Eurocode requirements on Adriatic arch bridges

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ABSTRACT: An important issue in developing maintenance strategy for Adriatic arch Bridges is determining reliability of the structure with respect to increase in loading as well as changes in the requirements of relevant codes and standards over the decades of bridge service. In this paper some relevant comparisons indicating the influence of Eurocode requirements on Adriatic arch bridges are shown.

1 INTRODUCTION

There are six major reinforced concrete arch bridges in Croatia located on Adriatic coastline, with spans ranging from 200 m to almost 400 m. Four arch bridges, the Šibenik Bridge, the Pag Bridge and the Krk Bridges (two arches) were built during the sixties and the seventies of the 20th century. They are usually referred to as the first generation of Croatian Adriatic arches. Two major bridge structures Maslenica and Skradin Bridges were constructed on Croatian motorways more recently, Maslenica Bridge in 1997, and Skradin Bridge in 2005.

Bridges built in Croatia after the World War II have been designed according to two essentially different loading schemes, first the so called PTP-5 scheme and after 1973 the DIN 1072 scheme, and by using both the working stress and the limit state design method. New Croatian standards have implemented European pre-standards (HRN ENV) with national specifications incorporated in National Application Documents as an innovative approach to safety, serviceability and durability of structures including the time variability of actions. All this together results in different reliability levels of older, of more recently constructed and of new bridges.

Adriatic arch bridges are exposed to the significant traffic loading during summer, they are located in regions of high seismicity, and they are exposed to the effects of sea salt and bora - wind which, in some specific locations of the coast, exceeds the maximum reference wind velocity of 35-40 m /s.

Over the years many deficiencies and rapid degradation were identified on older Adriatic bridges. The combination of aggressive exposure conditions, poor detailing, neglecting durability problems and construction errors resulted in serious deterioration of structural members, with reinforcement corrosion being a major issue (Radić et al. 2003). In addition to these, the importance of adequate and regular maintenance activities was completely underestimated. As a result, huge, complex and expensive repair works were needed.

To eliminate the errors of the past and ensure smooth service and efficient management of large Adriatic Bridges in the future, an extensive project to develop an appropriate maintenance strategy was started recently.

Due to ageing of structures, increase in loading and changes in requirements of new standards an important issue in developing proper maintenance strategy is to asses the bridge reliability. Reliability analysis of existing bridges will determine their safety margin, capacity to sustain present loads and ability to fulfill new requirements.

2 DESIGN, PERFORMANCE AND MAINTENANCE OF ADRIATIC ARCH BRIDGES

Adriatic arch bridges are very well known not only because of their large spans, but also due to introduction and subsequent improvements in construction of concrete arches using free cantilevering technique (Radić et al. 2004).

The most famous is the Krk I bridge, the reinforced-concrete arch with the span of 390 m built 25 years ago between the mainland and the Krk Island, which still hold the world-record among conventional reinforced concrete arches.

2.1 Adriatic arch bridges of first generation

The Šibenik Bridge spanning 246 m (Fig. 1 left), built in 1966 was the first in the world to be constructed entirely by the free cantilevering method. The Pag Bridge (Fig. 1 right) completed in 1968 with a span of 193 m is very similar in appearance and design. Three-cell box arches gradually increase depth from the springing towards the crown. They were constructed by suspended cantilever method with temporary stay cables and tie-backs anchored into the abutments of the superstructure at Šibenik Bridge and directly into the rock at Pag Bridge. The superstructure of both bridges is designed as a series of simply supported grillages consisting of four precast prestressed concrete girders joined by cast-in-place cross beams at supports and the thirds of span. Šibenik and Pag Bridge design was dominated by solving the construction methodology (very small concrete cover), which subsequently led to durability problems.

Repair works on the Pag Bridge started already after a decade of its service, but did not prove efficient in terms of stopping the corrosion process. Major reconstruction started in 1991 with the repair of the arch and was finally finished in 1999 when the original concrete superstructure was dismantled and replaced by a completely new structure in steel (Šavor et al.2001). Columns were repaired by encasing in steel and concrete. The rehabilitation design of Pag bridge has to reduce the permanent loads to allow for the increase in traffic loads. Since the arch axis is designed as a thrust line for certain permanent load, due to a change of superstructure dead weight distribution the numerical analysis showed high sensitivity of the relatively elastic arch. This is to be considered in a rehabilitation designs of other large arch bridges, specially Krk bridge.

Šibenik Bridge is less exposed to aggressive maritime environment and only minor rehabilitation work was performed so far. The bridge was thoroughly inspected last year, and the repair methodology is presently being discussed.

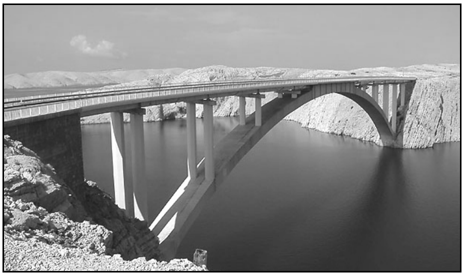

Figure 1: Šibenik bridge completed in 1966 and Pag bridge completed in 1968.

Figure 2: Krk bridges completed in 1980.

Krk Bridge (Fig. 2) consists of two large reinforced concrete three-cell box arches, of 390 m and 244 m span, constructed by an innovative procedure forming a trussed arch cantilevers. To achieve exceptionally large spans it was necessary to reduce the dead load as much as possible. The structural members of minimum statically admissible dimensions were utilized (very small concrete cover of 2.5 cm).

The repair works on the Krk Bridge started several years after its completion focusing on superstructure supports. In the 1990's the works were broadened to include the repair of columns of the smaller arch. Different repair techniques had to be devised for spandrel and approach columns. The smaller arch has been recently repaired by removal of the contaminated concrete, its subsequent replacement with shotcrete and adding protective coating. The repair of the larger arch presents major challenge to engineers. The arch is actually supported on submerged inclined struts. After many years of research and testing of various corrosion protection systems, cathodic protection method was selected for this part of the structure. All of these repair works are not only expensive, but technically demanding tasks and very difficult to perform.

2.2 New Adriatic arch bridges

The construction of the Maslenica highway bridge - a concrete arch of 200 meters span (Čandrlić et al. 1999), started during war and was completed in 1997 (Fig. 3 left). It symbolizes the continuation of the tradition of building large concrete arches in Croatia, such as famous Krk arches. The bridge design was strongly influenced by the severity of the marine environment and the seismicity of the site. In order to avoid reinforcement congestion and increase durability all structural dimensions were increased, compared to previously built concrete arch bridges in the Adriatic coast area. The low permeability concrete has been designed using Portland cement PC-30z-45s with 20 % slag addition which increased the bridge durability in marine exposures. The design concrete quality for the arch, piers and the deck slab was C-30, for precast prestressed girders C-40 and for foundations C-20 with water-cement ratio w/c less than 0.40. The minimum concrete cover for all the bridge structural elements was set at 5.0 cm and for the arch foundations nearest to the sea at 10.0 cm. The number of structural joints has been reduced to a minimum, with most of the piers fixed to the superstructure and the expansion joints placed at the abutments only.

A new concrete arch Skradin Bridge (Fig. 3 right) was constructed recently across the Krka River canyon (Šavor et al. 2005). Bridge spans 204 m with a rise of 52 m. It holds a unique position in the family of existing Croatian reinforced arch bridges because the bridge superstructure has been designed as a composite structure comprising steel girders and reinforces concrete deck plate, which resulted in substantial reduction of permanent actions. The arch itself is of considerably smaller dimensions than for the alternative solution with a prestressed concrete superstructure. The reduction of the total weight of the structure facilitated earthquake design as the bridge is located in the region of high seismicity. Steel corrosion protection has been adopted according to the latest standards for the most severe maritime environment.

New Adriatic arch bridges were equipped with a range of sensors for long-term control of stresses, strains and corrosion progress with the intention to closely monitor both structural performance and durability related performance in order to facilitate the future maintenance activities by triggering timely adjustments and interventions.

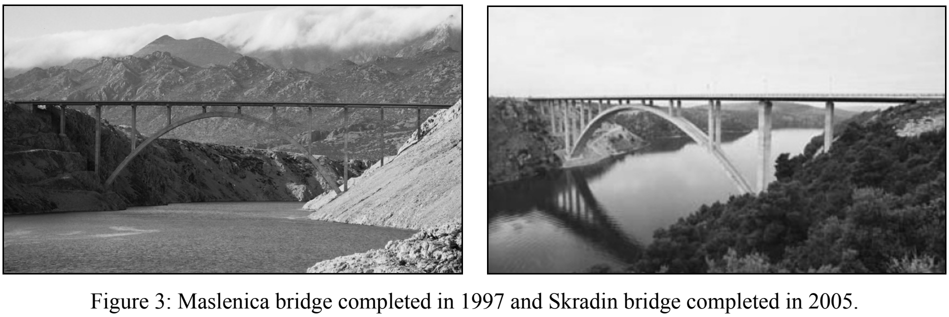

Figure 3: Maslenica bridge completed in 1997 and Skradin bridge completed in 2005.

3 EUROCODE REQUIREMENTS FOR RELIABILITY ANALYSIS

The analyses of these large arch bridges designed according different codes, thus with different reliability levels, in order to establish their limit states, will be a relevant issue of the presented maintenance strategy. The steps planned for this part of the research are as follows (Radić et al. 2006).

1. On the basis of data from bridge projects their designed carrying capacity and serviceability (R_d) will be stipulated. It will be necessary to amount old design concept to innovative approach.
2. Effective carrying capacity and serviceability of deteriorated concrete bridge members can be determinated by multiplying the design resistance Rd with the capacity reduction factor Φ using relevant parameters obtained from inspections of bridges (Znidarič et al. 1999).
3. Necessary carrying capacity and serviceability will be stipulated in accordance with realistic and standardized loading conditions. Traffic conditions and other relevant loads – wind, temperature changes, seismic action and their influence will be analyzed and compared.
4. To evaluate the reliability of existing bridges realistic (effective) carrying capacity and serviceability of existing bridges is to be compared with necessary carrying capacity and serviceability in accordance with realistic and standardized load conditions:

$$\Phi \times R_d \geq S_d = \left(\gamma_G \times G + \gamma_Q \times Q_{real}\right)$$

$$\Phi \times R_d \geq S_d = \left(\gamma_G \times G + \gamma_Q \times Q_{ENV}\right).$$

As a result the values of rating factor RF and reliability index β are to be obtained.

In this chapter some relevant comparisons indicating the influence of Eurocode requirements on Adriatic arch bridges are shown.

3.1 Traffic load

Old Croatian code PTP-5 employed on Sibenik, old Pag and Krk bridges predicts two motor vehicles with four axle load and uniformly distributed load p_e depending of the span with a dynamic factor k_d used for the main traffic lane. Recent Croatian code Pravilnik gives the load model composed of one heavy vehicle 600 kN for renewed Pag bridge and load model composed of two heavy vehicles 600 kN + 300 kN for Maslenica and Skradin bridges, together with uniformly distributed load. The dynamic factor k_d is also used just for one main traffic lane. HRN ENV 1991-3 predicts two lanes with the double-axle concentrated loads for Sibenik, Pag and Krk bridges and three lanes for Maslenica and Skradin bridges together with uniformly distributed load. The dynamic effects are already included into the load models. Altogether axle load in old PTP-5 consists of four-axle load (distances 4,5 m), in Pravilnik of three axle load (distances 1,5 m) and in new Croatian standard of double-axle load (distance 1,2 m). Uniformly distributed loads for the whole width of the bridge are very similar, but axle load are significantly larger with smaller axle load distances.

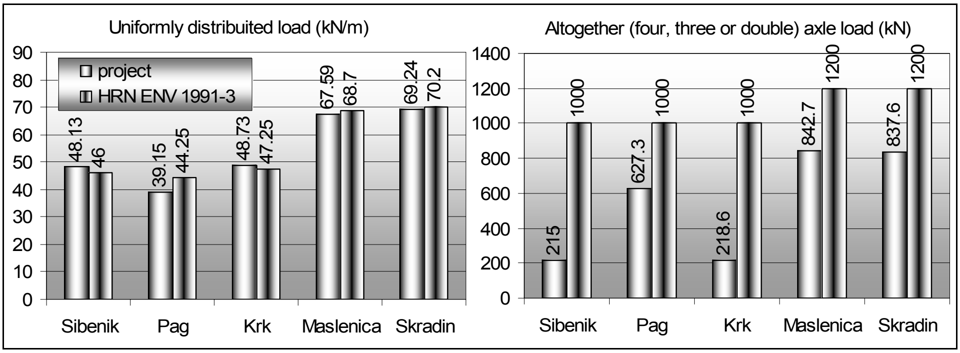

Figure 4: Comparisons of standardized traffic load models, uniformly distribuited load and axle load

Comparisons of traffic loading schemes used in bridge project design and schemes given in new Croatian standards based on european pre-standard HRV ENV 1991-3 are shown in Figure 5 for Pag bridge.

Flexural superstructure moments (Tab. 1) and arch inner forces (Tab. 2) are compared. Adoption of European models for traffic load results in increased serviceability limit state which is important for superstructure, not so much for the arch where dead load dominates. Old bridges need adjustments to come at serviceability level according new European standards. For ultimate limit state ratio of inner forces shows less differences (specialy for the arch were Eurocode can result in minor forces) due to minor partial factors for loads. Namely, design forces according new Croatian standard are compared with the design forces according old code using the following expressions:

$$M_{Sd}^{HRN-ENV} = 1{,}35 \cdot M_G + 1{,}35 \cdot M_Q$$

$$M_{Sd}^{PBAB} = \left(1{,}6 \cdot M_G + 1{,}8 \cdot M_Q\right)/1{,}15 .$$

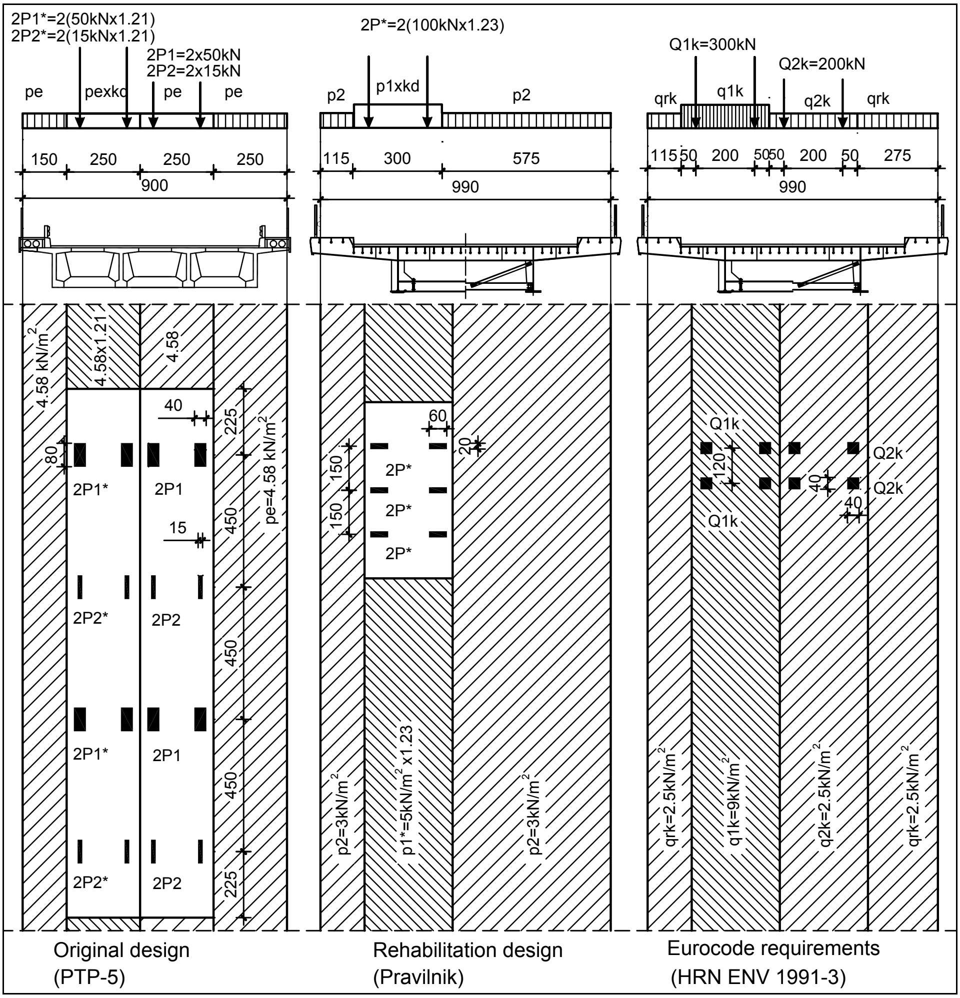

Figure 5: Comparisons of standardized traffic load models on Pag bridge.

Table 1 : Comparison of beam moments due to project and Eurocode requirements, Pag bridge.

	Maximal moment according project kNm	Maximal moment according Eurocode kNm	Ratio $M_{project}$ / $M_{Eurocode}$ /	Design moment according project kNm	Design moment according Eurocode kNm	Ratio $M_{Sd,project}$ / $M_{Sd,Eurocode}$ /
Superstructure (span)	7195	9295 8394*	**0.77** **0.86***	10861	12549 12246*	**0.87** **0.89***
Superstructure (bearing)	7881	9265 8734*	**0.85** **0.90***	11751	12508 12597*	**0.94** **0.93***

Table 2 : Comparison of arch inner forces due to project and Eurocode requirements, Pag bridge.

	Inner force according project kNm kN	Inner force according Eurocode kNm kN	Ratio project / Eurocode / /	Design inner force according project kNm kN	Design inner force according Eurocode kNm kN	Ratio project / Eurocode / /
Maximal moment	41586	43801 43444*	**0.95** **0.96***	59898	59132 60687*	**1.01** **0.99***
Belonging normal force	46416	47975 47632*	**0.97** **0.98***	65928	64766 65649*	**1.02** **1.00***
Belonging moment (abutment)	11135	13576 12875*	**0.82** **0.86***	15934	18328 18023*	**0.87** **0.89***
Maximal normal force (abutment)	50176	51800 51464*	**0.97** **0.98***	69682	69930 70977*	**1.00** **0.98***

European prestandard provides the possibility of applying a correction factor 0.8 for concentrated load in the first and second traffic lane as it is applied in the German DIN Fachbericht. In this code design value of traffic load is obtained by multiplying the characteristic load value with partial factor γ_Q = 1.5. Forces and their ratio due to this reduction are shown in the table marked with *. Differences are minor, specially for the serviceability level.

3.2 Wind action

Šibenik, old Pag and Krk bridge, located in the III zone of strong Bora wind, were designed according old code PTP-5 for the wind pressure of 1.3 kN/m^2 on the superstructure under traffic considering live load depth of 2.0 m. Without traffic they were designed for the wind pressure of 2.5 kN/m^2 on the superstructure.

In the Pag bridge renovation design more standards were considered (BS 5400, DIN 1072, SIA 1600) and at the end the wind pressure of 1,5 kN/m^2 on the superstructure under traffic and 3,0 kN/m^2 on the superstructure without traffic load were used.

Maslenica bridge was designed using German code DIN 1072, which specifies wind pressures of 2.5 kN/m^2 for bridges without live load, and 1.25 kN/m^2 for bridges with live load at the altitude between 50 and 100 m. Live load depth is here 3.5 m.

Wind loading on Skradin bridge was computed according to the British standards BS 5400 Part 2 which gives wind pressure of 1,16 kN/m^2 on the superstructure under traffic and 2,63 kN/m^2 on the superstructure without traffic load.

New Croatian standard based on European pre-standard HRN ENV 1991-2-4 provides guidance for assessment of wind loads for the structural design of highway bridges up to spans of 200 m. Wind forces on bridges may be determined by using the expression:

$$F_w = q_{ref} \cdot c_e(z_e) \cdot c_d \cdot c_f \cdot A_{ref}$$

where q_{ref} = reference wind velocity pressure depending on the reference wind velocity v_{ref}, $c_e(z_e)$ = exposure factor, c_d = dynamic factor of structural response, c_f = aerodynamic force coefficient, and A_{ref} = reference area.

Reference wind velocity map given in National Application Document refers to essential differences of inland and coastal parts of Croatia. In continental part reference wind velocity is at maximum values of 20 m/s, in coastal region values are between 15 and 25 m/s, and in some specific locations of the coast, wind Bora, with its turbulent action together with local terrain shape, exceeds the maximum reference wind velocity of 35-40 m/s. All Adriatic arch bridges in the coastal region are located in the zones of high reference wind velocities. For Sibenik bridge reference wind velocity for the bridge without traffic loading is 36.1 m/s, for Pag bridge 36.4 m/s, for Krk bridge 42.7 m/s for Maslenica bridge 54.5 m/s and for Skradin bridge 37.24 m/s. For the bridges under traffic wind velocity of 23 m/s is used.

Here (Fig 6) we present the comparison of the wind load pressures acting on the superstructure of Adriatic arch bridges, according to codes used in bridges' design and new Eurocode requirements. The influence of the much larger exposure factor than in the original ENV pre-standard, specified in the Croatian National Application Document, on design wind pressures is clearly visible.

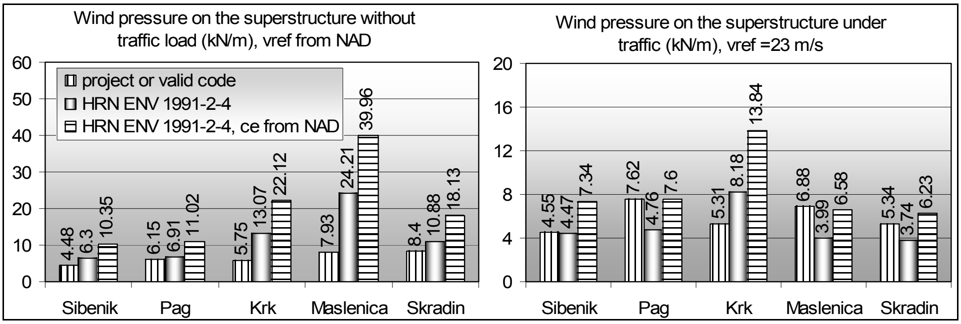

Figure 6: Wind pressure on the superstructures of large Adriatic bridges

Table 3 : Comparison of inner forces and stresses due to combination of dead and wind load according project and Eurocode requirements, Maslenica bridge.

	DIN 1072			HRN ENV 1991-2-4			Stress ratio DIN 1072 / HRN ENV
	Flexural moment	Normal force	Compression stress	Flexural moment	Normal force	Compression stress	
	M kNm	N kN	σ kN/cm^2	M kNm	N kN	σ kN/cm^2	/
Arch abutment	108360	133570	**10.9**	232240	133570	**14.6**	**0.75**
Portal pier bottom	6250	14909	**7.4**	13995	14909	**10.8**	**0.68**

As an example influences of Eurocode wind load requirements are compared with ones resulting from wind load used in design of Maslenica bridge (Tab. 3). Only transversal wind load, without traffic load is considered. According DIN 1072, used in bridge design, wind pressure acts on superstructure, piers and arch elements depending on the altitude. In the bottom zone with the altitude 0-20 m wind pressure is 1,75 kN/m^2, in the middle zone 2,1 kN/m^2 and in the top zone 2,5 kN/m^2 . According HRN ENV 1991-2-4 wind pressure acts on each element depending on its altitude and dimensions in accordance with the expression on the previous page.

Stresses in the arch abutment are 75 % of value resulting from Eurocode requirements. In the bottom of the portal pier Eurocode is more acquiring (ratio is 0.68).

3.3 Seismic design

Old codes for seismic design used in Croatia either do not consider dynamic properties of structure or they do not account for different ductility levels. Current seismic design procedures are based on ductile nonlinear behaviour of bridges under seismic action. Croatian standard based on European pre-standard HRN ENV 1998-2 assumes that bridge structure may dissipate seismic energy through formation of plastic hinges, normally in the piers. This allows the application of linear seismic analysis based on the design response spectrum.

Šibenik and Krk bridge were designed using PTP-12, in Pag bridge design it was recognized seismic design is not the magisterial one, Maslenica bridge was designed both using old Pravilnik and at that time developed Eurocode 8, Part 2, and Skradin bridge according Eurocode 8, Part 2. Here we compared some results of seismic design of Krk bridge with data given in original bridge design (actual built-in reinforcement, present design compressive strength. Reinforcement is not sufficient in the arch abutment and in the superstructure, above the bearings. Stresses in concrete are significantly enlarged.

Table 3 : Comparison of seismic design results and project design data on Krk bridge.

	Reinforcement				Compression stress			
	Arch L=244		Arch L=390		Arch L=244		Arch L=390	
	Design %	EC8 %	Design %	EC8 %	f_{cd} N/mm^2	EC8 N/mm^2	f_{cd} N/mm^2	EC8 N/mm^2
Arch abutment	1.5	**2.00**	1.66	**2.82**	22.67	**24.6**	25.5	**34.3**
1/4 L arch	1.20	1.20	1.19	1.19	22.67	16.0	25.5	18.6
Arch crown	1.17	1.17	1.16	1.16	22.67	19.9	25.5	21.0

4 CONCLUSIONS

An important issue in developing maintenance strategy for Adriatic arch Bridges is determining reliability of the structure with respect to increase in loading as well as changes in the requirements of relevant codes and standards over the decades of bridge service. The reliability analysis is necessary to determine the safety margin of existing bridge structures and their capacity to sustain present loads and fulfil all requirements of the new European standards. In this paper some relevant comparisons indicating the influence of Eurocode requirements on Adriatic arch bridges are shown. Additional measurement, inspections and analysis are carried out to prosecute with steps of the reliability analysis procedure. The final result of this investigation will be development of limit states estimation procedure of existing major bridges as a contribution to management of large and significant bridges.

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