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Seismic Assessment of Existing Bridges: Kukesi Bridge

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Abstract: In this presentation, we are showing the application of methods for assessing the structural capacity of existing reinforced concrete bridges, constructed before the application of European Norms. The safety check presented in the various guidelines includes a structural non-linear analysis, followed by checking especially the deformations and resistance of all piers. In most cases, the superstructure is not significantly involved in the seismic response of the structure, then continues with the controls of the substructure (piles and facades) and the foundations, as well as with the systems of the connections of the structure and those of the interconnection between the structural elements. The structural model reflects the real conditions of the structure, the structural model must contain all the relevant degrees of freedom, characterizing the dynamic response of the structure, the inertia characteristics i.e. the stiffness of the structure, and the superstructure connections. For the case of non-linear analysis, the structural model must be able to follow the development of stresses and deformations of the structure beyond the elastic phase caused by the formation of an increasing number of plastic zones.

Key words: Bridge capacity, push over, time history.

1. Introduction

The Kukes Bridge over the Black Drin is a bridge of great importance. The importance of this bridge has increased with the construction of the new road Durres-Morina and further in Kosovo, which has brought an immediate increase in traffic. Today it is the only one that connects the city of Kukes with the rest of Albania. The bridge is also of critical importance for international traffic, since the connection with Kosovo, Serbia, etc. is realized through it, alternative routes are few and in bad conditions.

The bridge was designed in 1974 (about 50 years ago), by the ISDA (Institute of Studies and Design No. 2 of Albania) at that time [1]. To date, the bridge has successfully withstood traffic and seismic loads, however, it has not been tested in extreme design situations such as cases of an earthquake with ground acceleration comparable to the design acceleration. The superstructure, columns and other component parts of the piles are designed with M-300 concrete and ST-3 grade steel with yield stress up to 240 MPa. The foundations are designed with M-170 concrete and ST-3 steel. ST-5 steel was used for the prefabricated beams, with a maximum yield stress of 280 MPa. It must be said that, based on the available documentation, rigorous quality control measures were taken during the construction of the bridge in order to respect the resistance characteristics of the materials. However, compared to today's conditions of technology development, steel ST-3 and ST-5 presents low resistance characteristics. For mild steels such as ST-3, according to the literature of the time, deformations can go up to 25% at the rupture stage, which is a satisfactory property in terms of ductility. However, it should be noted that there is a lack of accurate data on the plastic behavior of the steel. The laps of the steel bars are mainly made by welding, which calls into

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question the ductile capacities of the reinforced concrete sections. Care has been taken in positioning the laps away from areas with the potential to create plastic hinges.

The superstructure of the bridge is of the "continuous Gerber type beam" type. The main beams have strong brackets on which the secondary beams rest. The latter are "T" beams connected with diaphragms in the transverse direction. Fig. 1 shows a schematic cross-section of the secondary beams. The secondary part is supported on the consoles through movable and stationary hinges. The movable hinges are built with metal plates and discs and the free space for moving the parts relative to each other is 13.5 cm but that is limited by the dimensions of the beams to a joint of only 6 cm. The main beams of the superstructure (cantilever part above the support) are of variable height from 5 m near the pile supports to 2 m near the secondary beam supports. The cross-section is of the "box" type with several "chambers" as shown in Fig. 1 below. The substructure consists of three composite piles, two of which consist of grouping of four circular columns with longitudinal transverse connections, while another pile is of the ram type with two columns in the transverse direction of the bridge. The circular piles have a diameter of 240 cm and are reinforced with 66 bars with a diameter of 36 mm, except for the anchorage areas at the foundation (7 m) and the upper 14 m (near the superstructure), where the reinforcement consists of 88 bars with a diameter of 36 mm.

2. Theoretical Background

Based on the design philosophy of Euro norms, especially in our case EN-1998-1 2004, and EN-1998-2 2005 regarding the seismic resistance of bridges, it is important to say that they must meet the requirements for maintaining their function in case of emergency, with a necessary degree of reliability in function of its importance, after the seismic design event or action.

The structural model must reflect the real state of the structure, it must be able to contain all the relevant degrees of freedom, which characterize the dynamic response of the structure, the characteristics of inertia, i.e. the stiffness of the structure, and the connections of the superstructure. In the case of non-linear analysis, the structural model must be able to follow the development of stresses and deformations of the structure beyond the elastic phase caused by the formation of an increasing number of plastic zones. Plasticity can be considered concentrated at the edges of the elements in "plastic hinges", the behavior of which can be determined by a relation of the moment to the curvature (of the deformed element). Regarding the control against seismic actions, we checked the bearing capacity of the piles, using the "time history" method for two main purposes: first, to study the effects on the structure as a function of time; secondly, this type of analysis enables the study of the support detachment phenomenon, which is impossible to achieve with other analyses and is quite specific for the

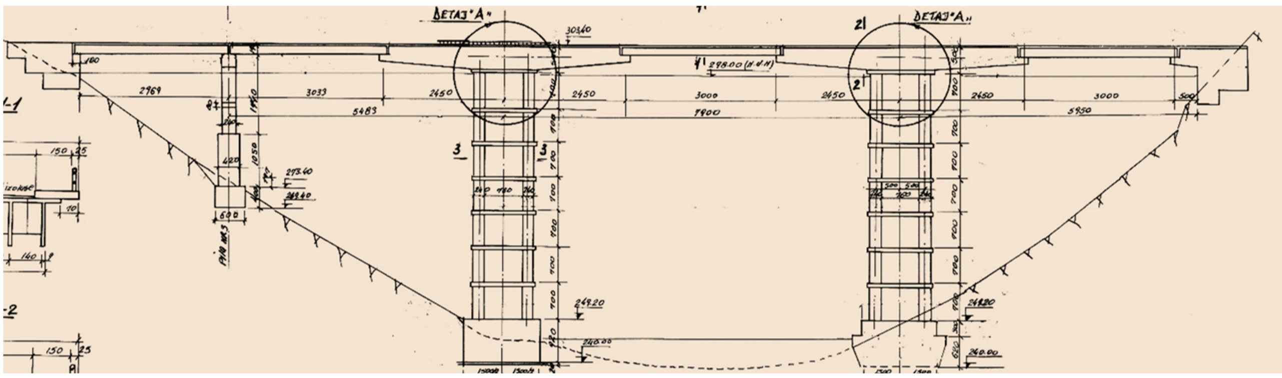


Fig. 1 General front view of the Kukesi Bridge.

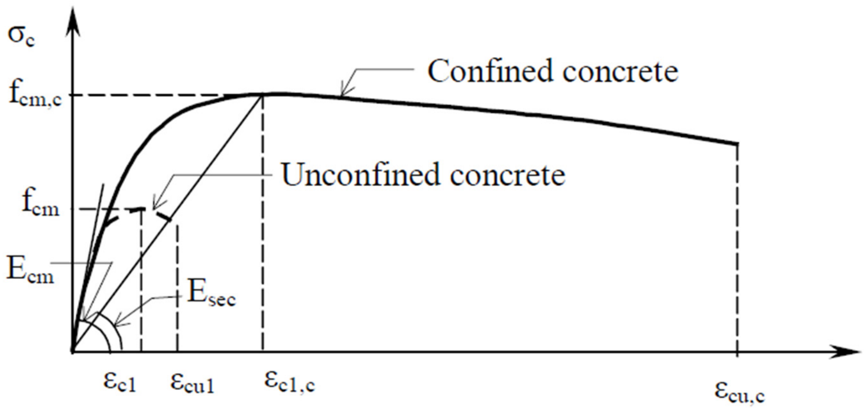


Fig. 2 Stress-strain relation for confined concrete.

bridge under study, as well as the "pushover" analysis method among the most efficient methods for evaluation of the capacity of a structure which consists of gradually increasing lateral forces up to a certain level (of forces or displacements) and recording certain displacements to be monitored. In this way, the base force-displacement curve is built, which gives a picture of the behavior of the structure in linear and non-linear phase. Properties of materials, for concrete is used stress-strain relation diagram as shown in Fig. 2. [2].

For reinforcement steel in the absence of relevant information on the specific steel for the project, following values are been used:

$$f_{ym}/f_{yk} = 1.15, \text{ and } \varepsilon_{su}/\varepsilon_{uk} \quad (1)$$

The stiffness of the elements for the case of nonlinear analysis with concentrated plasticity, in the model should be taken in relation to the level of cracks from the stresses caused by the seismic action [3]. The in-plane component elements (beams, cross-beams, slabs), which generally remain in the linear elastic phase with limited cracks, can be attributed to the characteristics of the entire section as a whole. For piles, which in most cases pass the "yield" limit state, the effective shear stiffness can be calculated with the expression:

$$E_c J_{eff} = \nu \frac{M_{Rd}(N_{Ed})}{\phi_Y} \quad (2)$$

where:

ν = 1.20 is a correction coefficient reflecting the stiffening effect of the uncracked parts of the pier.

J_{eff} is the effective moment of inertia, J_{eff} of a pier of constant cross-section may be estimated as follows:

$$J_{eff} = 0.08 \cdot J_{un} + J_{cr} \quad (3)$$

J_{un} is the moment of inertia of the cross-section of the uncracked pier; J_{cr} is the moment of inertia of the cracked section at the yield point of the tensile reinforcement. This may be estimated from the expression:

$$J_{cr} = M_y/(E_c \cdot \Phi_y) \quad (4)$$

M_y and Φ_y are the yield moment and curvature of the section respectively;

E_c is the elastic modulus of concrete.

$M_{Rd}(N_{Ed})$ is the ultimate limit moment of the design of the section on the basis of calculations for the value of the normal force from permanent loads.

Φ_y, curvature at yield, may be assessed as follows:

$$\Phi_y = (\varepsilon_{sy} - \varepsilon_{cy})/d_s \quad (5)$$

d_s is the depth of the section to the centre of the tensile reinforcement,

ε_{sy} is the yield strain of the reinforcement,

ε_{cy} is compressive strain of the concrete at the yield of the tensile reinforcement.

Strain ε_{cy} may be assessed by a section analysis on the basis of ε_{sy} and the actual force of the seismic combination N_{Ed}.

The values for the curvature of the yielding are as follows:

- for rectangular sections of $\Phi_y = 2.1\ \varepsilon_{sy}/d$

- and for circular sections of $\Phi_y = 2.4\ \varepsilon_{sy}/d$

where d is the effective depth of the section given in general satisfactory approximation.

For a plastic hinge occurring at the top or the bottom junction of a pier with the deck or the foundation body, with longitudinal reinforcement of characteristic yield stress f_{yk} (in MPa) and bar diameter d_s, the plastic hinge length L_p may be assumed as follows [4].

$$L_{py} = 0.10L + 0.015 f_{yk} \cdot d_s \qquad (6)$$

3. Analysis and Results

As mentioned above the bridge was designed in 1974 and constructed soon after so its working life is approximately 50 years old. Superstructure scheme is statically determinate beam, box type cantilevers and statically secondary precast T beams, frame type piers. Bridge is with height of 54 m, and total span 223 m.

Materials:

Superstructure and piers:

Concrete R_{ck} = 30 MPa;

Reinforcement ST-3 (f_{yk} < 240 MPa);

Foundations and abutments:

Concrete R_{ck} = 17 MPa;

Reinforcement ST-3 (f_{yk} < 240 MPa);

Precast beams:

Concrete R_{ck} = 30 MPa;

Reinforcement ST-5 (f_{yk} < 280 MPa);

Bridge substructure, Abutments has a "cantilevered" scheme, on which the secondary elements are supported.

Piers of the bridge are frame type, four circular columns each with D = 240 cm, and 7 stories.

The bridge has classic steel movable bearings and fix bearings.

Condition of the bridge up to now are: No mayor earthquake has hit the bridge; Successful history of resisting vertical loads and good condition of concrete (visual inspection).

The reasons for a detailed assessment of the bridge consist of: increased traffic loads; new code requirements (Eurocodes) for traffic and seismic design; increased importance of the bridge; possible degradation of materials; and new techniques are developed.

Fig. 3 Kukesi Bridge.

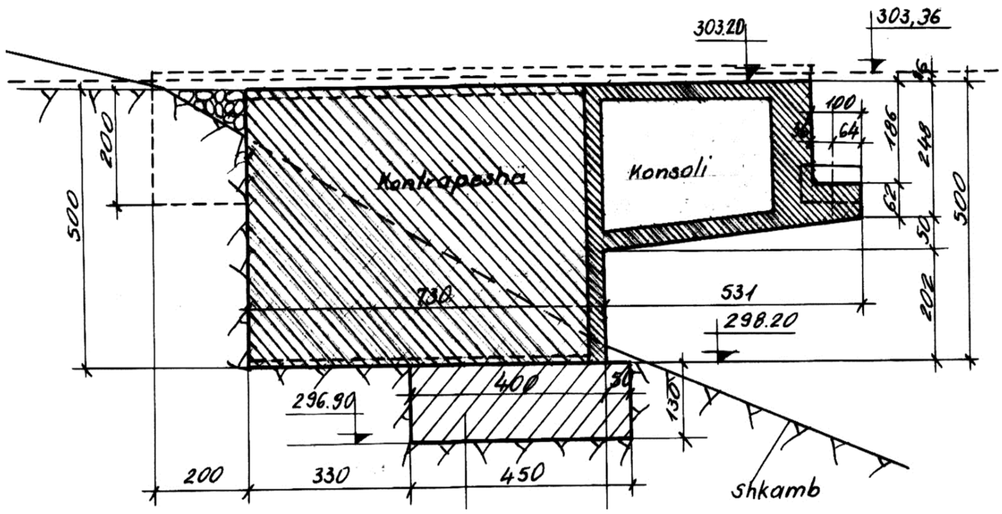


Fig. 4 Abutments design drawings.

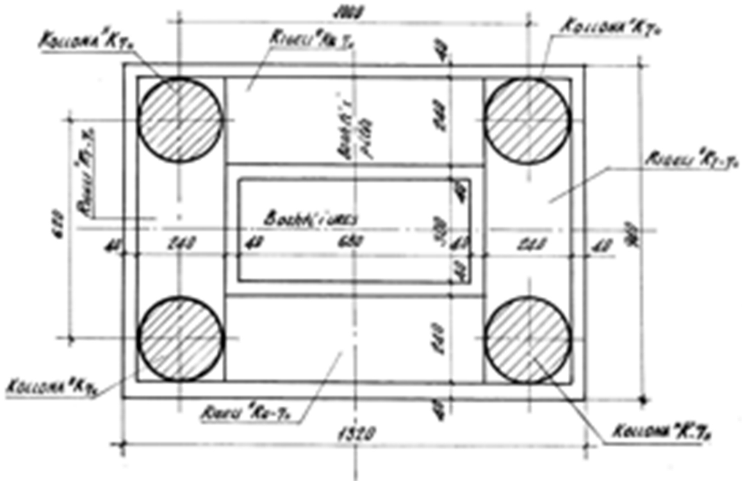


Fig. 5 Piers section.

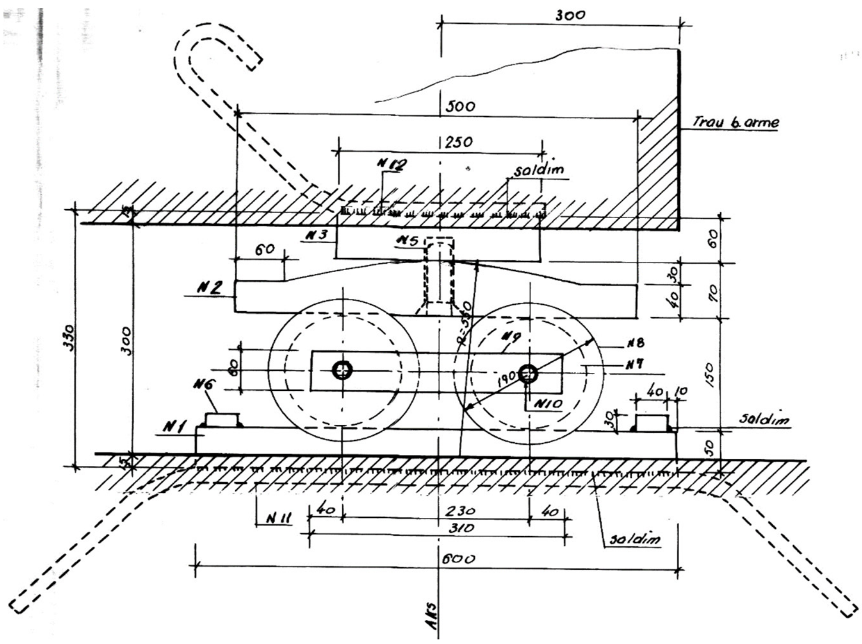


Fig. 6 Movable steel bearings

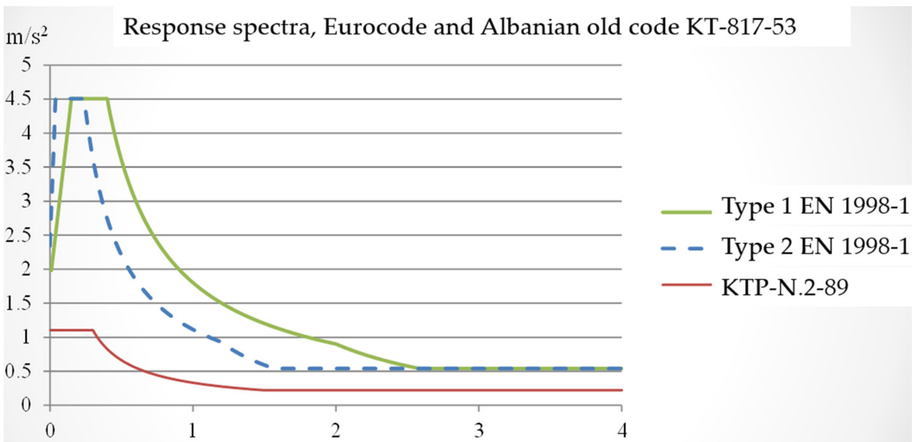


Fig. 7 Comparison of seismic demands, and EN-1988 vs. Albanian code KTP-N.2-89 [4].

3.1 Structural Modeling and Analysis

The bridge FE models constructed in CSI Sap2000 the models are based on available data collected at ISDA. Several other bridge models were constructed to facilitate calculations and presentation of results, mainly for nonlinear time history, nonlinear static pushover.

3.2 Time History Analysis

With time history analysis for realization of the above goals, 5 accelerograms were used. All accelerograms were scaled to the maximum ground acceleration level ag = 0.2756*g* (according to seismic risk data for Kukes). For clarity, the accelerograms are presented in separate in the Fig. 9, the Ulqini earthquake (year 1979) and Altadena. Dashed lines show the ag level of Kukës.

3.2.1 Displacements

The maximum displacements refer to the top of the pile cantilever and are summarized in Fig. 10, from the graph of displacements as a function of time in the longitudinal direction of the bridge, it is noted that the maximum displacement is Δx = 15.3 cm.

In the transverse direction of the bridge, the situation is somewhat different, due to the lack of obstacles to displacement in this direction.

From Fig. 10, where the displacements as a function of time in the transverse direction of the bridge are presented, it is noted that the maximum displacement in this direction is Δy = 18.85 cm.

In relation to the height of the piles, which are about 54.2 m, the maximum displacement lies in the ratio H/Δ = 5,420/18.85 = 287.

3.2.2 Impact at the Joints

The gap space of 6 cm between the secondary and primary elements. In case that the gap space is closed during the seismic action, additional forces arise which cause pressure on the secondary beams, on the other hand, if the gap becomes too large, there is a risk of the secondary beams breaking off or hitting on the other side. Fig. 12 shows the joint spacing between the main and secondary beams. Positive values indicate joint opening while negative values indicate joint closure.

In the case of El Centro and Ulqini (Fig. 12), the free space of the gap exceeds the 6 cm, thus making the parts of the bridge hit in the longitudinal direction.

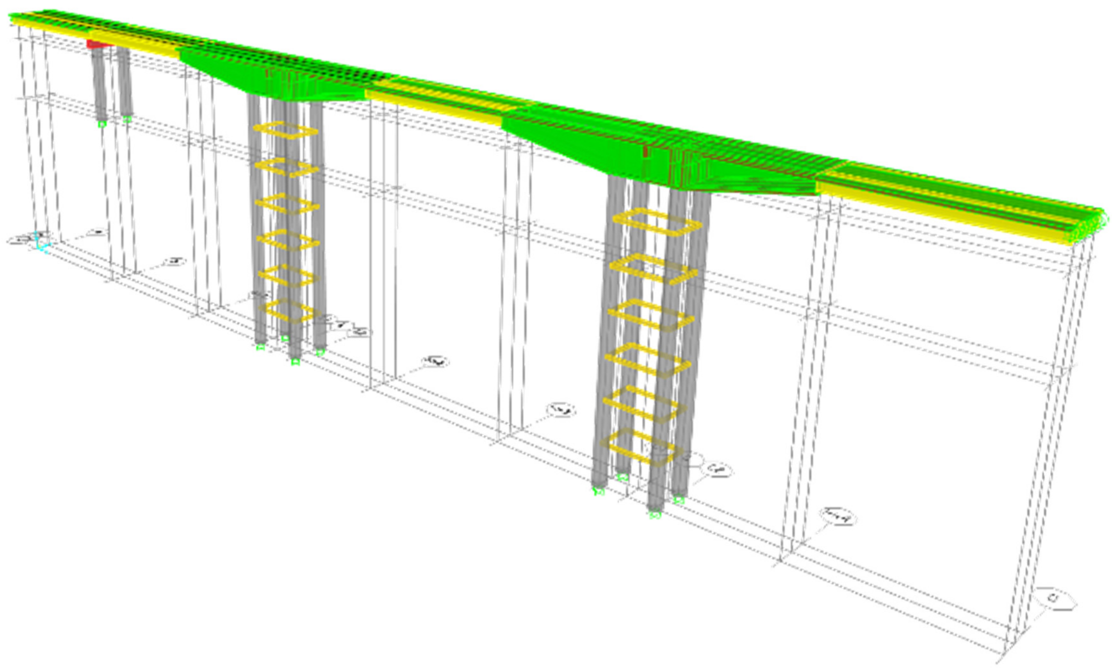
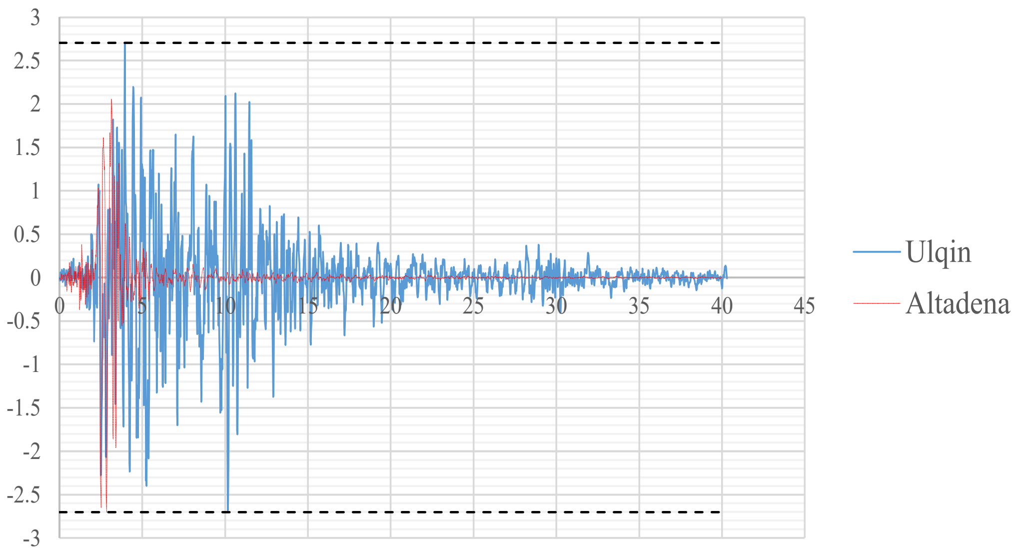


Fig. 8 3D view of the Kukes bridge model.



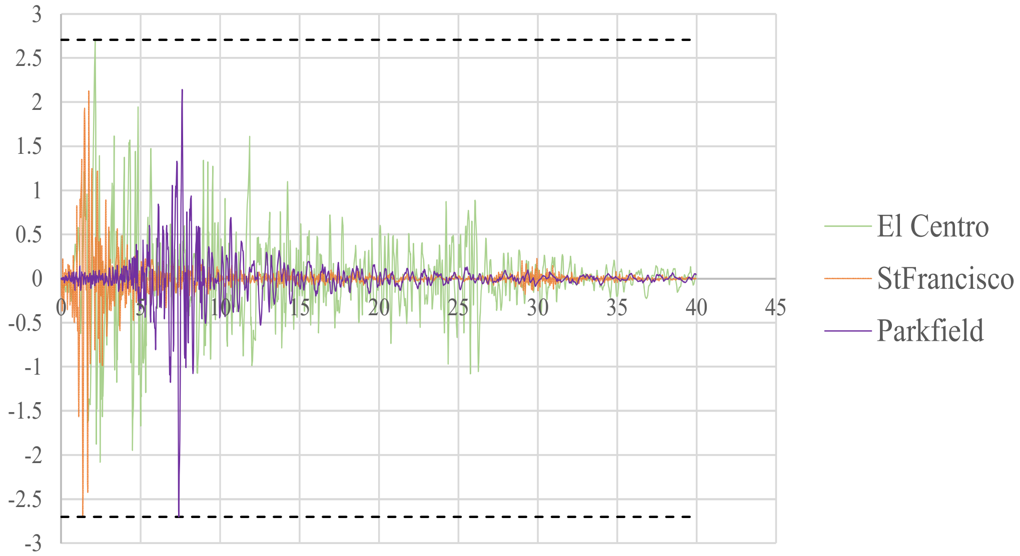


Fig. 9 Accelerograms are scaled to 0.2756*g* of the selected earthquakes, horizontal axis time in s, vertical axis acceleration in m/s².

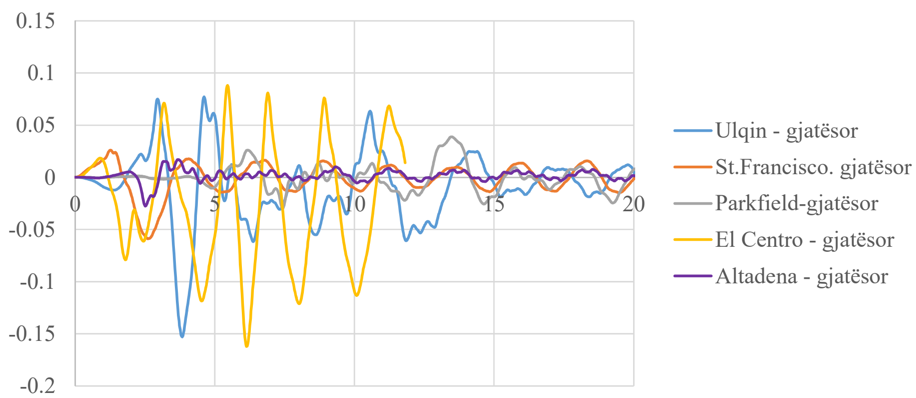


Fig. 10 Maximum displacements, horizontal axis time in s, vertical axis displacement in the longitudinal direction of the bridge in m.

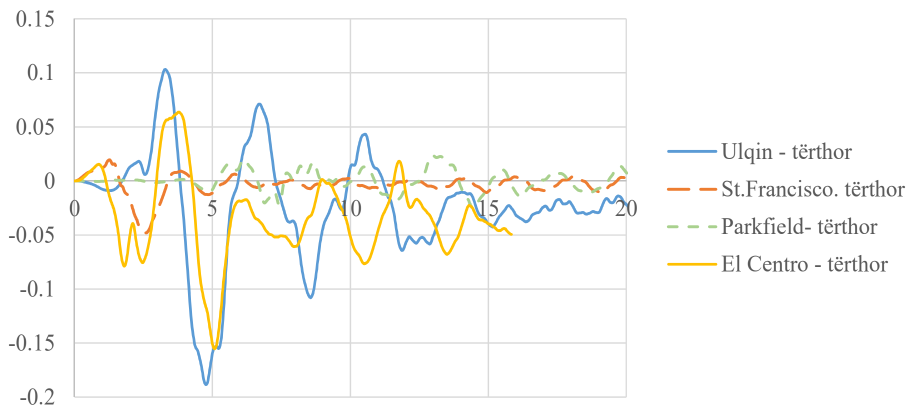


Fig. 11 Maximum displacements of top of cantilever in the transverse direction of the bridge.

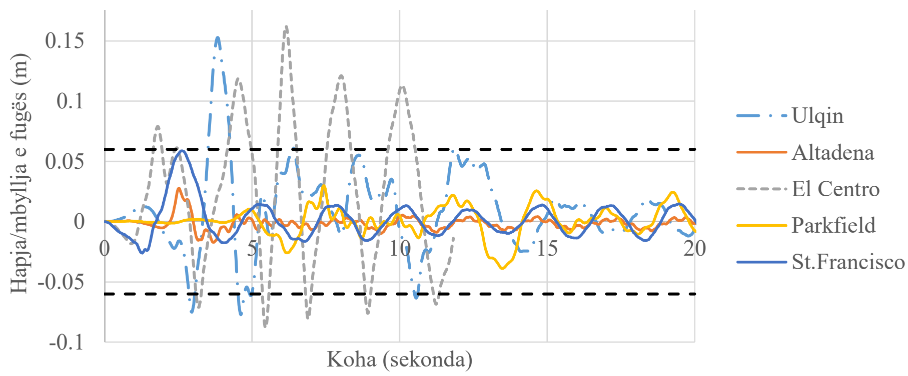


Fig. 12 Opening and closing of the gap as a function of time under the action of the considered accelerograms.

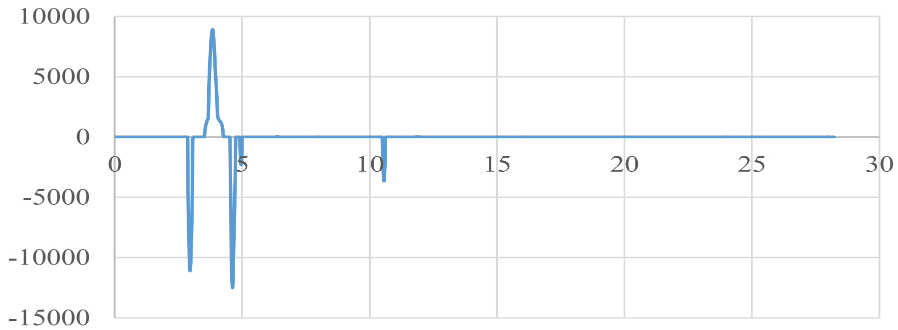


Fig. 13 The forces received by the hinges along the Ulqini earthquake.

Through modeling with non-linear connecting elements, it was possible to obtain Fig. 13, which represents the force that the hinges must withstand at the moment of impact during the simulation of the Ulqini earthquake.

The maximum force is 12,524 kN, which must be supported by 7 dowells of 30 mm diameter, with a total cross-sectional area of 4,945.5 mm^2 and a bearing capacity of 607 kN. It is obvious that fixed hinges are vulnerable in a seismic situation.

3.2.3 Plastic Hinges

The nonlinear analysis in the time domain also enables the study of the creation of plastic hinges as a function of time in the action of simulated earthquakes. Thus, for none of the five simulated bridge earthquakes there is plasticity beyond the yielding of the steel reinforcement.

Even under the action of the accelerogram of the Ulcinj earthquake scaled up to the seismic risk level of Kukesi, it was observed that, for the longitudinal direction (Fig. 14), no large plasticizations of the sections appear, except for the entry into fluidity of the steel reinforcement of the connecting beams of the columns.

In the transverse direction of the bridge, the maximum deformed scheme for the same accelerogram is shown. The same stage of development of plastic hinges is observed: based on the performance levels of plastic hinges according to FEMA [5], yielding of steel. In this case, unlike the longitudinal direction, it is noticed that the steel of the column base section also enters the yield.

3.3 Nonlinear Static Analysis "Pushover"

In the transverse direction, in order to consider the behavior of the structure as realistically as possible, both piles were modeled, despite the fact that the connection between them in this direction is realized only through hinge pins.

For the longitudinal direction of the bridge, a simplified model was built considering the two piles, connected between them with special elements that come into operation the moment the distance between the two piles becomes smaller than the free space of the joint (6 cm in the case of ours).

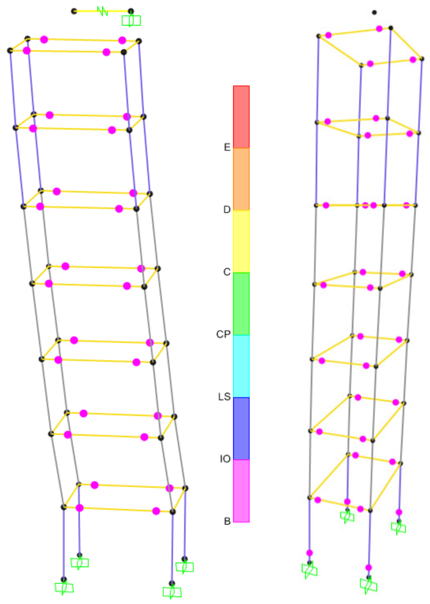


Fig. 14 Deformed scheme (maximum) in the longitudinal direction of the bridge (left), in the transverse direction (right) and the development of plastic hinges.

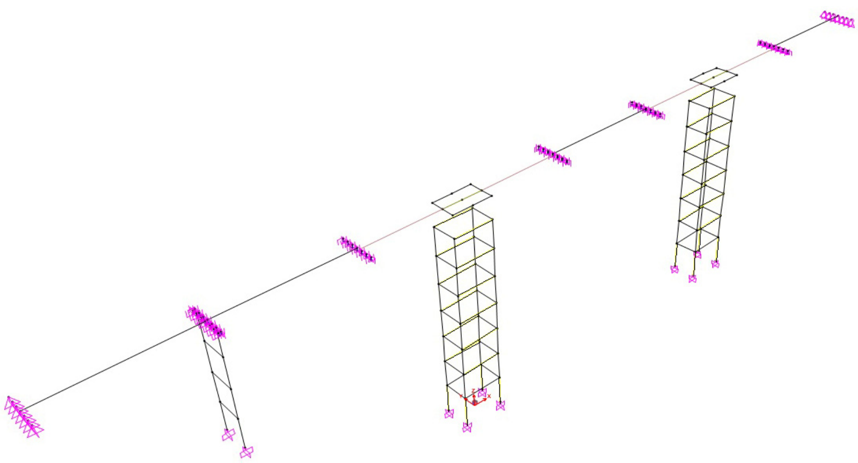


Fig. 15 Calculation scheme of the bridge in the transverse direction.

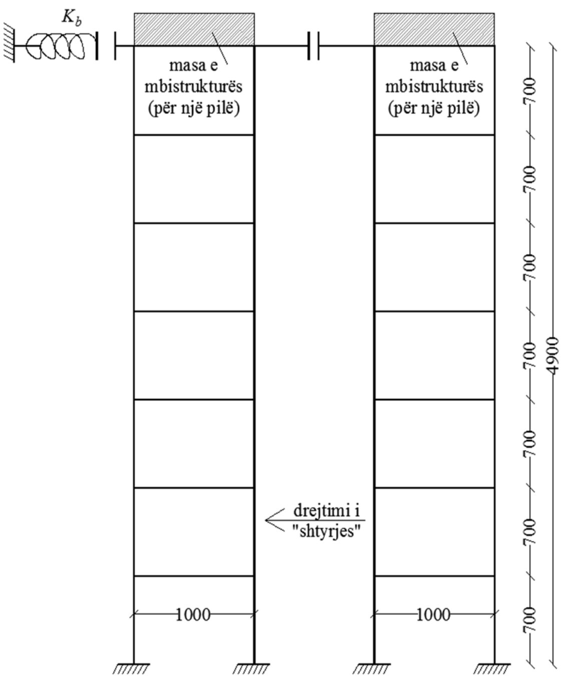


Fig. 16 Calculation scheme of the bridge in the longitudinal direction (dimensions in cm).

In the longitudinal direction, the ties representing the fronts were also placed, which were modeled with a weight:

$K_b = W_b \times h_b \times k_s = 2.021$ GN/m,

where:

$W_b = 8.6$ m; abutment width

$h_b = 5.0$ m; abutment height

$k_s = 47{,}000$ kN/m^3 is the spring coefficient that represents the behavior of soils after filling.

The elastic behavior with buckling K_b continues up to the level when the stresses in the backfill behind the face become 370 kN/m^2 [6] which corresponds to a force of 15,910 kN. After this "yield" strength of the face is reached, the deformations continue up to about 1 m without increasing the strength (ideal plastic behavior), and further the stiffness becomes negative.

As for the connection between the two piles, also in the connection with the fronts, elements were placed that come into operation when the gap space is closed.

Results of "push-over" analyses show that:

In Longitudinal direction: Displacement 92.0 cm, Force 31,568 kN

In Transverse direction Displacement 44.5 cm, Force 19,823 kN

As it appears, the structure has significantly higher capacity in its longitudinal direction. This is an expected result, for two reasons. First, movement in the longitudinal direction encounters obstacles such as end faces and closing joint spaces. Secondly, the pile itself is the heaviest in this direction, since the pitch of its component columns is 10 m in the longitudinal direction, compared to the pitch of 6.2 m in the transverse direction.

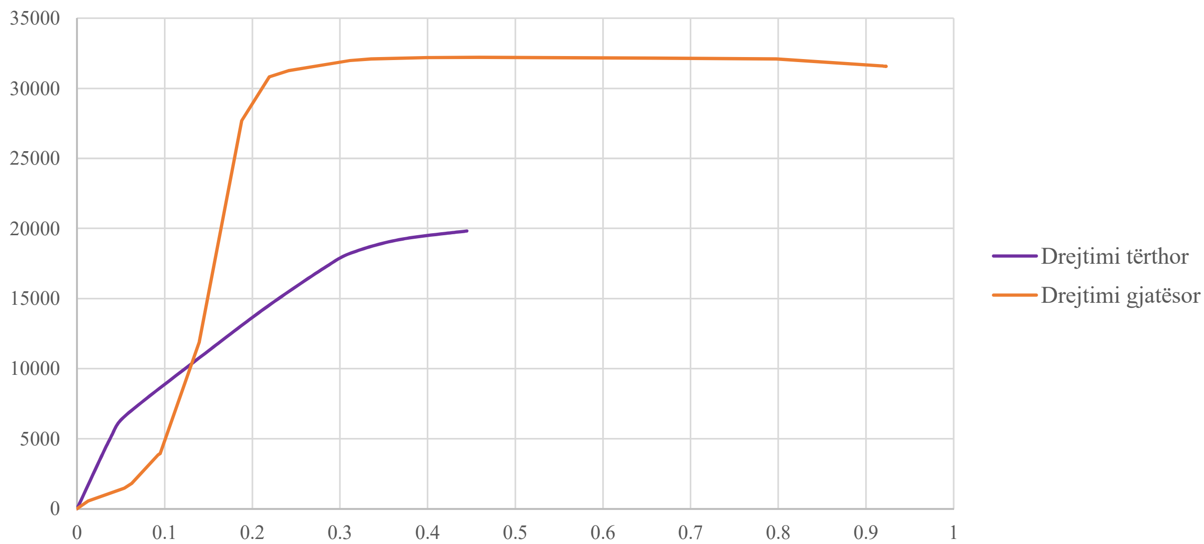


Fig. 17 Pile "pushover" curve, transvers and longitudinal direction (horizontal axis displacement in m; vertical axis shear force in kN).

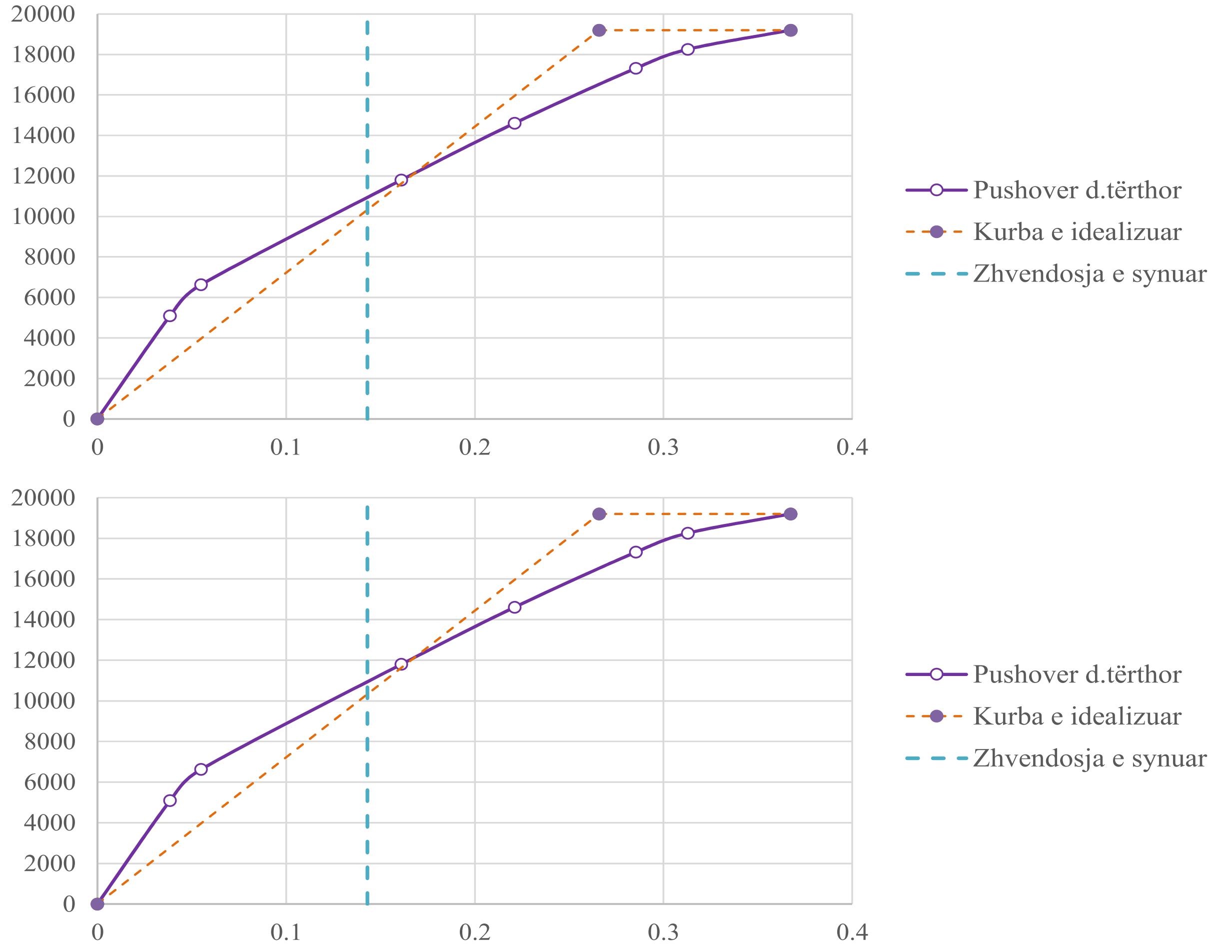


Fig. 18 Target displacement in transverse direction.

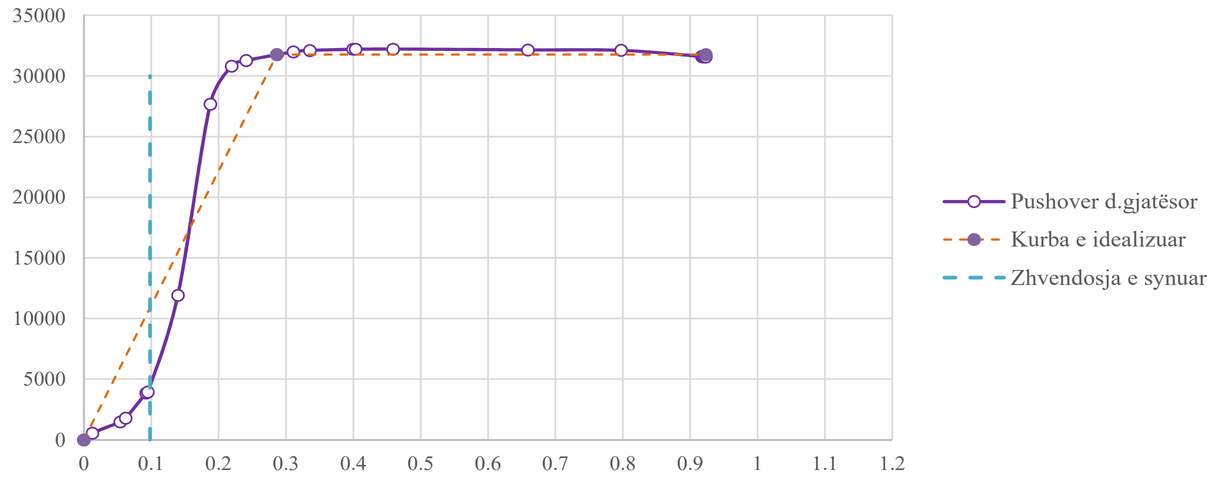


Fig. 19 Target displacement in longitudinal direction.

According to EN 1998-2, for capacity evaluation through "pushover" analysis, the concept of "target displacement" can be used. Thus, for the transverse and longitudinal direction, to find the target displacement the procedure is used in accordance with Annex B.

The target displacement of the bridge pile in the transverse direction is 0.143 m, it corresponds to the elastic phase of the idealized system, if we pass this while in the real curve, it corresponds to a state where we have plastic hinges of some sections.

It is noted that the performance level for this displacement is satisfactory, as we have only sections in which the steel reaches yield, which makes the bridge serviceable immediately after the earthquake.

In the same way for the longitudinal direction, here it appears that the target displacement is 10 cm which is a relatively small size and belongs to a quasi-elastic phase.

It is noted that the structure has sufficient capacity to withstand the seismic action within the target performance scale.

4. Conclusions

(1) This study shows that, regardless of the time when it was designed, the Kukesi bridge in Black Drin generally meets the contemporary requirements of seismic design.

(2) As intuitively understood and as described in the international literature, the seismic assessment of existing structures differs from the design of new structures.

(3) The techniques used in this study such as nonlinear static analysis or nonlinear analysis in the time domain, the results of which turned out to be comparable, are quite efficient for the seismic evaluation of structures. These analyses showed that the global response of the bridge is acceptable, within target levels.

(4) At the local level, it was shown that the elements showing deficiency are hinges and "secondary" beams.

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