SOLUTIONS OF THE INTERMEDIATE SUPPORT STRUCTURES OF THE NORTHERN MARMARA HIGHWAY (ISTANBUL'S RING ROAD) IN THE CONTEXT OF SEISMIC ACTIVITY

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In 2014–2018, as a result of the expansion of the city of Istanbul in Turkey, a project was implemented consisting of building a northern ring road, called the Northern Marmara Highway. The concept of the structural design of the ring road's intermediate supports aims at constructing supports that according to the TURKISH DLH 2008 standard must comply with the design requirements for the three calculated earthquake insensitivity levels (D1, D2, D3). The article discusses the modelling of plastic hinges in the reinforced concrete intermediate supports using finite elements methods. The Ductility Demand-Capacity method was used to determine the geometrical parameters of the cross-section plasticisation zones, their ability to move and rotate, and their ductility. Due to the varied geometry and stiffness of the supports and their non-linear behaviour under dynamic load, this method was concluded to be imperfect. Therefore, an improved algorithm was proposed by determining the main parameters of the plastic hinges depending on the degree of concrete degradation according to Lubliner's assumptions. The new algorithm was used to design the structure of viaducts of the Istanbul Northern Marmara Highway ring road implemented by the authors. The obtained results may help to prevent failures and damage to the viaducts' supports structure and thus ensure the safety of all users exploiting the objects. Based on the collected results, it was proven that the proposed concept of intermediate supports with a variable geometry and stiffness design

including the plastic-damage reinforced concrete model, and on the basis of the plastic hinges concept, broadens and enriches the issue of analysis of bridge structures exposed to earthquakes.
Keywords: viaduct, supports, concrete damaged-plasticity model, seismic impact, plastic hinges

1. Introduction

In the years 2014–2018, in connection with the expansion of Istanbul in Turkey, a project was implemented to build the northern bypass of this city, the Northern Marmara Highway. The bypass is located in areas with a very high risk of earthquakes. In 1999, an earthquake with a magnitude of 7.6 on the Richter scale occurred in Kocaeli province, several dozen kilometres from Istanbul. In addition, studies were carried out in 2000 which showed that there is a 70% risk that by 2030 there will be a magnitude 7 earthquake in Istanbul, which will test the resistance of the viaduct structures of the Northern Marmara Highway bypass.

Designing supports to withstand earthquake loads has for years been one of the most demanding challenges for designers and researchers. When designing supports resistant to seismic impacts, the basic calculation method is elastic and plastic analysis [1]. The method of the theory of elasticity should be used when and only when it is clearly proven that the material (reinforcement in the case of reinforced concrete supports) is not able to achieve a sufficient plastic state and the cross-section of the support element in critical cross-sections has sufficient freedom of rotation. If the above conditions are not met, a plastic analysis should be performed, which is necessary when designing with seismic actions [2, 3].

Plastic analysis methods lead to optimal design by using plastic reserves in the form of plastic hinges. A plastic hinge does not allow rotation with a load lower than that causing the hinge plasticisation, while with a higher load it transfers the limit moment and allows rotation [4]. In the case of a support made of concrete, the correct plastic analysis, taking into account the concept of plastic hinges, requires the use of a concrete plastic degradation model. In this model, when an element is loaded, the load capacity is exhausted as a result of material degradation due to an increase in the external load, and thus the load capacity is lost as a result of an increase in plastic deformation [5–9]. It is a very important element of the analysis, because thanks to the knowledge of the location of the fastest and greatest degradation of the intermediate support material, it is possible to determine where plastic hinges should be created. This is due to the fact that steel reinforcement begins to play the main role in the load transfer where concrete loses its load-bearing capacity [10–12].

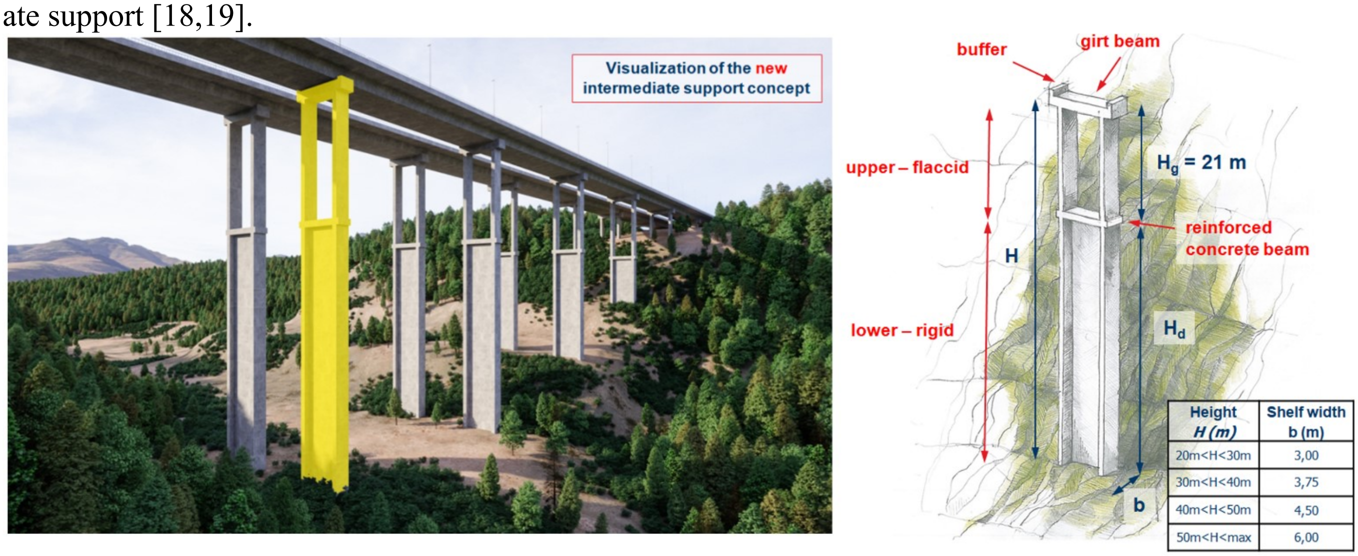
Currently, there are no comprehensively developed recommendations for the method of designing supports with variable geometry and stiffness, taking into account the concept of plastic hinges created with dynamic earthquake loading in mind, and taking into account the complicated plastic-degradative model of concrete with reinforcement of the intermediate supports under dynamic load. When analysing the relevant Polish and foreign literature, the information on this issue was found to be incomplete. The guidelines included in the standards do not cover the issue comprehensively and no other recommendations similar to [13] are given. This knowledge gap was the main premise for the research topic related to the methodology of designing intermediate supports with variable geometry and stiffness, taking into account the concept of plastic hinges based on the plastic-degradation model of concrete with reinforcement in the context of seismic interactions [14–16].

The article presents solutions for the structure of the viaducts of the Northern Marmara Highway bypass in the context of seismic impacts. The concept of the structural shape of the intermediate supports of the viaduct was aimed at designing supports which, according to the Turkish TURKISH DLH 2008 standard, must meet the design requirements for three calculated levels of earthquake intensity (D1, D2, D3) [17]. According to the requirements, the object subjected to seismic loads at the D1 level, which corresponds to an event with a frequency of 72 years, will not suffer any damage. Under the influence of the D2 level, with a frequency of 457 years, the viaduct will be damaged, but the damage will occur in controlled places and can be repaired within a few months with few traffic restrictions. However, for the D3 level, with a frequency of 2,475 years, the controlled destruction of the upper parts of the intermediate supports is to occur through the formation of plastic hinges – the platform should not fall off the cap. In order to meet the above assumptions, a concept for the construction of intermediate supports with variable geometry and stiffness was proposed.

2. Concept of an intermediate support with variable geometry and stiffness

In the original design of the Northern Marmara Highway North Bypass, the supports were as a whole one rigid member with either an I-section or a box section. Due to the locations of the viaducts in seismic areas, this solution was considered inappropriate in the preliminary analyses. In this case, when the supports and their foundations are relatively rigid, the horizontal forces generated by earthquakes reach their maximum value, which is equal to the product of the structure mass and the soil acceleration. In the case of objects with significantly variable heights, such as the intermediate supports, one of the basic problems is the redistribution of internal forces that arise during an earthquake. If supports are used that have uniform stiffness, it will result in the creation of very large internal forces in the longest, stiffest supports. In order to avoid such a situation, intermediate supports with variable geometry and stiffness were designed, thus obtaining more flexible elements.

The new concept of shaping the intermediate supports is based on dividing the support structure into two parts – a rigid (lower) and a flexible (upper) part (Fig. 1). The upper part of the support consists of a pile cap beam, the dimensions of which are: 12 m long, 3 m wide and 2 m high. In the case of high-intensity seismic impacts, the cap is equipped with a reinforced concrete buffer that absorbs the impact of the platform against the support and is connected with two slender reinforced concrete columns with dimensions of 3 x 1 m and a height of 21 m. The geometry of the upper part of the supports has a constant height and the same dimensions for the entire structure, assuming that their total height does not exceed 21 m. The lower part of the support consists of a rigid I-section, terminated with a reinforced concrete slab. The web thickness of the I-section is unchanged and equals 80 cm, and the height is 7 m. However, the flange width of the I-section is variable and depends on the total height of the intermediate support [18,19].



Height H (m)	Shelf width b (m)
20m<H<30m	3,00
30m<H<40m	3,75
40m<H<50m	4,50
50m<H<max	6,00

Figure 1: New support concept

3. Analysis of the dynamic response of the V17 viaduct, taking into account the concrete plastic degradation model

In order to carry out a detailed analysis of the elastic and plastic work of intermediate supports with variable geometry and stiffness, the V17 viaduct was selected for testing, as it has the most diverse range of support heights in relation to each other, and the results of such analysis here can be directly translated to the remaining 35 viaducts.

A 3D model of the viaduct was made in the LS-DYNA program (Fig. 2) [20, 21]. The analysed structure is located on the western side of the ring road, and its total length is 640 metres. The deck is divided into 13 sections. The span of the first and last spans is 45 m, while the span in the middle is 50 m. The platform rests on 14 supports. The height of the highest support is 70 m. In the model, all of the supports are made in accordance with the new concept of shaping the support with variable geometry and stiffness. The platform has a box cross-section. Expansion joints were made for the platform, which was divided into 13 segments. During the calculations, the parameters of the plastic-degradation model of CDPM2 concrete with the strength of C 30/37 were used, in which the load capacity is exhausted as a result of its degradation from the increase of plastic deformation. The parameters of the CDPM2 model were developed on the basis of experimental and numerical studies available in the literature [22]. After the correct validation of the CDPM2 model, it was implemented in the 3D model of the viaduct. The reinforcement was made of ϕ40 steel bars with a plasticity limit of 420 MPa.

The viaduct model was very detailed similar to [23], as an exact numerical model of the elastomer bearings was made. The stressing of the deck load-bearing structure is provided by the centric prestressing cables, located both in the top and bottom plates, and by eccentric cables. Eccentric prestressing cables run through the length of two spans and are staggeredly anchored on deviators located in the top slab. The cables are deflected through intermediate deviators located in the bottom plate (Fig. 2). The platform has been fully reinforced.

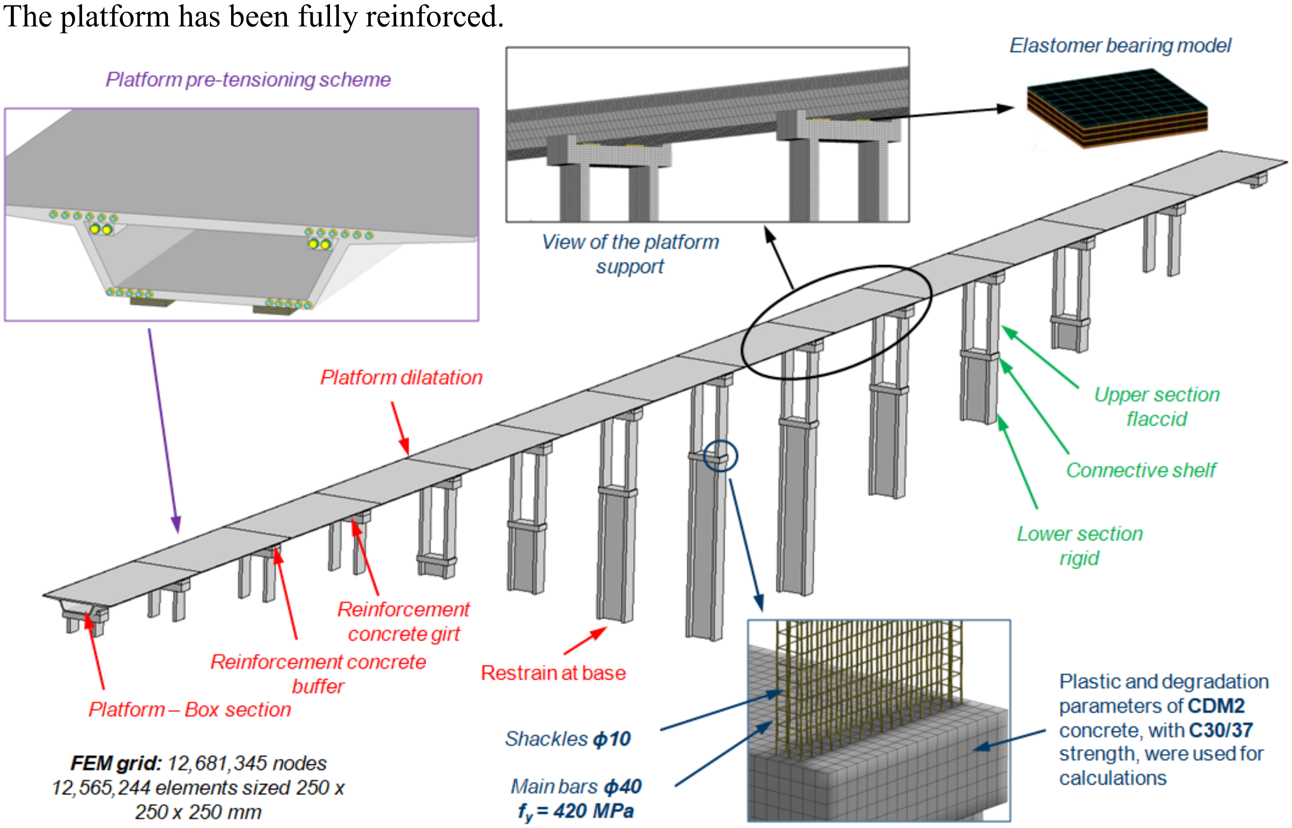


Figure 2: Numerical 3D model of the V17 viaduct with details in LS-DYNA

Below, we present our algorithm, which shows how the degradation is calculated in each iteration step. The new algorithm, DDC2 (Ductility Demand-Capacity 2 method), consists of two parts. The first part describes how the concrete is degraded, while the second part describes the plastification of the reinforcement bars taking into account the reinforcement bond's transition zone.

Algorithm 1: New DDC2 algorithm for determining a cross-section's plastification parameters

PART 1 – *Concrete degradation*

1. Determine the elastic stiffness matrix C^e
2. Deform at the beginning of the increment $(\varepsilon)^{t+\Delta t} = (\varepsilon)^t + \Delta\varepsilon$
3. Effective deformation $(\tilde{\varepsilon})^{t+\Delta t} = exp(-L^2/2)(\varepsilon)^{t+\Delta t}$
4. Effective stresses $\tilde{\sigma} = C^e : (\tilde{\varepsilon})^{t+\Delta t}$
5. Determine the non-local elastic deformation energy $\bar{Y}^*$
6. Check for plastification $f(\bar{Y}^*) - r(L^{t+\Delta t}) \geq 0$
7. If condition (6) is met:
   a) initialise: $f^{(0)} = f(\bar{Y}^*), \Delta L^{(0)} = 0, i = 0$
   b) calculate correction δL (equation 22)) and update of the increment $\Delta L^{(i+1)}$
calculate correction $\delta\lambda$ (equation 10) and update $\Delta\kappa^{(i+1)}$
calculate the non-local reinforcement parameter $(\Delta\hat{\kappa}^*)^{(i+1)}$
   c) update the load function $f(\bar{Y}^*)^{(i+1)}$
   d) verify the convergence condition $\left|f(\bar{Y}^*)^{(i+1)} - r\left(L^t + \Delta L^{(i+1)}\right)\right| \leq Tr^{(0)}$
8. If condition (7d) is met: stress update $\sigma^{t+\Delta t}$ (equation 26) or determine strains $(\varepsilon^e)^{t+\Delta t}$and $(\varepsilon^p)^{t+\Delta t}$
   (equation (20–21)), otherwise, return to point (7b).
9. If condition (6) is not met: $L^{t+\Delta t} = L^t$
10. Implementation of: elastic-plastic model's parameters with stiffness degradation that characterises the emergence of plastic deformations, variable compressive and tensile deformation, and the parameters that can simulate the *stiffness recovery effect*

PART 2 – *Reinforcement bar plastification with reinforcement transition zone taken into account*

1. Calculate the idealised yield curvature defined by an elastic-perfectly-plastic representation of the cross ϕ_{Yi}
2. Calculate the distance from the point of maximum moment to the point of contra-flexure L_i
3. Determine the idealised yield displacement of the column at the formation of the plastic hinge $\Delta_{Yi}^{col} = \frac{L_i^2}{3}\phi_{Yi}$
4. Determine the transition zone range $L_{p2}^{zone} = max(0.3L_i; 1.5\ metres)$
5. Calculate the equivalent analytical upper plastic hinge length $L_{p1} = \begin{cases} \mathbf{0.08}\boldsymbol{L} + \mathbf{0.15}\boldsymbol{f_{ye}d_{bl}} \geq \mathbf{0.3}\boldsymbol{f_{ye}d_{bl}} \\ \mathbf{0.08}\boldsymbol{L} + \mathbf{0.02}\boldsymbol{f_{ye}d_{bl}} \geq \mathbf{0.4}\boldsymbol{f_{ye}d_{bl}} \end{cases}$
6. Calculate the equivalent analytical lower plastic hinge length $L_{p2} = \begin{cases} 0.08L + L_{p2}^{zone} + 0.15 f_{ye} 2d_{bl} \geq 0.3 f_{ye} d_{bl} \\ 0.08L + L_{p2}^{zone} + 0.02 f_{ye} 2d_{bl} \geq 0.4 f_{ye} d_{bl} \end{cases}$
7. Determine the curvature capacity at the Failure Limit State, defined as the concrete strain reaching or the longitudinal reinforcing steel reaching the reduced ultimate strain ϕ_{ui}
8. Calculate the idealised plastic curvature capacity (assumed constant over L_{pi}) $\phi_{pi} = \phi_{ui} - \phi_{Yi}$
9. Calculate the plastic rotation capacity (radian) $\theta_{pi} = L_{pi}\phi_{pi}$
10. Determine the idealised plastic displacement capacity due to rotation of the plastic hinge $\Delta_{pi} = \theta_{pi}\left(L_i - \frac{L_{pi}}{2}\right)$
11. Determinate the total elastic and plastic displacement $\boldsymbol{\Delta_{ci}} = \Delta_{Yi}^{col} + \boldsymbol{\Delta_{pi}}$

During the modelling of the connection of bars in the transition zone, boundary conditions were created describing the connection of bars coming from the rigid part with bars located in the flexible part of the support. The CONSTRAINED_GENERALIZED_WELD_SPOT function was used to describe this connection. The nodes of the bars (rigid part and flexible part) are spot welded together with failure criteria.

The results obtained during the numerical analysis are presented in Fig. 3 and Fig. 4.

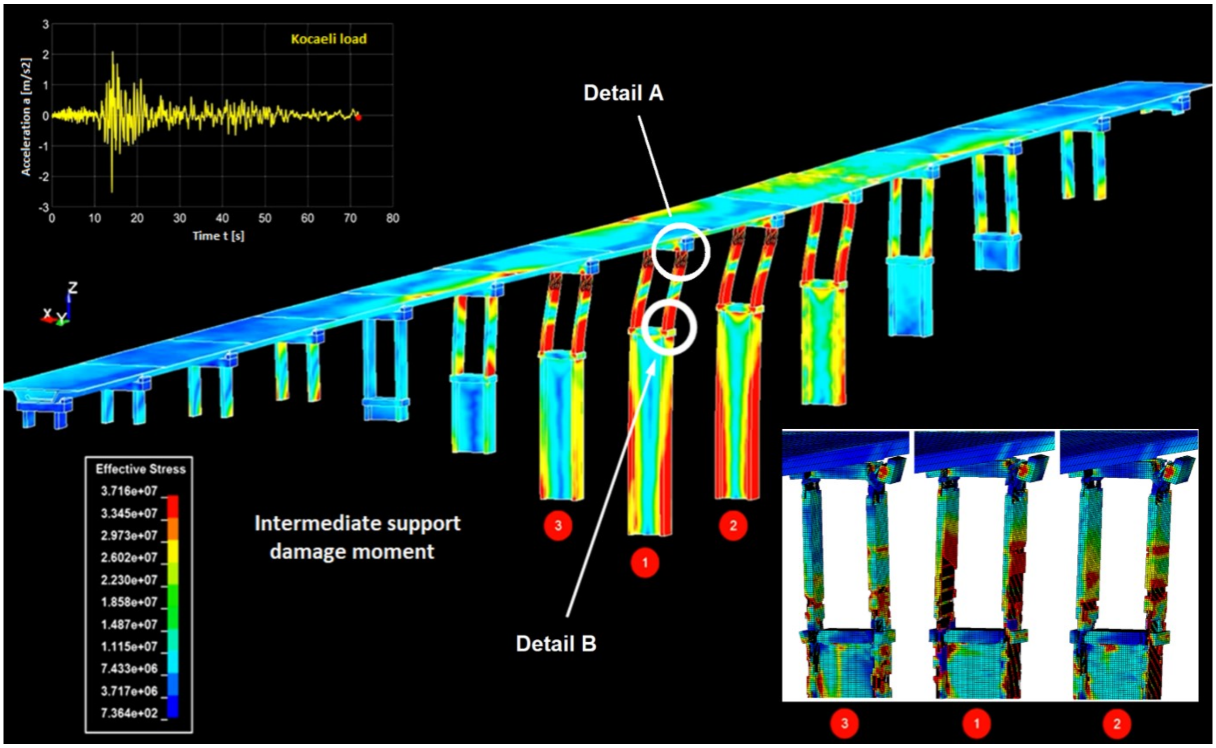


Figure 3: Destruction of the V17 viaduct in the 72nd second of the Kocaeli level D3 earthquake

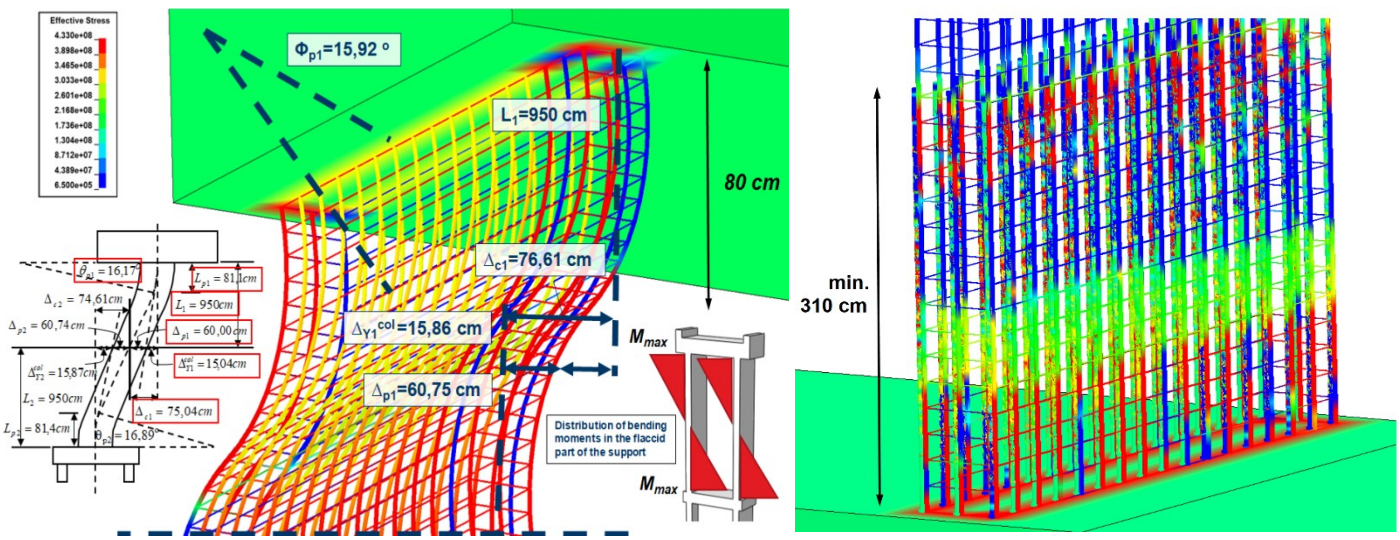


Figure 4: View of detail A (left) and detail B (right)

4. Conclusions

Based on the obtained results, it was found that controlled destruction and plasticisation would take place only in the upper part of the 3 highest supports. Thanks to this, the damage to the intermediate supports will be reduced to a minimum. Possible repairs of supports will be possible under the viaduct platform. On the stress map of detail A, the upper plastic hinge will be created at a height of about 80 cm. The reinforcing bars reached the yield point at stresses of 433 MPa. The stress map of detail B shows a zone which indicates that when the reinforcement is bonded at a minimum height of 310 cm, the bars coming out of the rigid part of the support and the bars starting in the flexible part work perfectly together. As the concentration of maximum stresses shifted to the very bottom of the bond, the stress value was 314 MPa, which is much less than the yield point. The proprietary concept of shaping the armor is the only one that meets the requirements of the D1, D2, D3 level of the Turkish standard.

The usage of the new DDC2 algorithm in numerical calculations in other facilities may help to prevent failures and damage to viaduct support structures, thus ensuring safety for all users using these facilities. Based on the results obtained, we have proven that the proposed new DDC2 algorithm and the new concept of shaping intermediate supports with variable geometry and stiffness enriches the analysis of bridge structures exposed to earthquakes, based on the finite element method.

Photos from the implementation of the results obtained based on the new DDC2 algorithm on the construction site of the Istanbul Northern Marmara Highway ring road are presented in Fig. 5.

Figure 5: Photos from the implementation of the new DDC2 algorithm on the Northern Marmara Highway construction site

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