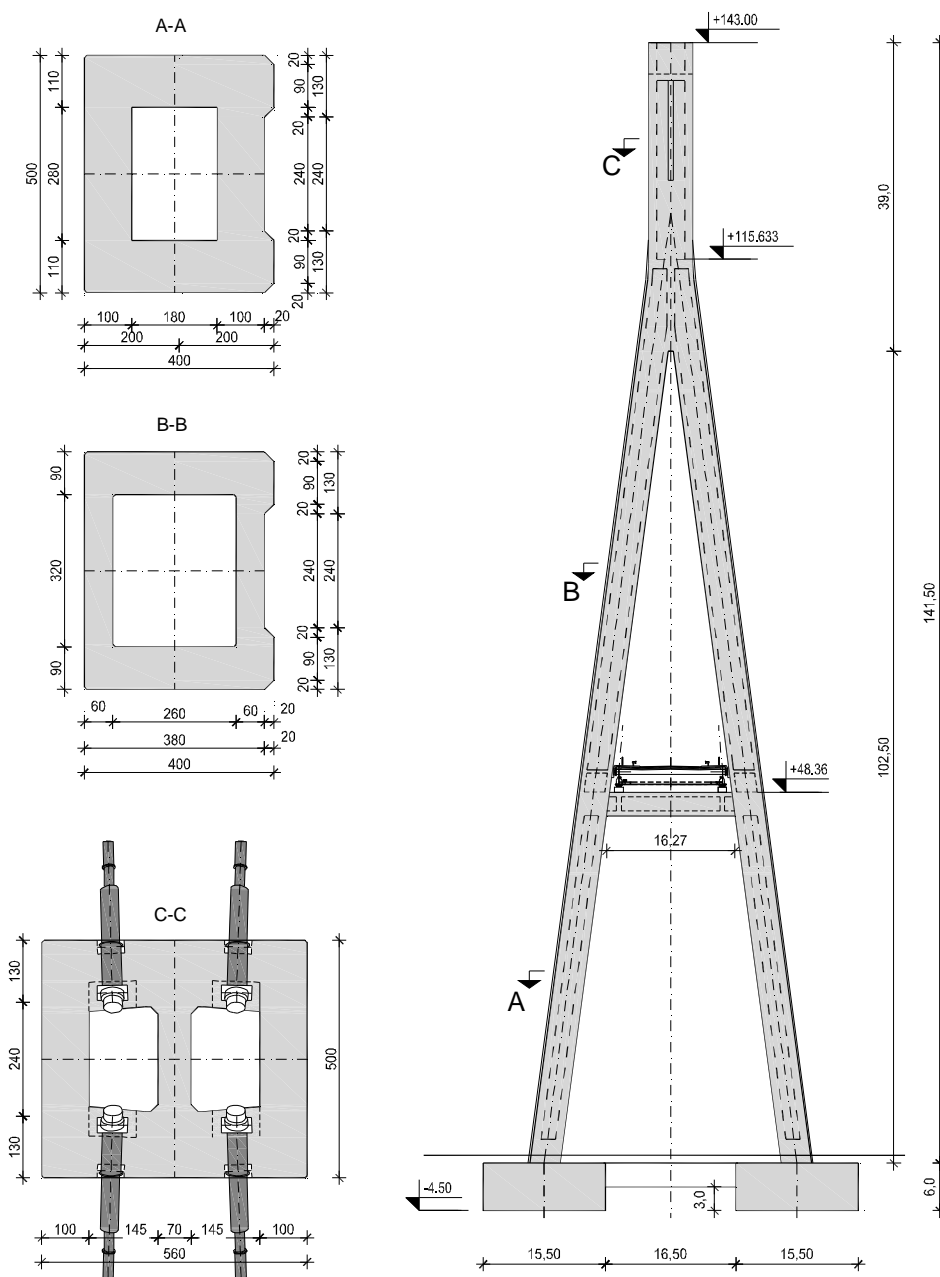


Figure 1.4: Pylon of the Rijeka Dubrovačka bridge.

The prestressed approach viaduct is partially curved in plan. The superstructure depth varies from 3,2 m at the west bank abutment and at the cantilever end to 8,2 m at the fixed connection to the pier (Fig. 1.3). Two hydraulic dampers of 200 tons capacity are installed at the connection of the viaduct to the west bank abutment, acting in the direction of the bridge axis to act as fixed bearings in case of a major earthquake and thus reduce earthquake actions on the pier (Fig.1.5). The pier base is designed to act as plastic hinge in seismic event.

Design concrete grades were C 35/45. The concrete cover was 5 cm for the pylon, the pier and the superstructure, and 10 cm for the foundations and piles under sea level.

The substructure of the bridge across the Rijeka Dubrovačka comprises:

- west bank abutment which has to take up large vertical actions from the superstructure and uplifting forces in the case of a earthquake (Fig. 1.5),
- approach viaduct pier (Fig. 1.6)
- pylon, whose foundations are located very close to the sea
- east bank abutment which must carry very large uplifting force due to six cable-stays anchored in the abutment.

The west bank abutment has been built before the war, according to the original design. Actions of the superstructure increased due the enlargement of the adjacent span in the new design solution (from the original 43,5 m to 87,35 m). The bearing layout was also changed. Additional bearings for the uplift reactions were installed. For this reasons, the abutment had to be reconstructed and strengthened.

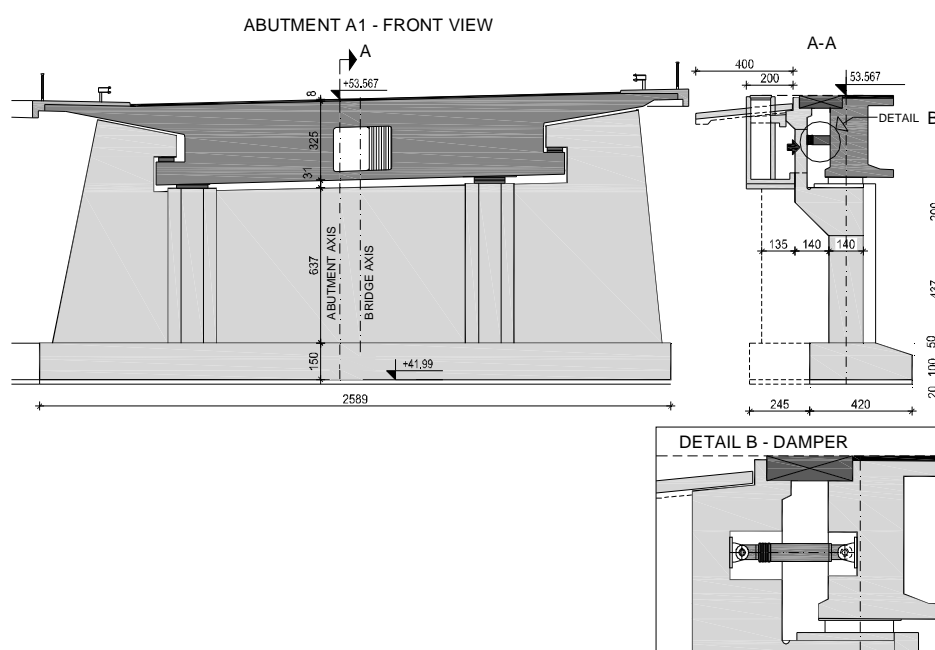


Figure 1.5: West bank abutment

The approach viaduct pier is placed into the narrow space between the sea and the main road. The pier cross-section is of box-type, with outer dimensions of 8.0*5.0 m. Foundations are on bored piles, connected at the top by a 4,0 m deep concrete plate (Fig. 1.6).

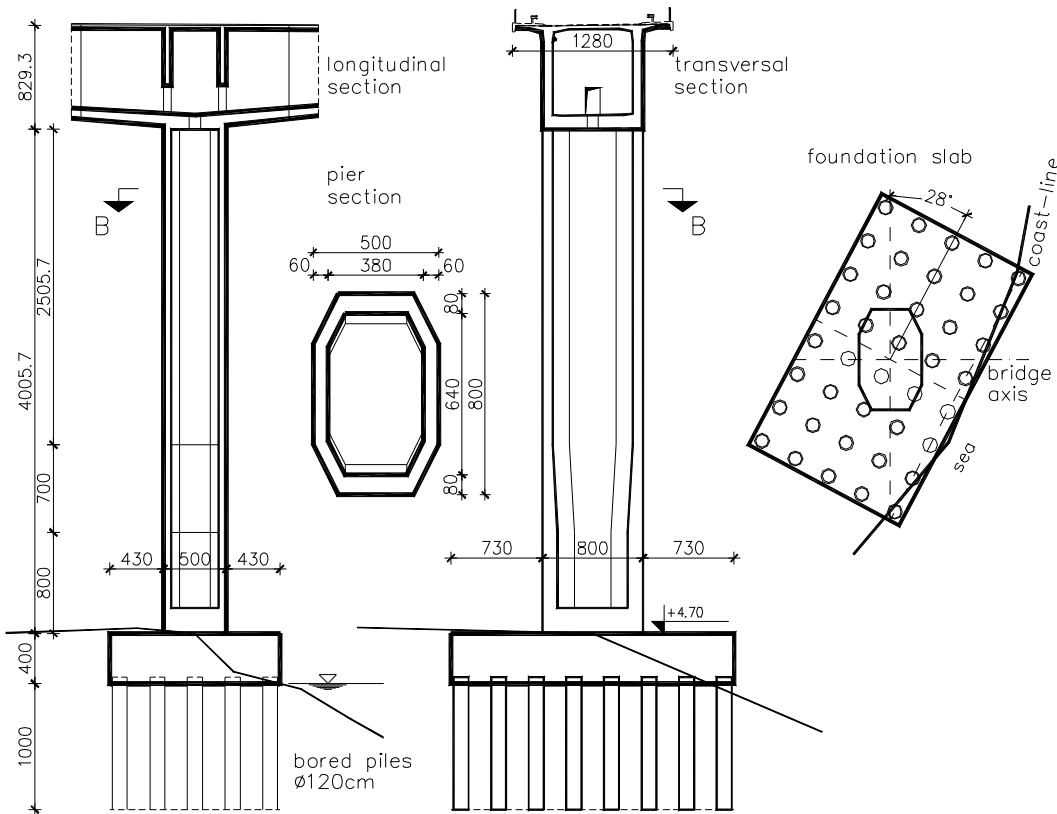


Figure 1.6: Approach viaduct pier

The east bank abutment (Fig. 1.7) is designed as a massive box structure, with vertically prestressed side walls. The interior is filled with stone. Six pairs of cable - stays are anchored into this abutment, and the main bridge superstructure is fixed into it.

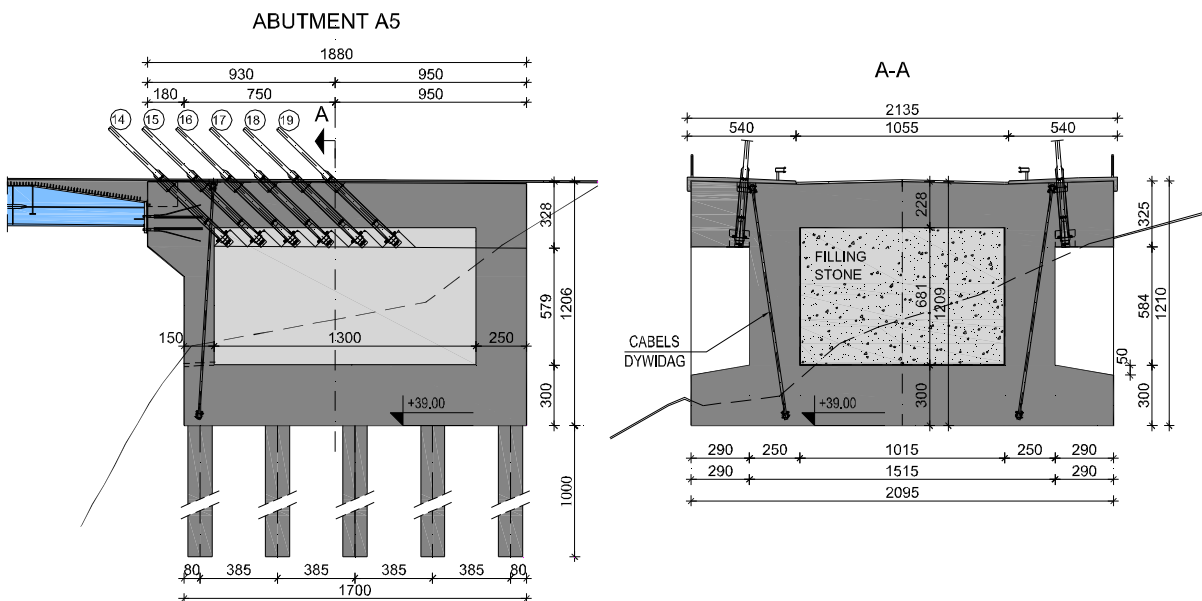


Figure 1.7: East bank abutment

Originally, the shallow slab foundation was designed for the east bank abutment. In the final solution deep bored piles foundation was adopted instead which enabled us to increase the behaviour factor q for the abutment to $q=1.5$, as bored piles are expected to behave with limited ductility.

Pylon legs are founded on slabs, 15.50*15.50 m large and 6.0 m thick. Slabs are interconnected in transversal direction by a beam, 4.0 m wide and 3.0 m thick. Twenty permanent geotechnical anchors had to be installed in longitudinal bridge direction, due to unfavourable inclination of the rock layers, in order to restrain the foundations against differential movements.

1.3 Structural analysis

The location of the bridge is characterised by influences of sea (salt), strong winds and high probability of strong earthquakes, so that extent preliminary investigations were necessary. The contract between the Croatian Government and Walter-Bau as the main contractor prescribed the utilisation of German DIN codes, except for the seismic design, which was to be performed according to EC8.

The Seismological study of the bridge location was performed on Zagreb University. The RWTH institute in Aachen performed non-linear dynamic analysis of the bridge, based on the artificial accelerograms, designed on the basis of real recordings of actual earthquakes in the area. The design acceleration was prescribed as 0,38g, based on 500-year return period.

Wind gusts from the north at the bridge location can reach speeds of up to 50 m/s. The wind tunnel testing was performed at RWTH, together with numerical simulations of dynamic behaviour of the structure. As a result, the wind load on the structure was determined as follows.

Wind loading on the superstructure:

| | |
|-------------------|------------------------------|
| Horizontal force: | 8.85 kN/m', |
| Uplifting force: | 10.4 kN/m', |
| Moment: | 13.06 kNm'/m' (beam torsion) |

Wind loading on the pylon: 3.25 – 6.64 kN/m² depending on the height above sea level.

All numerical analyses were performed utilising SOFISTIK program package. Various numerical models were used to simulate structural behaviour of completed bridge, including construction phases (global behaviour) and also structural behaviour of critical structural elements (local structural behaviour), such as the concrete deck plate of the composite superstructure, anchorage zone of the pylon, the hinge between the composite cable stayed superstructure and the approach viaduct, deep bored piles' foundations and similar.

Global behaviour was modelled with beam elements, while detailed local models combined beam and plate elements.

All governing loadings were analysed for prescribed combinations, for time $t=0$ and $t=\infty$:

- main loadings: self-weight, permanent load, traffic load, creep and shrinkage
- additional loadings: wind load, temperature, braking load
- accidental loadings: earthquake.

The governing DIN live loading scheme is SLW 60.

The main cable-stayed bridge was modelled as a space frame taking into account appropriate stiffnesses of the superstructure, steel only for dead weight and composite for all other loadings.

The deck slab model included both, beam type and plate type finite elements, representing 120 m of the deck, from the east abutment to the first stay behind the pylon. It was used to analyse influences of live loading and of stay connections, transmitting large normal forces into the deck. The nonprestressed reinforcement in the slab was designed superimposing actions of both global and local numerical models.

The stress-check of the main steel girders was performed utilising allowed stress concept, as prescribed in relevant DIN codes. The stability check of the main steel girder was also performed according to the German regulations.

The stays were dimensioned with maximum allowable stresses $0.45 \times 1770 = 796.5 \text{ N/mm}^2$ for main loadings and $0.5 \times 1770 = 885.0 \text{ N/mm}^2$ for main and additional loadings.

The largest stress for fatigue check is 116.9 N/mm^2 , which is less than allowed 124 N/mm^2 .

Cable stay steel anchor plates had to be designed with partial plastification in critical areas (Fig.1.8).

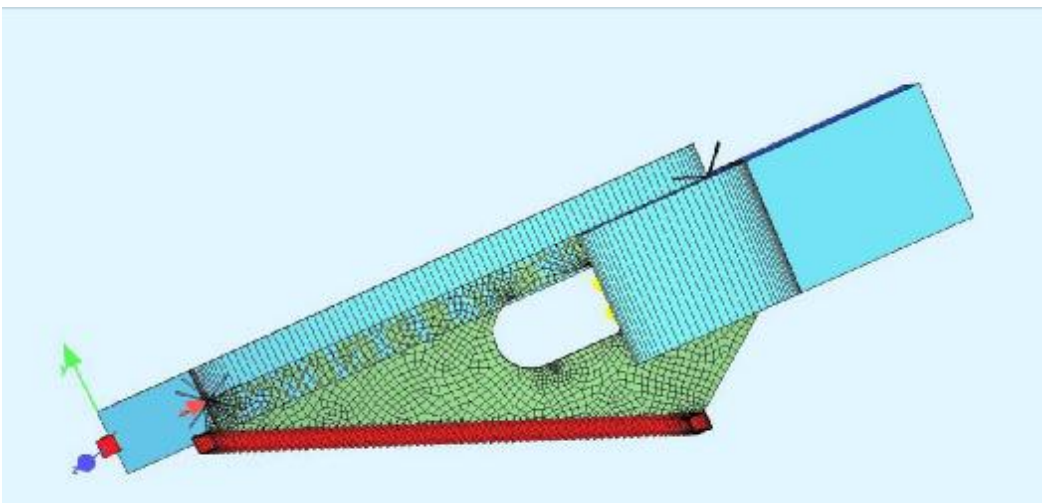


Figure 1.8: Numerical model of cable stay steel anchor plate

The construction sequence was strictly prescribed in the final design documents. Various possibilities for the construction of the deck slab were analysed; precast slab, semi-precast slab and finally monolithic slab was chosen.

A separate numerical model was used for the analysis of erection phases. The stiffness of the superstructure segments was adjusted to simulate the placing of the concrete slab. 141 phases were analysed on 12 different numerical systems. As a result, the construction procedure manual was created, defining all construction phases, including the values of the prestressing forces for all stays.

All activities were planned taking into account the following:

- stay tension should be within the prescribed limits
- stresses in steel girders and in the concrete slab should be within allowable limits
- pylon must remain nearly vertical (front and back stays must have been stressed in turns)
- transverse force at the pylon top should be limited
- composite beam deflections have to be under control
- derrick crane was too wide to pass between already installed pair of stays, so that it had to be moved forward in 10 m increments.

The wind loading on the pylon was based on wind tunnel tests. The values used for the bridge in service were based on the hundred year return period and for construction phases on a ten years return period.

The pylon was numerically modelled with beam elements, and the soil with springs. 26 construction phases were analysed. All relevant loads were taken into account: self weight of the pylon, scaffolding and formwork, the influence of the crane fixed to the pylon walls and the wind load. Horizontal deformations occurred due to inclination of the pylon legs. They were calculated and compensated for during construction sequence.

The pylon during construction is longitudinally a cantilever 142.5 m high. Later, when the stays are in place the pylon is partially restrained and its slenderness decreases. The wind load on completed and longitudinally free pylon was governing for the dimensioning. The geometrical and material non-linear calculation was performed. The increase of the non-linear bending moment, compared to the linear one was 38%.

The pylon top with cable stays anchorages was analysed separately, utilising several plane and space models (Fig. 1.9). Stays introduce large horizontal forces into the pylon, which had to be transmitted using large reinforcement, both prestressed and non-prestressed, stretching in longitudinal and transverse direction.

The statical analysis of the PC structure on the west bank was made separately, strictly following the construction sequence. Creep and shrinkage was accounted for based on the age of a particular superstructure segment. Actions from the cable stayed superstructure were introduced in the form of concentrated loads. The scaffolding (moving gantry) and the formwork load, amounting to 2×650 kN were accounted for.

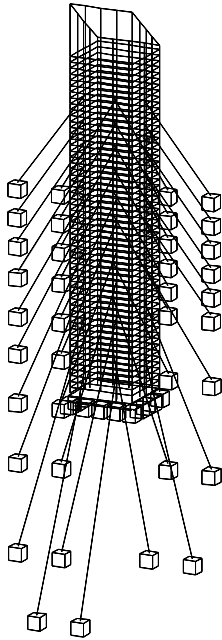


Figure 1.9: Spatial model of the pylon top, with cable stay anchorages

The calculation of the camber was performed, based on all construction stages to achieve desired geometry (grade-line). Altogether 120 construction phases were analysed, including concreting phases, stressing of the tendons, moving of the scaffolding, activating and removing of the temporary piers. All considered phases were included into the final construction protocol.

The dynamic seismic analysis was performed utilising spectrum analysis procedure, according to EC 8, for soil class B.

The cable stayed superstructure was analysed as limited ductile with behaviour factor $q=1.5$. This behaviour factor was utilised for the composite superstructure, the pylon and east bank abutment.

The seismic design of the PC beam bridge on the west coast was specially demanding. It was performed using equivalent linear dynamic analysis of the bridge. Obtained forces were modified by behaviour factors, according to EC 8/2. The reduction of the seismic forces, by the behaviour factor $q > 1.2$ could have been performed under the condition that the adequate measures are provided in order to avoid premature damage in the dissipative zone. The additional non-linear analysis have shown that the behaviour factor $q = 3.5$ may be applied. The most sensitive part is the dissipative zone at the pier bottom. Preliminary studies showed that sufficient ductility has to be achieved by introducing the plastic hinge at the base of the pier. The RWTH study concluded that

the designed horizontal displacements must be 1.43 times elastic movements ($q=1$). The real movements shall be smaller due to the introduction of the hydraulic dampers.

The foundation slab of the west pier was modelled utilising plate elements on elastic subsoil. The stiffness of the soil was modelled with tension soil stresses excluded.

1.4 Construction

The superstructure of the approach viaduct was executed by free cantilevering procedure symmetrically from the pier. Two auxiliary concrete columns were utilised to stabilise the construction in the 87.4 m span (Fig. 1.10).

The superstructure was prestressed in two phases. In the first phase, during cantilever construction, tendons in upper flange and in the webs were prestressed. After the completion of each segment at least two tendons ending at the segment end were prestressed. After the completion of the structure additional tendons in the lower flange were prestressed to cover for sagging bending moments (Fig.1.11).



Figure 1.10: Approach viaduct of the Rijeka Dubrovačka bridge in construction phase

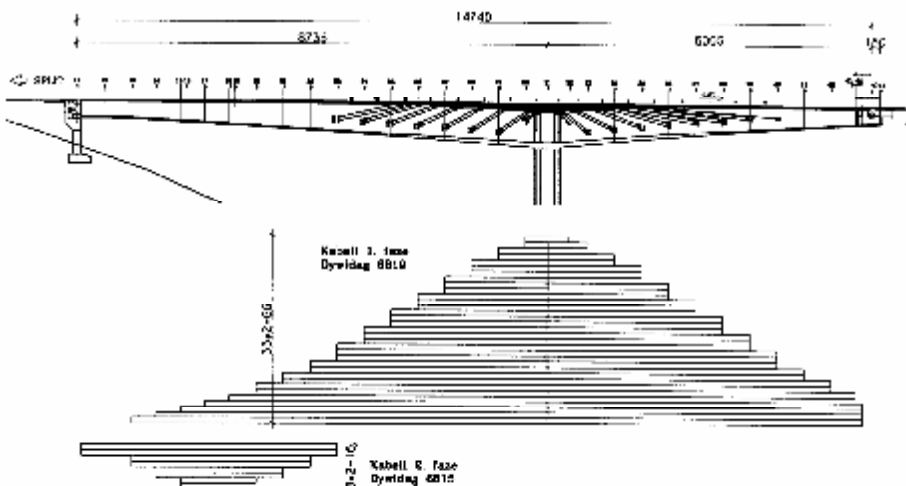


Figure 1.11: Tendon layout for the approach viaduct

Two different erection procedures were used for construction of the main bridge superstructure. The steel grillage of the 80,7 m side span and 33 m long adjoining part of the main span was erected by incremental launching from the left bank abutment (Fig. 1.12). The mass of the structure to be launched at the end of the procedure reached 508.2 tonnes. The erection procedure was completed in 4 months. The combination of two procedures was chosen in order to avoid auxiliary piers in the side span over inhabited area.



Figure 1.12: Launching of the steel grillage from east-end abutment

The remaining 211m long steel grillage of the main span was erected in 20 m long segments by free cantilevering. The derrick-crane of 90 tons capacity was used to lift the segments from the barge floating on the sea underneath (Fig. 1.13). Concreting of the deck plate followed two segments behind current derrick-crane position.



Figure 1.13: Lifting of one steel segment

The final adjustment of the cable-stayed, superstructure geometry was successfully performed after placing of furnishings by appropriate additional tensioning of the cable-stays, based exactly on the results of numerical analysis.



Figure 1.14: The last segment of the steel structure was installed from the approach viaduct, using mobile crane

| Structural part | Quantity | Quality of the material |
|------------------|---|-------------------------|
| Pier | Concrete: 614 m ³ | C 45 |
| | Reinforcement: 136 tones | St 500 |
| Pylon | Concrete: 3554 m ³ | C 45 (C55 – pylon top) |
| | Reinforcement: 1048 tones | St 500 |
| | Tendons: 11 tones | St 1570/1770 |
| Prestressed beam | Concrete: 1,08 m ³ /m ² | C 45 |
| | Reinforcement: 230 kg/m ² | St 500 |
| | Tendons: 60 kg/m ² | St 1570/1770 |
| Composite beam | Concrete: 0.3 m ³ /m ² | C 55 |
| | Reinforcement: 129 kg/m ² | St 500 |
| | Steel: 310 kg/m ² | St 52-3 |
| Stays | 309 tones | St 1570/1770 |

Table 1.1. Structural material quantities.

Test loading was performed, proving the accuracy of the design and statical analysis. The largest measured deflections in the main span were 289 mm, compared to the calculated value of 298 mm.

The minimum attained concrete grade for all structural parts was C 45/55, basically due to the very low prescribed water to cement ratio of 0.35.

Basic data:

| | |
|-------------------------|--------------------------|
| Total bridge length: | 518.23 m |
| Bridge width: | variable, 12.6 - 16.25 m |
| Pylon height: | 141.5 m |
| Pylon height/main span: | 0.33 |
| Prestressed beam: | 147.4 m |
| Cable-stayed bridge: | 324.7 m |

The bridge across the Rijeka Dubrovacka was opened for traffic in May 2002

MAIN PARTIES IN DESIGN AND CONSTRUCTION

| | |
|---------------------------|---|
| Client: | Croatian Roads, Zagreb |
| Main design: | Structural Department, Civil Engineering Faculty, Zagreb |
| Detailed concrete design: | Structural Department, Civil Engineering Faculty, Zagreb |
| Erection design: | Structural Department, Civil Engineering Faculty, Zagreb |
| Steel workshop drawings: | CIMOLAI, Pordenone, Italy |
| Main Contractor: | Joint Venture Walter-Bau AG - Konstruktor Inženjering, Split |
| Steel Subcontractor: | CIMOLAI, Pordenone, Italy |

2 MIRNA VIADUCT

2.1 Introduction

The Mirna viaduct lies on the west leg of the Istrian “Y” motorway on section Nova Vas – Višnjan, at the location where it crosses the Mirna valley and the Mirna River.

The layout of the viaduct was defined by constraints of its location in the Mirna River valley, particularly because the soil is alluvium of very low bearing capacity. It was necessary to reduce the total weight of the viaduct as much as possible because of extremely adverse soil conditions and to fulfill this requirement a relatively light steel superstructure was chosen. On the other hand, span lengths must not have been too small to reduce the number of costly foundations. The typical span length of 66.5 m was chosen.

The final bridge layout was obtained by taking account of all the constraints on the ground below the bridge, the Mirna River bed, existing irrigation channels, etc.

The motorway design for the viaduct motorway section was developed by the Engineering design office (IPZ) from Zagreb and comprised the full four lane motorway profile and all ground plan and vertical alignments were given for the motorway axis in the middle of median strip. The viaduct axis is 5.5 m away from the motorway axis and with ground plan alignment of the motorway in 4,000.0 m curvature, the ground plan alignment of the viaduct is in constant curvature of 4,005.5 m.

The vertical grade line alignment is in uniform concave curvature of 17,500.0 m, following the terrain profile to limit the height of viaduct columns. The overall viaduct length, measured in the viaduct axis, is 1,378.03 m with spans:

$$L=51.07+15 \times 66.491+70.096+2 \times 50.069+63.087+42.558+30,542=1354.86 \text{ m.}$$

Layout

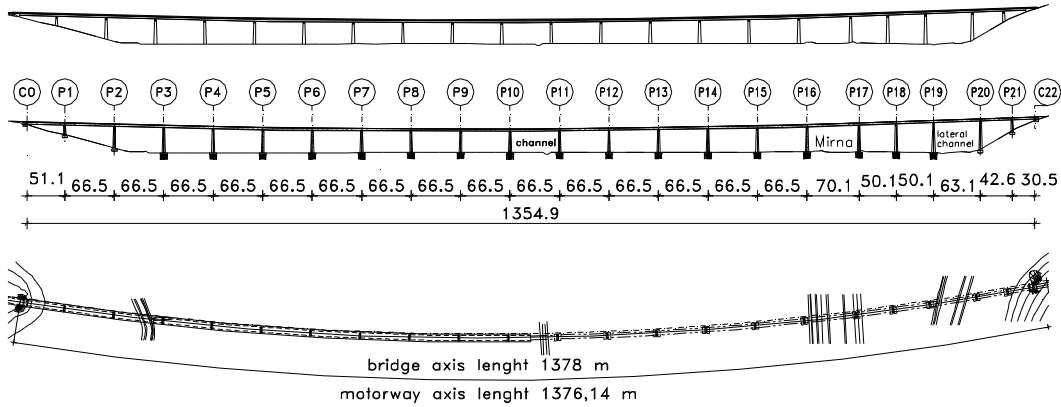


Figure 2.1: Longitudinal section and ground plan alignment of Mirna viaduct layout

The overall width of the viaduct deck without cornices is 10.10 m, the width between safety barriers is 9.10 m and the roadway width between curbs is 8.62 m, comprising two traffic lanes of 3.5 m each and safety strips.

2.2 Substructure

Both abutments are standard of closed-end type with short cantilevered parallel wings. Ample space is left between the abutment backwall and the superstructure making possible the installation of longitudinal viscous dampers to enhance the viaduct behavior under seismic loads. During construction abutments had to be redesigned to fit the requirements of expansion joints and viscous dampers actually installed, which were quite different from those defined in the final design. The constructed abutment C0 is shown in Figure 2.2.

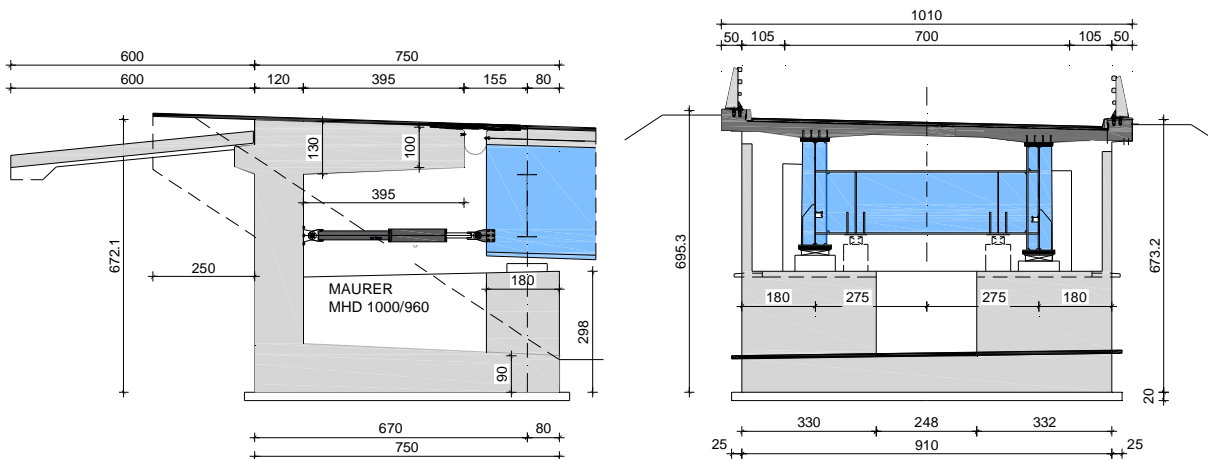


Figure 2.2: Abutment C0

All viaduct piers are very slender of H shaped cross section, comprising a web and two flanges extending in the direction of the bridge axis. The pier height, measured from the hammerhead top to the pile cap top, varies from the minimum value of 13.45 m for P1 pier to the maximum value of 40.026 m for the pier P19. Wall thicknesses of all piers are equal with webs 30.0 cm thick and flanges 35.0 cm thick. All piers are of constant width of 4.5 m perpendicular to the bridge axis extending along their height. In the bridge axis direction all piers are tapered, following the same functional relation, with the flange length of 2.5 m on top increasing towards the bottom at the slope of 40:1. The 7.5 m long hammerhead, 2.5 m wide and 1.5 m deep with recess, is provided on a pier top to support the superstructure. Structural bearings are placed on concrete blocks on tops of hammerheads.

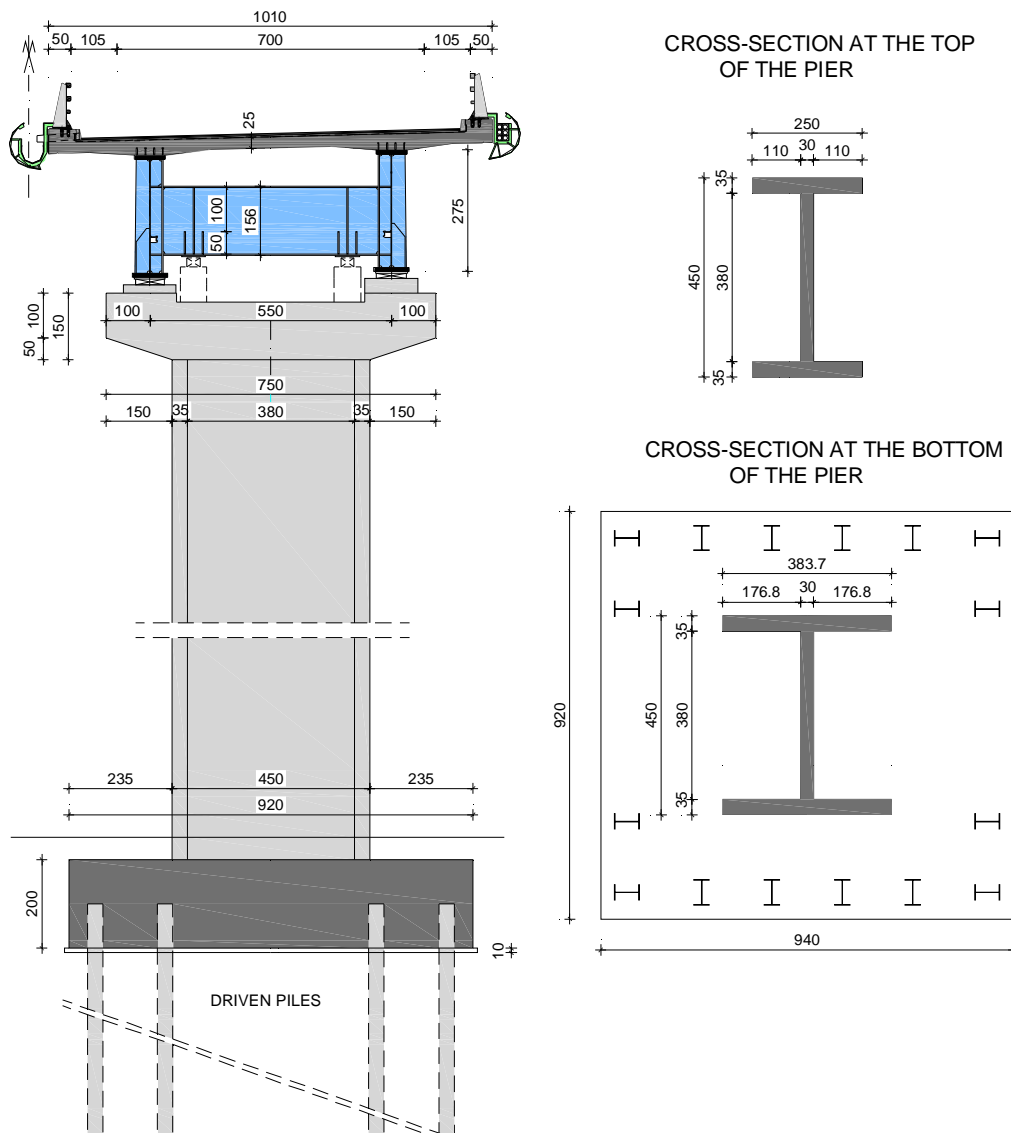


Figure 2.3: Typical pier with steel piles layout

Both abutments and piers P1÷P2 and P20÷P21 are founded on shallow spread footings on the solid rock of Mirna valley slopes. Piers P3÷P4 and P18÷P19 are founded on bearing steel driven piles HTM 600x149 penetrating the rock with their tips, while all other piers in the valley P5÷P17 are

founded on friction steel driven piles HTM 600x149 with the toe level at -65.0 m elevation. Steel piles penetrate concrete pile caps on the length of at least 1.0 m. Pile caps are 2.0 m deep and 9.4×9.2 m in ground plan, constructed with their tops at about -0.5 m elevation. A typical pier, including the layout of steel driven piles in ground plan, the pile cap and the hammerhead is shown in Figure 2.3.

2.3 Composite superstructure

The relatively slender superstructure consists of two main steel plate girders spaced at 5,500.0 mm, cross beams in frame action and of a concrete deck plate in composite action with main girders. The superstructure is continuous over 22 spans with overall length of 1,354.86 m. Main steel plate girders have uniform overall depth of 2,750.0 mm to facilitate erection by longitudinal launching. The top flange is of constant width of 700.0 mm and the bottom flange of uniform width of 800.0 mm along the whole viaduct length. The typical thickness of the top flange is 30.0 mm, used in all spans (except for the 70.0 m L17 span with 35.0 mm thickness), at both abutments C0 and C22 and at piers P18 and P21. At typical supports the top flange consists of two welded plates $700 \times 50 + 580 \times 48$ mm and at the most stressed pier P16 of two welded plates $700 \times 60 + 560 \times 50$ mm.

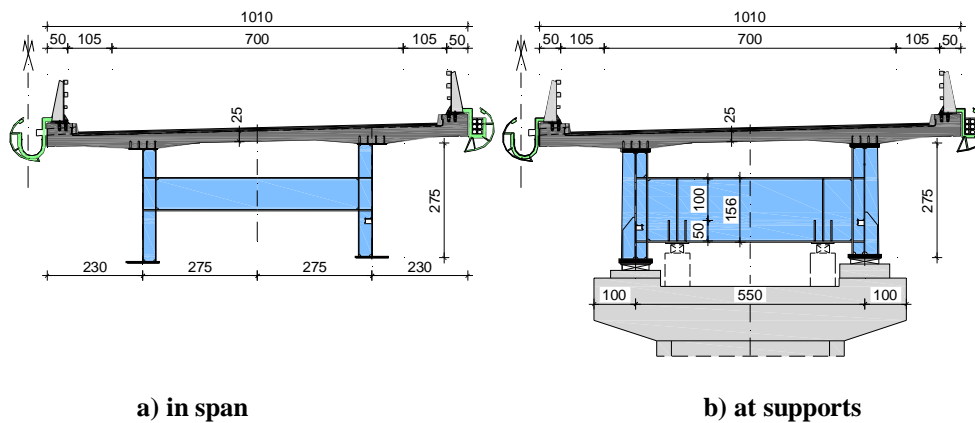


Figure 2.4: Superstructure cross section

The standard thickness of the bottom flange is 35.0 mm. At typical supports the bottom flange comprises two welded plates $800 \times 50 + 680 \times 48$ mm and at the most stressed pier P16 it comprises two welded plates $800 \times 60 + 660 \times 50$ mm. Webs are of uniform thickness of 16.0 mm along the whole viaduct length and of variable depth, depending on the flange thickness. Cross beams in spans are of welded plate girder type with flanges 300×20 mm and the web 760×12 mm. They are typically spaced at 7,540.0 mm in spans and at 4,800.0 mm near supports to fulfill their function of stabilizing bottom main girder flanges against lateral-torsional buckling in regions of hogging bending moments in combined action with vertical web stiffeners. Stiffer horizontal welded plate girder cross beams are provided at supports, with flanges 500×30 mm and the web $1,500.0 \times 25$ mm. These cross beams allow the positioning of jacks to lift the superstructure when necessary to

compensate for soil settlements with the re-adjustment of the bearings or to make possible replacement of bearings. They also transfer transverse horizontal forces to structural bearings in combined action with corresponding vertical web stiffeners. The reinforced concrete deck plate is of basic 25.0 cm depth with the depth increase to 35.0 cm at the connection to main steel plate girders top chords. Deck plate outstand tips are stiffened by edge beams 54.0 cm deep.

2.4 Structural bearing layout

A relatively unusual structural bearing layout was adopted, made possible by the low horizontal stiffness of friction steel pile foundations in adverse soil conditions. On piers P5÷P17 pairs of longitudinally fixed structural bearings were installed with one of them movable in the transverse direction and the other fixed. The fixity of both bearings on a particular pier in the bridge axis direction was necessary to limit torsional effects on torsionally weak piers of H shaped cross section. One of structural bearings on both abutments C0 and C22, and piers P1÷P4 and P18÷P21 is free sliding and the other longitudinally guided. The superstructure is longitudinally restrained at both abutments by two installed viscous dampers of 1,000.0 kN allowable capacity each. These devices allow slow longitudinal movements due to uniform temperature changes, creep and shrinkage, but lock in for sudden movements due to braking forces, longitudinal wind actions and seismic events. Structural pot type bearings, installed on the Mirna viaduct, were delivered by the Italian manufacturer Fip Industriale and the installed viscous dampers by the German producer Maurer&Söhne.

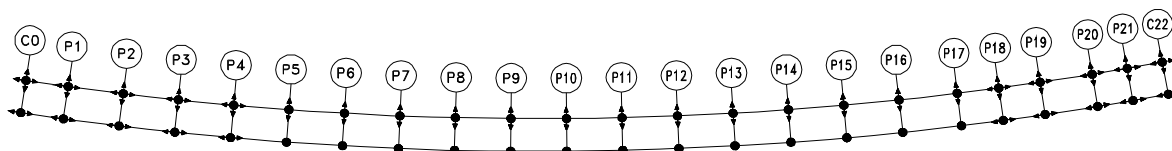


Figure 2.5: Structural bearing layout

2.5 Material grade and consumption

The material consumption for the Mirna viaduct without foundations is listed in Table 2.1, together with the grades of both steel and concrete material.

| Structural part | Concrete grade | Quantity (m ³) | | Nonprestressed reinforcement | Quantity | |
|---------------------|----------------|----------------------------|--------------------|------------------------------|-------------------------------|----------------------|
| | | total | per m ² | | Total (t) | (kg/m ²) |
| Substructure | B 35 | 6920 | 0,55 | BSt 500 S | 750,23 | 60 |
| Concrete deck plate | B 45 | 4271 | 0,34 | BSt 500 S | 1081,4 | 86 |
| | | | | | | |
| Superstructure | structural | Steel grade | Quantity (t) | | Quantity (kg/m ²) | |
| steel | | St 52-3 | 2 720, 9 | | 217 | |

Table 2.1: Material grade and consumption

2.6 Numerical calculations

Statical, dynamical analysis, stress verifications and required reinforcement determinations were all executed with the software package SOFISTIK. The loadings were based on the German bridge loading norm DIN 1072, except for and the seismic loading which was assessed on the basis of EC 8. Positions of the live loading producing maximum effect were determined on the basis of influence lines. Creep and shrinkage effects were computed utilizing DIN 4227. Loads were divided into basic loads (H) and additional loads (Z), according to utilized norms.

2.6.1 Superstructure

Calculations of the composite superstructure were executed for both the uncracked concrete deck plate along the whole bridge length and also under the assumption that concrete has cracked near supports on the length of 7.0 m, measured from pier axis and extending symmetrically to both adjoining spans.

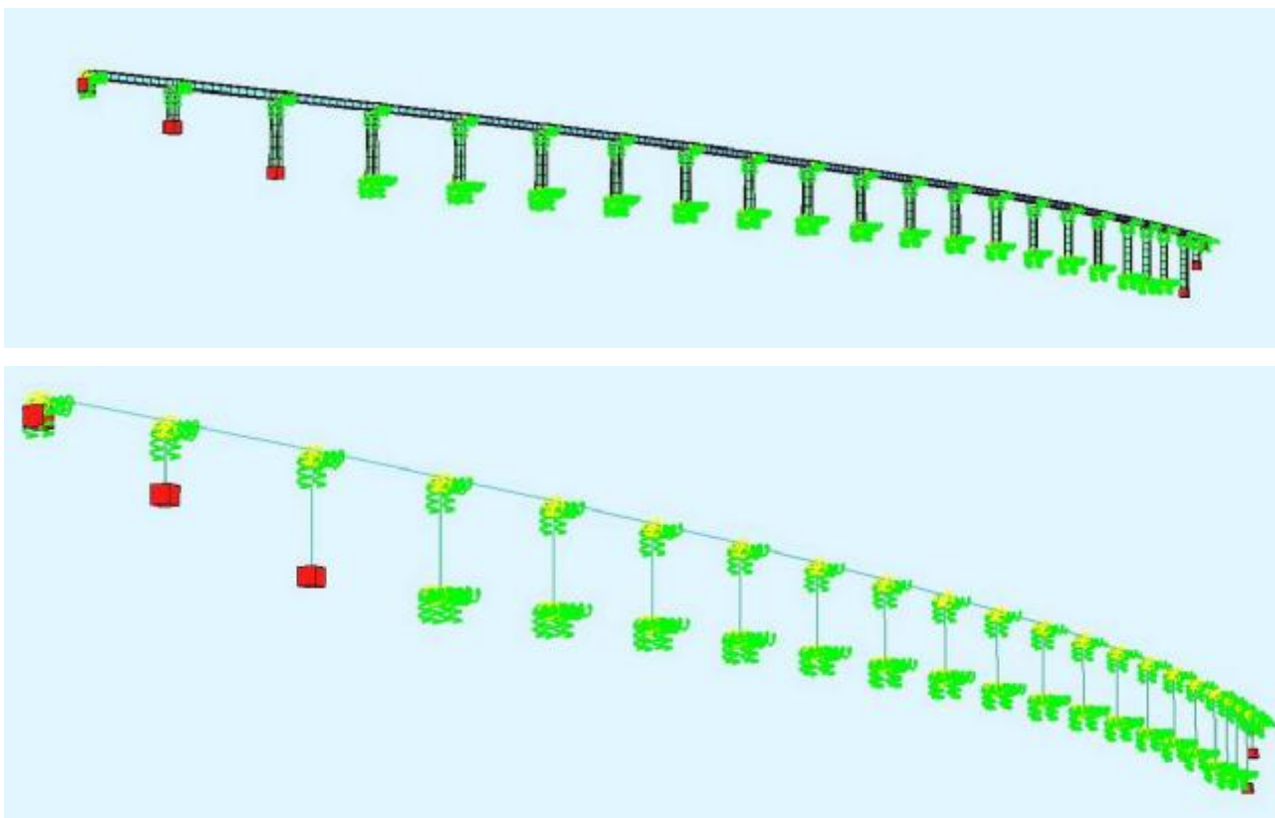


Figure 2.6: Numerical model

Maximum bending tensile and compression stresses in top and bottom main girder chords are summarized in Table 2.2 for both alternative analyses.

It is evident from Table 2 that the only considerable difference in bending stresses develops in the top chord in the hogging moment region above supports.

| | Loading combinations (H) | | Loading combinations (HZ) | |
|--------------------|--------------------------|-------------|---------------------------|-------------|
| | In span | at supports | In span | at supports |
| Top chord | | | | |
| Cracked concrete | -198,4 MPa | 211,5 MPa | -199,2 MPa | 220,9 MPa |
| Uncracked concrete | -194,5 MPa | 150,4 MPa | -195,5 MPa | 152,6 MPa |
| Bottom chord | | | | |
| Cracked concrete | 229,5 MPa | -200,5 MPa | 246,0 MPa | -212,7 MPa |
| Uncracked concrete | 222,4 | -198,0 MPa | 240,5 MPa | -206,6 MPa |

Table 2.2: Maximum bending stresses in top and bottom chords for cracked and uncracked concrete deck

Additionally the influence of sequence of concreting the deck plate on ensuing stresses and deflections was investigated. The comparison between stresses in main girder chords for deck concreting sequence accounted for and not taken into account is given in Table 2.3.

| | Top chord | | Bottom chord | |
|--|------------|-------------|--------------|-------------|
| | in span | at supports | In span | at supports |
| Deck concreting sequence accounted for | -185,9 MPa | 144,4 MPa | 114,3 MPa | -187,6 Mpa |
| Deck concreting sequence not accounted for | -192,4 MPa | 157,5 MPa | 113,2 MPa | -187,0 Mpa |

Table 2.3: Maximum bending stresses in top and bottom chords with deck concreting accounted for and not accounted for

2.6.2 Substructure

The stability verification of piers was executed by geometrical and material nonlinear analysis on deformed structure for critical loading combinations multiplied by the global safety factor of 1.75, according to DIN 1045. The maximum geometrical imperfection of 1/300 of the relevant buckling length was used, distributed along the pier height, following the shape of the fundamental mode of free vibration. The maximum obtained nonprestressed reinforcement percentages in relation to the relevant concrete area amounted to between 0.61%-1.65%. Anchoring of steel piles was executed by their penetration into the pile cap concrete on the length of at least 1.0 m, based on a proposal by the Contractor. The safe transfer of forces between a steel pile and pile cap concrete was provided for by placing shear connectors made of 32.0 mm reinforcing bars, as shown in Figure 2.7.

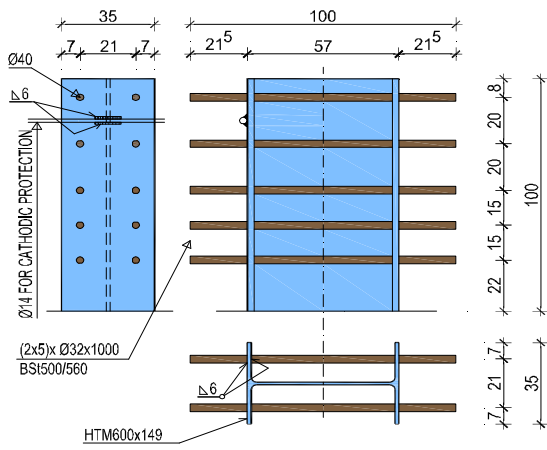


Figure 2.7: Anchoring of steel piles to concrete pile cap

Small pile cap depth and plan dimensions, in conjunction with unfavorable positions of concentrated forces acting on the pile cap, required heavy reinforcement in two layers in both directions. The 32.0 mm bar spacing was in the module of 15.0 cm. The transfer of forces to reinforcement bars occurs at pile cap edges or at the bar starting point. Because of that main reinforcement bars in the anchorage zones had to be spread and the actual behavior of the reinforced concrete plate was perfectly simulated by a space truss structural system, designed by Mr Mujkanović. The reinforcement was placed so that it encompassed steel piles and their anchorages. The nonprestressed reinforcement layout is depicted in Figure 2.8. An exact installation sequence had to be prescribed for such a complicated reinforcement layout.

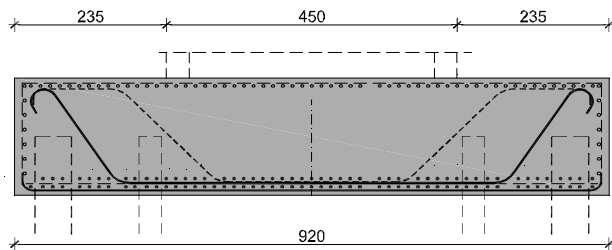


Figure 2.8: Nonprestressed reinforcement of pile cap

2.7 Construction

The main contractor was the French company BOUYGUES TRAVAUX PUBLICS and the subcontractor for the production, assembly and erection of the steel part of the superstructure was Italian company CIMOLAI. Shallow spread foundations on solid rock were executed in wide excavation in standard formwork. Steel piles were driven prior to soil excavations for pile caps. Piers were constructed in climbing formwork in 5.0 m long segments. Very high geometrical precision of these works was required, because of extreme slenderness of piers. Working joints had to be prepared and executed with utmost care. In the final step pier hammerheads were concreted.



Figure 2.9: Construction of foundations

The erection of the superstructure commenced with the assembly of steel members to segments, comprising main longitudinal girders, cross beams and temporary horizontal bracing on a plateau behind the abutments. Longitudinal launching of steel grillages, with most part of the concrete deck reinforcement already in position, was executed from both abutments.



Figure 2.10: Some construction phases of Mirna viaduct

After both launchings were completed and the steel grillage connected to continuous structure, there followed the lowering to permanent pot bearings under intensive surveying control to achieve the design geometry by final adjustments. Concreting of the deck plate was executed on temporary formwork, resting on main longitudinal girders, in a strictly prescribed order. There followed the installation of viscous dampers and expansion joints at both abutments and the viaduct construction was completed with the installation of furnishings.

MAIN PARTIES IN DESIGN AND CONSTRUCTION

| | |
|---------------------------|---|
| Client: | BINA – Istra, Pula |
| Main design: | Structural Dept. of Civil Engineering Faculty, Zagreb, |
| Foundation design: | Civil Engineering Institute of Croatia, Zagreb, |
| Detailed concrete design: | Civil Engineering Institute of Croatia, Zagreb, |
| Steel workshop drawings: | CIMOLAI, Pordenone, Italy |
| Main Contractor: | Bouygues Travaux Publics, Paris, France |
| Steel subcontractor: | CIMOLAI, Pordenone, Italy |

3 RECONSTRUCTION OF THE SAVA RIVER ROAD BRIDGE AT JASENOVAC

3.1 Introduction

The bridge crossing across the Sava and Una Rivers at Jasenovac, comprising seven independent bridge structures, provides the road link between Croatia and Bosnia and Herzegovina. The location plan of the bridge is depicted in Fig. 3.1 and the original main bridge across the Sava River is shown in Fig. 3.2. The bridge was opened for traffic in 1973.

During the Croatian war for independence several parts of the bridge were destroyed or damaged. The design and construction of the main Sava River bridge renewal is briefly reviewed in this paper.

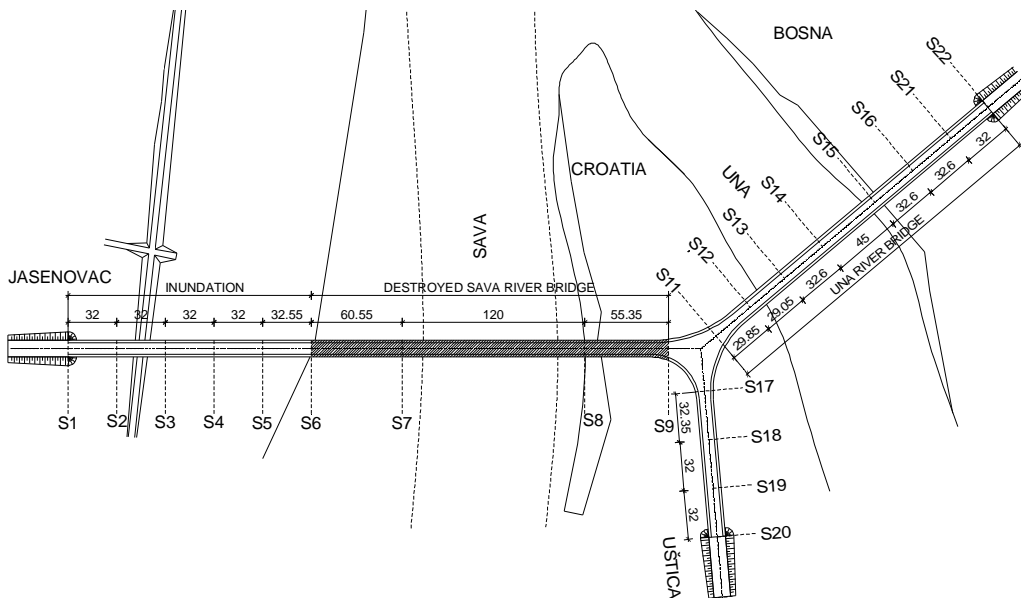


Figure 3.1: Location plan of the roadway crossing across Sava river at Jasenovac

3.2 Original main bridge across sava river

The original main bridge superstructure was a three-span prestressed concrete continuous beam, constructed by free cantilevering. The choice of the box type superstructure cross section and of the

side spans to main span ratios was stipulated by the employed construction procedure without auxiliary supports.

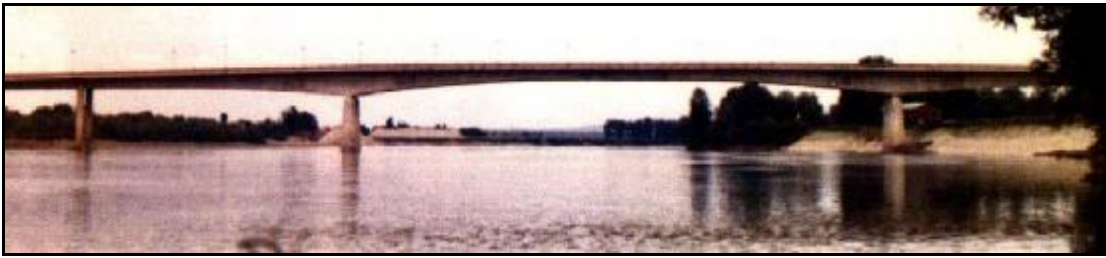


Figure 3.2: View of original prestressed concrete bridge across Sava River

Deep foundations were executed on reinforced concrete bored piles in mixed sand and gravel soil layers under the protection of steel sheet cofferdams. Middle river piers S7 i S8 were founded on eight bored piles, 1.5 m in diameter, 14,0 m to 16,0 m long. Tops of the piles were connected by 2.5 m deep pile caps. Tapered massive piers rested on pile caps. Transition two-column bents S6 and S9 at both ends of the main bridge rested directly on circular bored piles. Columns were of 1.50 m diameter and bored piles of 1.20 m diameter. The transition from 1.20 m to 1.50 m was executed at ground level, where a transverse pile cap was constructed. Tops of two columns of S6 bent were connected by a cap providing support to superstructures of both the approach bridge on the left bank of the river and of the main bridge. Superstructures were supported on the bent cap utilizing outstands of reduced depth, so that the overall structural depth of both the bent cap and superstructure outstands was equal to the the superstructure depth in span. The other main bridge side span superstructure and the intersection superstructure were supported on S9 bent cap in the same way. The cross section of the bridge was of box type with two vertical webs (Fig. 3.3b).

3.3 New bridge superstructure

The applicability of different materials, concrete and steel for the new bridge superstructure was addressed to. Alternative solutions comprised a continuous prestressed superstructure in accordance with the destroyed bridge and a continuous steel superstructure with orthotropic deck plate. It soon became obvious, that the the only possible solution is the steel superstructure, if existing foundations were not to be strenghtened, which was to be avoided because of economic and construction considerations, particularly because live loadings currently used are much larger than previously and the seismic loadings had not been accounted for in the original design at all.

New steel superstructure cross section is of box type with vertical webs (Figure 3a).The box width is 6.0 m. The deck surface and the vertical alignment are equal as in the original bridge, with slightly reduced depth at river piers from the original 5.50 m to 4.50 m. and the same depth of 2.50 m at midspan and bridge ends as in the original design. The orthotropic deck plate under the roadway is longitudinally stiffened by 275 mm deep closed 8.0 mm thick ribs, spaced at 600 mm

axis to axis and supporting the deck plate at 300 mm. The minimum deck plate thickness is 14 mm increasing to 16 mm along the length of 16.0 m in the intermediate support regions.

The deck plate under sidewalks is 12 mm thick stiffened by open ribs #200x15 mm. Webs are 12 mm thick in side spans on 20.0 m length and in the middle of the main span on 48.0 m length and 14 mm thick on 72.0 m length near intermediate supports. They are longitudinally stiffened by a system of angle profiles L200x100x10. The horizontal bottom plate thickness varies from 10-30 mm and the same longitudinal ribs as in the deck plate are utilized. In the sagging moment regions three ribs are placed and in the hogging moment regions seven ribs. Cross frames are typically provided at 4.0 m spacing. Cross bracing is provided at every second cross frame. Diaphragms are provided at all supports.

SAVA RIVER BRIDGE - LONGITUDINAL LAYOUT

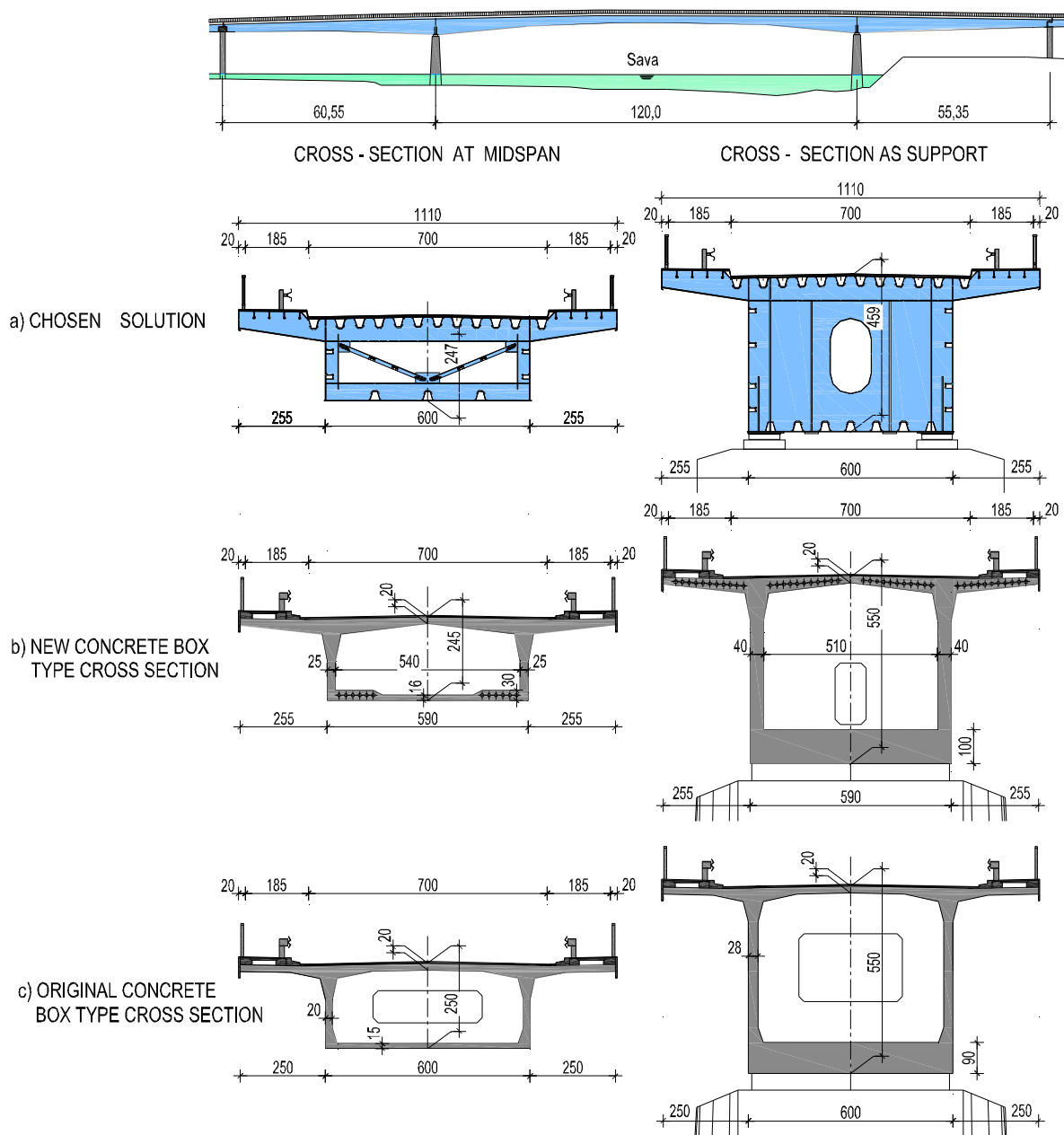


Figure 3.3: Alternative solutions for main bridge across Sava River

The existing span lengths of $L=60.55+120+55.35$ m are unfavorable for the continuous superstructure, resulting in uplift reactions on bridge end bents. This problem was partly remedied by lowering the superstructure at intermediate supports on solid piers S7 and S8 for 300.0 mm. This was not enough to completely eliminate uplift reactions with the required safety factor against uplifting of 1.30 and to attain this, parts of the steel box close to the end bents were filled with concrete. Close to the end bent S6 33.4 m³ of concrete had to be placed and close to the end bent S9 36.4 m³ of concrete.

Maximum deflection at midspan of the 120.0 m main span under design live loading amounts to 296.0 mm ($L/405$), and the maximum uplift in side spans is -84.0 mm, which clearly shows the significant flexibility of the superstructure.

5. NUMERICAL ANALYSIS

Numerical models for statical and dynamical analysis of the main load bearing system comprised the whole bridge structure, steel superstructure and concrete piers, utilizing SOFISTIK software.

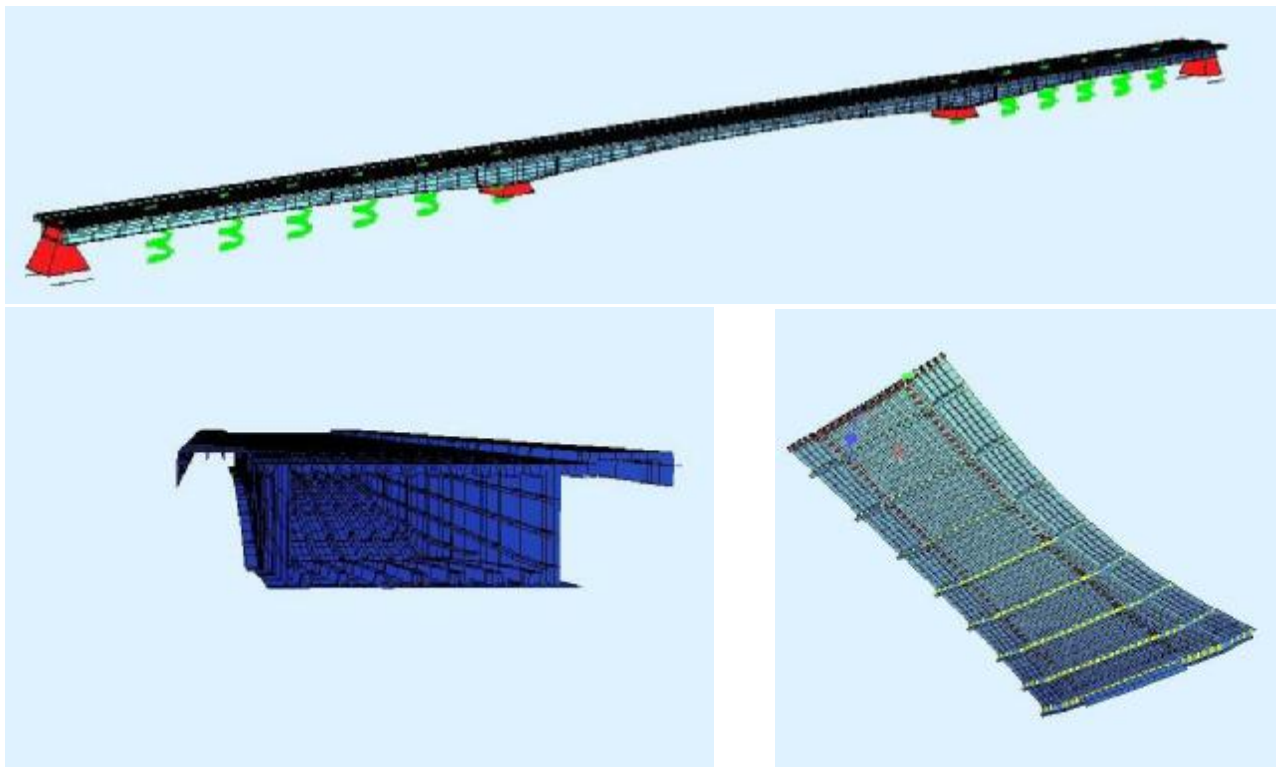


Figure 3.4: Numerical model

All loadings were based on the German bridge loading norm DIN 1072, except for the seismic loading which was assessed on the basis of EC 8.

Beam type elements were utilized in the general analysis, taking into account reduced flange widths due to shear-lag.

Local behavior of the orthotropic deck plate was modeled with plate type elements.

Detailed analysis of all construction phases of the superstructure erection procedure by free cantilevering was also performed, resulting in an erection strip, which was strictly adhered to during bridge construction.

All steel superstructure elements were designed according to allowable stress method, as required in relevant German bridge norms. Global and local buckling of steel members was verified according to the German code DAST Ri-012.

3.4 Erection of new main bridge superstructure

Two construction procedures were utilized for the bridge superstructure. Side spans between bent S6 and solid pier S7, and solid pier S8 and S9 bent, were erected on auxiliary scaffolding and the main 120.0 long span was constructed by free cantilevering symmetrically from piers S7 and S8. A derrick barge of forty tons capacity was utilized for local transport of steel segments. Steel sections were 12.0 m long. All on site steel connections were executed by welding, except for the webs, which were joined by high strength friction bolts. The total structural steel consumption for the superstructure amounted to 1,240.0 t of steel grade S 355, or 480 kg/m² of the bridge deck area between railings.



Figure 3.5: Free cantilever erection of new steel superstructure

3.5 Conclusion

The design of the reconstruction of the main bridge across the Sava River at Jasenovac was constrained by the fact that it had to be fitted between two existing bridge sections, the approach bridge on the left bank and the intersection on the right bank with predefined road alignment and deck widths. The chosen color for the superstructure is dark grey to accentuate steel, but not aggressively, to blend harmonically with other concrete bridge sections.

The reconstruction took place under difficult circumstances, mined surroundings, removal of the destroyed prestressed concrete bridge, damages to piers, damaged foundations, localized damage and worn-out structural parts and furnishings. With combined effort of all involved parties the bridge was successfully reconstructed in seventeen months from January 2004 to May 2005.



Figure 3.6: View of reconstructed main bridge across Sava River

MAIN PARTIES IN DESIGN AND CONSTRUCTION OF RECONSTRUCTED BRIDGE:

Client: Croatian Roads, Zagreb

Designer: Structural Department, Civil Engineering Faculty, Zagreb

Contractor: Đuro Đaković Montaža d.d., Slavonski Brod

4 KRKA RIVER ARCH BRIDGE

4.1 Introduction

The new Adriatic highway crosses the Krka River canyon in the proximity of the entrance to the environmentally very valuable and protected area of the Krka River national park. The highway alignment at the bridge lies in a straight line in ground plan and the vertical alignment is in a constant slope of 1.326%, lying approximately 66 m above the water level. The width of the river bed is 190 m, and the width of the canyon at the grade line elevation is about 390 m.

The overall width of the four lane roadway is 21.0 meters, including the median strip of 3.0 m and total width of the superstructure is 22.56 meters.

The designer tried to enrich the beautiful environment with the new bridge and the arch type structure seemed like a logical choice for the canyon. A concrete arch of 204.0 m span and 52.0 m rise was selected with the rise to span ratio of 0.25.

4.2 Bridge essentials

The arch is fixed of double cell box type cross section with constant outer dimensions $b/h=10.0 \times 3.0$ m. The arch cross-section is uniform along the length with 40.0 cm thick top and bottom chords, outer webs of 50.0 thickness and inner web 30.0 cm thick (Fig. 4.1).

Only near arch abutments, on the length of 10.0 m measured horizontally, chord thicknesses are enlarged to maximum of 60.0 cm. The arch is connected to arch abutments by a 3.0 m long

diaphragm. Six double spandrel columns are positioned symmetrically on the arch at respectively 14 m, 42 m and 74 m distances, measured from the arch crown. Diaphragms are placed inside the arch under spandrel columns. Their width is equal to the width of spandrel columns that they support. The arch and double columns at arch abutments are founded on joint foundations. The designed concrete class for the arch was C45/55.

The soil is limestone rock. Abutments and columns are founded on shallow spread foundations. Each double column has a joint foundation. Abutment foundations are of cascade type following the terrain slope. Arch foundations are massive with the cascade type bottom to reduce excavations and attain the minimum foundation depth of 2.0 m for the whole foundation. All foundations are of C25/30 concrete class.

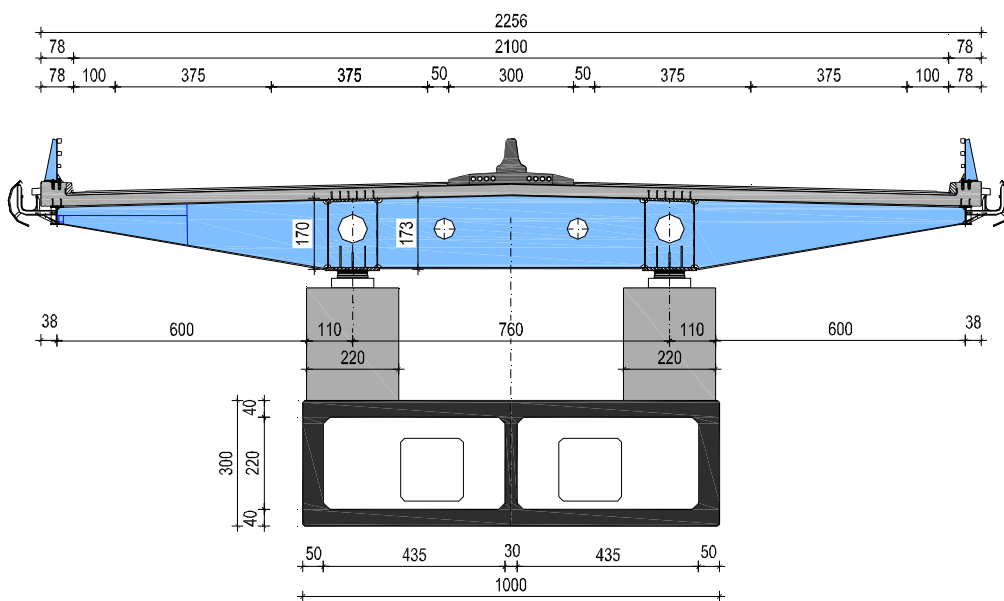


Figure 4.1: Arch and superstructure cross section of Krka River bridge

Abutments are standard of closed-end type with parallel wings of C30/37 concrete class and all columns are of box-type cross section made of C35/45 concrete class. Cross sections of particular columns were determined on the basis of stability calculations (Fig. 4.2).

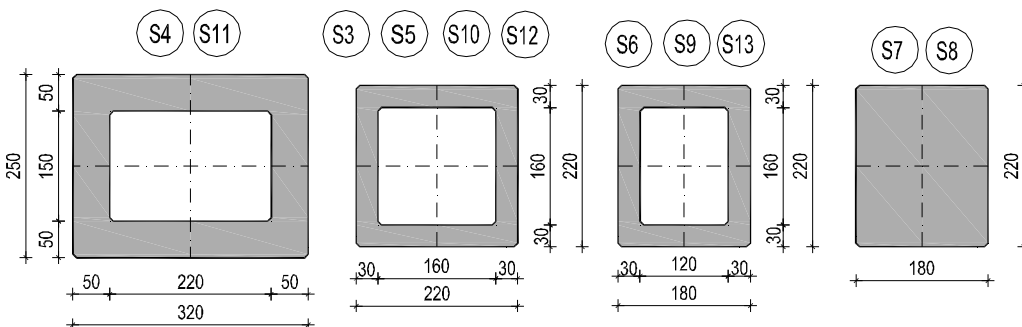


Figure 4.2: Column cross sections of Krka River bridge

Columns S3, S5, S10 and S12, with the height ranging from 27.64 m to 36.49 m, all have square outer dimensions 2.2x2.2 m and the walls 0.30 m thick. Spandrel columns on the arch S6 to S10

and the column S13, with the height range of 3.39-19.66 m, have outer cross sectional dimensions of 1.8x2.2 m and 0.30 m thick walls. Columns at arch abutments are 52.76 m and 55.57 m high, respectively. Their cross sections are rectangular 3.2x2.2 m with 0.50 m thick walls.

Instead of the usual prestressed concrete superstructure a composite superstructure was chosen to save weight thus reducing seismic forces and also to blend more harmonically into the beautiful environment. An outstanding aesthetical impression was achieved, because cross beams connecting column tops could had been avoided (Fig. 4.3).



Figure 4.3: View of Krka River bridge

The superstructure is continuous with spans $L=3 \times 32.0 + 3 \times 28.0 + 3 \times 32.0 + 28.0 + 24.0 = 360.0$ m and the overall bridge length amounts to $L=391.16$ m (Fig. 4.4). The composite cross section comprises a steel grillage and a concrete deck plate. The superstructure steel grillage consists of two main longitudinal beams spaced at 7.6 m, strong cross beams spaced at 4.0 m and edge girders.

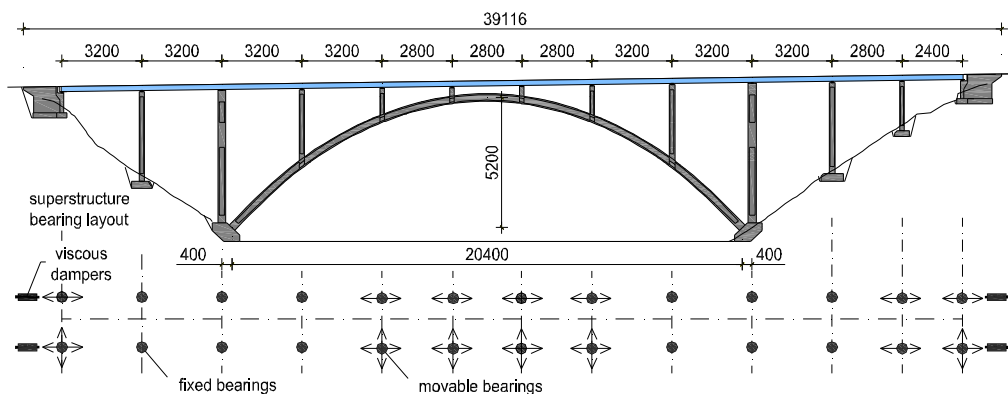


Figure 4.4: Longitudinal section and bearings layout of Krka River bridge

Box type main longitudinal beams have the inclined upper chord following the 2.5% transverse slope of the roadway, so that the total depth at the outer web is 1,700.0 mm and at the inner web 1,730.0 mm. These depths are uniform along the whole length of the bridge and all changes of chord thickness are made to the inside of the box. Outer web spacing amounts to 1,200.0 mm. The chord width of 1,300.0 mm is constant along the bridge length. The upper chord thickness varies

from minimum of 20 mm in spans to maximum of 70 mm in the vicinity of support at column S3 and the thickness close to all other supports is 55 mm. The lower chord thickness varies from minimum of 30 mm in spans to maximum of 80 mm close to support at column S3 and the thickness at all other supports amounts to 60 mm. Webs are 16 mm thick in spans and 20 mm thick close to the supports.

The top edge of cross beams is positioned at the same level as the top edge of main longitudinal beams and follows the transverse slope of the roadway.

The deck slab 22.56 m wide, made of C45/55 concrete class, has uniform 25.0 cm depth. Its main load-bearing direction is longitudinal.

The material consumption for the Krka River bridge, without abutments and foundations, is shown in Table 4.1.

| Concrete | Class | quantity | consumption |
|-------------------------|--------------|------------------------|-------------------------------------|
| Arch | C 45/55 | 2,988.0 m ³ | 0.40 m ³ /m ² |
| Columns | C 35/45 | 2,019.0 m ³ | 0.27 m ³ /m ² |
| Deck plate | C 45/55 | 2,172.0 m ³ | 0.29 m ³ /m ² |
| Reinforcement | | | |
| Arch | BSt 500 S | 747.0 t | 98 kg/m ² |
| Piers | | 363.4 t | 48 kg/m ² |
| Deck plate | | 630.0 t | 83 kg/m ² |
| Structural steel | | | |
| Superstructure | S 355 | 1,700.0 t | 224 kg/m ² |

Table 4.1: Material consumption of Krka River bridge

The choice of structural bearings non-movable in the direction of the bridge axis is conceptually interesting (Fig. 4). It seemed structurally logical to place them on shortest spandrel columns close to the arch crown, because the arch is much stiffer than any of the columns and hence its resistance to longitudinal horizontal forces is the largest. The problem arises for seismic events though, because it is almost impossible to design structural bearings able to take up practically the whole superstructure horizontal seismic load in the bridge axis direction and besides short columns supporting these bearings would be highly overstressed.

Therefore it was decided to install special longitudinally fixed structural bearings on tops of double spandrel columns S8. These structural bearings can sustain longitudinal horizontal forces for normal working conditions of up to 1,000.0 kN, but when these values are exceeded, in the case of large earthquake, they controllably transform to the longitudinally movable bearings by yielding of special sacrificial elements. Other structural bearings are of standard pot type, longitudinally movable bearings on stiff short piers and abutments, and longitudinally fixed bearings on tall

flexible columns S3÷S5 and S10÷S12. Thus the stiff structural system is achieved in the longitudinal direction for normal working conditions and at the same time adequate safety is attained for seismic events with the most of seismic loads taken by viscous dampers, provided at both abutments and just a fraction by tall flexible columns.

Because of the importance of the Krka River bridge and aggressive environment monitoring system was installed for measurements of deformations, local temperature values, movements in service at critical points and corrosion rate.

Decorative lighting was also installed, rendering beautiful view of the bridge in the night (Fig. 4.6).



Figure 4.5: Night view of Krka River bridge

4.3 Numerical analysis

Numerical models for static and dynamical analysis of the main load bearing system comprised the whole bridge structure, the arch, columns, structural bearings and composite grillage superstructure, utilizing SOFISTIK software. All loadings were based on the German bridge loading norm DIN 1072, except for the wind loading which was computed according to the British standard BS 5400 Part 2 and the seismic loading which was assessed on the basis of EC 8. Positions of the live loading producing maximum effect were determined on the basis of influence lines. Creep and shrinkage effects were computed utilizing DIN 4227. The creep coefficient for the arch, after completion of its construction, was calculated as $\varphi=1.876$ and the shrinkage coefficient as $\varepsilon=-20.2E-5$. Creep and shrinkage of the concrete deck plate was analyzed with the creep coefficient $\varphi=2.63$ and the shrinkage coefficient $\varepsilon=-24.8E-5$. Movements of structural bearings and expansion joints were calculated for uniform temperature changes +75 K/-50 K.

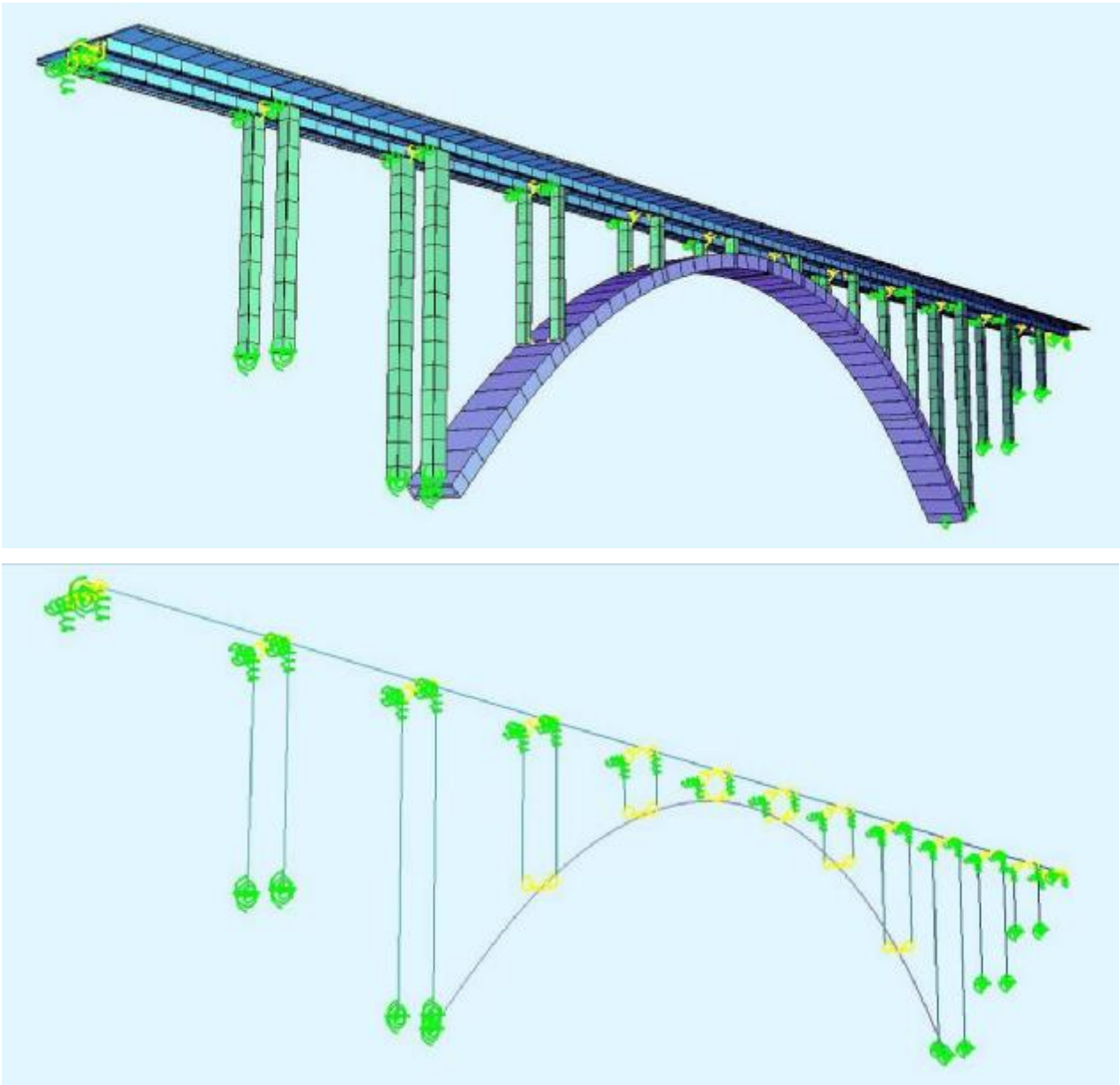


Figure 4.6: Numerical model

All critical loading combinations were additionally investigated by applying geometrical and material nonlinear numerical analysis. The arch stability was assessed by such an analysis on deformed arch shape, obtained on the basis of shapes of first three modes of free vibrations in the vertical plane normalized to the maximum initial eccentricity of $1/400$ of the buckling length and by applying the global safety factor of 1.75 to all loadings. Buckling stability of columns as cantilevers during erection was also verified by geometrical and material nonlinear analysis with initial eccentricity for relevant loads, dead weight and wind.

The composite superstructure was analyzed for the uncracked concrete deck plate and alternatively for the cracked concrete plate at all cross beams and at main beams near their supports on the length of $0.15xL$ in both neighboring spans. This implicitly means that the steel cross beams are sufficiently stiff to carry all loadings without the help of the concrete deck. Special FE models were introduced to assess the local behavior of the concrete deck plate and for the analysis of abutments.

Detailed analyses of all construction phases of the arch free cantilevering and longitudinal launching of the steel superstructure grillage were also performed, resulting in an erection strip, which was strictly adhered to during bridge construction.

All steel superstructure elements were designed according to allowable stress method, as required in relevant German bridge norms. Global and local buckling of steel members was verified according to the German code DAST Ri 012

Seismic design was based on the design response spectra for the design acceleration of 0.2g and the soil category A. The preliminary dynamic analysis by SOFISTIK software was executed under the assumption that the longitudinally fixed structural bearings on columns S8 have yielded and without dampers. Thirty eigenmodes were calculated and obtained eigenfrequencies ranged from 0.301 Hz for mode 1 to 4.224 Hz for mode 30. The total active mass was 22,910.0 tons. Seismic response was computed by modal superposition utilizing CQC method. Behavior factors were defined as $q=1.2$ for the arch, as $q=1.0$ for the superstructure and structural bearings and as $q=1.5$ for limited ductile columns. Under these assumptions the bridge turned to be very flexible in the direction of the bridge axis, which resulted in large longitudinal horizontal movements of 340.0 mm in case of a design seismic event. These large movements would have had an adverse effect on all piers with longitudinally fixed, structural bearings on top. For this reason viscous dampers were introduced at both ends of the superstructure to dampen the structure and transmit longitudinal horizontal seismic induced forces to massive abutments. Two dampers were installed at both abutments of 2,000.0 kN capacity each. The nonlinear dynamical analysis performed utilizing different software, with dampers included, revealed that longitudinal horizontal movements under seismic loading were reduced to 50.0 mm, which justified our design.

4.4 Bridge construction

The construction of the bridge across the Krka River canyon commenced in the year 2003 and was completed in two years in 2005.

The sequence of construction works was, as follows. In the first phase all foundations, including arch abutments, were executed. Columns on the river banks were constructed next, utilizing climbing formwork in 5.0 m long segments, followed by the construction of both bridge abutments. Then the whole cross section of the arch was constructed by free cantilevering, on travelling formwork carriages, in segments 5.25 meters long, starting symmetrically from arch abutments (Fig. 4.7). Piers at the arch abutments had to be extended by auxiliary steel staying pylons to facilitate successive cantilevering. The arch was supported during construction by stays equilibrated by anchor stays, connected to rock anchors. Temporary stays were of Dywidag system, consisting of naked 0.62" strands of St 1570/1770 steel grade, grouped at three levels.

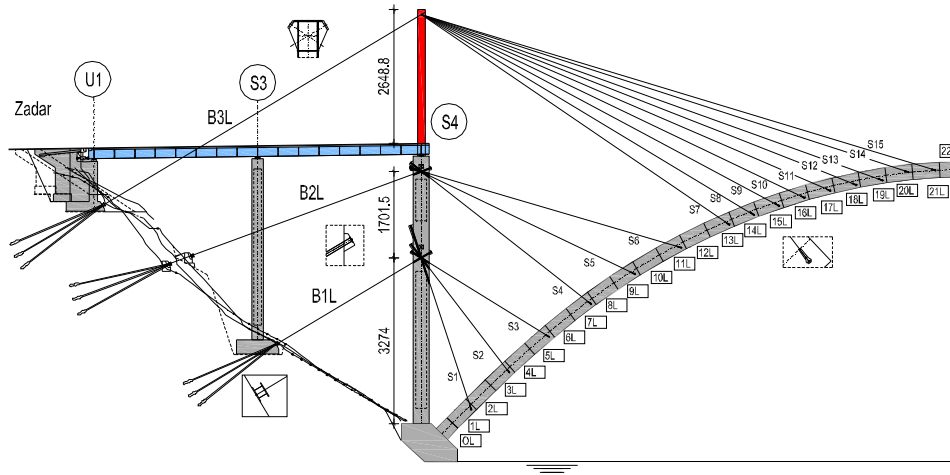


Figure 4.7: Arch construction scheme of Krka River bridge

The number of strands for a single stay varied from 12 to 22.

During the construction of first twelve arch segments, the steel superstructure grillage was assembled behind the abutments. Elements of the steel structure were transported to the site by truck and connected by welding to erection segments, comprising the total grillage cross section. Before the construction of the remaining arch segments the erection of the superstructure from both abutments to columns at arch springings had to be executed to facilitate the local transport required to service stressing activities of temporary stays anchored on steel pylons. This erection was done by the longitudinal launching procedure over special sliding devices placed on column tops with the help of special light steel nose to reduce stresses during construction. After the launching procedure, the steel grillage superstructure was lowered on permanent structural bearings. In the next construction stage, 25.0 cm thick precast reinforced concrete slab elements were placed, the middle one measuring 6.44x3.44 meters in plan between the main girders and the two side ones measuring 6.4x3.7 m in plan on cross beam outstands (Fig. 4.8).



Figure 4.8: Placing of middle precast slabs

Precast slabs were interconnected by on site concreting of longitudinal and transversal joints above shear connectors, thus completing the composite superstructure between both abutments and columns at arch springings. Special care was taken of proper preparation of precast slab surfaces to ensure adequate continuity by careful removal of laitance and exposure of the coarse aggregate utilizing wire brushing after initial set (Fig. 4.9).



Figure 4.9: On site joint preparation

The nonprestressed reinforcement connection at a typical joint is shown in Figure 4.10, while at joints above columns the nonprestressed reinforcement was connected by welding. Edge beam were concreted on site. After completion of the composite superstructure between both abutments and columns at arch springings, temporary steel pylons were erected on these columns to make possible anchoring of the third level group of stays and corresponding anchor stays in the cross beams connecting pylon tops to facilitate further arch construction.

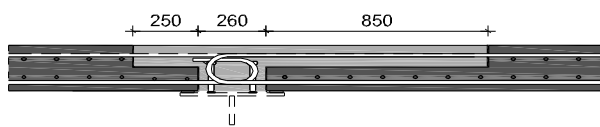


Figure 4.10: On site joints with reinforcement overlapping

Steel pylons were 26.25 m high, including the cross beams. Temporary pylons and the already constructed parts of the superstructure had to be fixed in the bridge axis direction during the subsequent arch erection, which was done by installing temporary steel restraining beams on the abutments. The last arch closure segment, 2.0 m long, was completed in April 2004 (Fig. 4.11).

The installed oscillatory cable-crane of 500 meters span and 6.0 tons capacity was utilized for local transport on the site.



Figure 4.11: Arch closure segment construction of Krka River bridge

After the completion of the arch, temporary stays and backstays were removed. Casting of spandrel columns on the arch in a strictly prescribed order followed. In the next construction phase the steel superstructure grillage was erected by longitudinal launching from the west bank and lowered down on permanent structural bearings. The composite superstructure above the arch was completed by placing precast concrete slabs and concreting wet joints, using the same procedure as before. In the final step bridge furnishings, including waterproofing, roadway pavement comprising two layers of hot asphalt, concrete median strip with installation pipes, safety barriers and lighting, were installed to complete the bridge construction.

The bridge was subjected to test loads, both statical and dynamical, prior to opening for traffic. The efficiency of the statical test load (ratio to design live load), performed in 33 different phases, was 78%-99%, depending on the investigated structural part. For some of the test loading schemes deflections were equal to numerically computed values and the maximum difference between measured and computed deflections was about 25%. Measured arch stresses were about 25% smaller than numerically computed values. This can be attributed to the fact that the actually installed nonprestressed reinforcement was not accounted for in numerical analysis. Measured bridge eigenfrequencies of vibrations in the vertical plane were almost equal to the theoretical values with the maximum difference of 5%.

4.5 Conclusion

The bridge was designed by a team from the Structural Department of Civil Engineering Faculty in Zagreb, Croatia. After completing the main design, construction plans were executed by the same team, together with the analysis of all construction phases. Designers were permanently in touch with the Contractor and every phase of the building process was monitored on site, thus providing necessary input for the adjustments of the structural analysis covering the bridge construction. This model of work proved to be very successful, resulting in high quality of works. The deviations of

bridge geometry from the designed one were reduced to the minimum, amounting to about 2.0 cm for the arch axis.

We believe that true concrete arch bridges may be competitive to other structural types in the span range from 200 to 400 meters for favorable site conditions, if the structural weight is minimized by applying a light composite superstructure reduced, as was done for the Krka River bridge.

MAIN PARTIES IN DESIGN AND CONSTRUCTION

Client: **Croatian Highways (HAC), Zagreb**
Preliminary design, Main design,
Detailed design, Construction phases,
Construction drawings: **Structural Department, Civil Engineering Faculty, Zagreb**
Contractor: **Konstruktor-Inženjering, Split**
Steelwork subcontractor: **Đuro Đaković Montaža d.d., Slavonski Brod**

5 SUPPLEMENTARY PROGRAMS

In order to speed up the numerical analysis of these bridges by Sofistik a few supplementary programs have been written.

One of these programs called Ana (Fig.5.1), written in 2001, generates Sofistik input data for free cantilevered bridges for selected longitudinal bridge layout and prescribed variables of the bridge cross-section. Automatic generation comprises cross-sections, numerical model, loadings and prestressing tendons. If one of the variables is left out the program itself generates it based on empirical data.

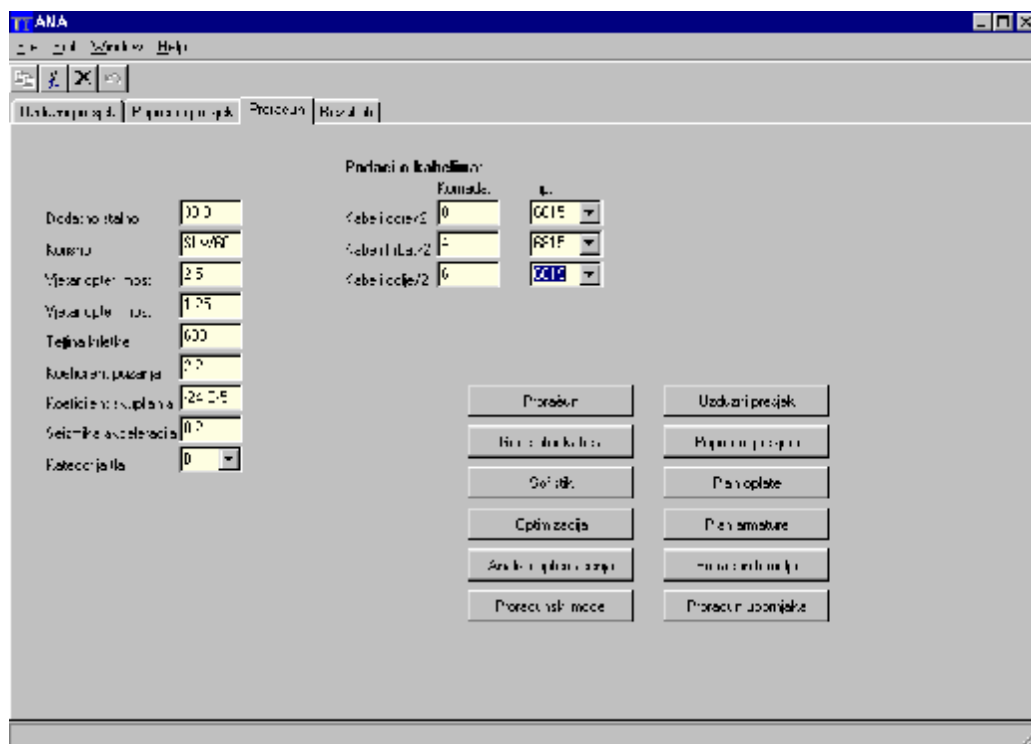


Figure 5.1: Supplementary program Ana

Based on obtained results of statical and dynamical analyses for all loadings (dead weight, taking into account construction phases, additional dead loading, prestressing, concrete creep and shrinkage, live loading, wind, temperature and seismic loadings) stresses are checked and necessary nonprestressed reinforcement calculated. In the first step the number and the stressing force in the tendons are assumed.

On the basis of the generated data basis the program itself changes certain variables (box depth, bottom flange depth or number and stressing force of tendons) for optimization of cross-sections with all stress constraints satisfied. Results of these calculations are transferred to AutoCad and input data for Sofistik

Are changed accordingly. The procedure is iterative and the optimal solution is usually obtained in a couple of steps. The program also calls MS Word and automatically produces adopted loadings and other data necessary for the design in written form. Two main objectives are realized with the program, the speed and the accuracy of the performed analysis.

The program also contains „masks“ for the calculation of abutments and foundations.

6 CONCLUSION:

The Structural Department of the Civil Engineering Faculty in Zagreb has extensively utilized Sofistik software in the design of Croatian bridges. Only a few more important bridges have been included in this presentation.

In order to facilitate the utilization of older versions of Sofistik some supplementary programs have been written, very similar to new Sofistik moduls CSM and SSD.