Extended Abstract – Seismic evaluation and reinforcement of a viaduct

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Abstract

The present study aims to carry out the evaluation and structural strengthening of an existent viaduct. For this, was used the methodology of evaluation of seismic resistance of existing structures, presented in part 3 of Eurocode 8. The current version of this standard only contemplates structures of buildings, however due to the relevance of the bridges and the growing awareness to the level of safety of existing structures, it has become imperative to include this type of structures in the standard. The future version of EC8-3, which was the basis for this dissertation, is being developed and includes seismic evaluation of both type of structures, and the reinforcement alternatives if the structures prove to be seismically ineffective.

At the beginning, two introductory and illustrative chapters are presented. In the first one is made a reflection of the effects of the earthquakes in bridges, and it is shown the most common collapses in these structures in order to interpret the causes that led to their occurrence. Next, are exposed the various reinforcement alternatives to make the structures seismically efficient.

There are two methodologies of seismic design, one based on forces and another based on displacements. In the analysis of existing structures, the methodology based on displacements is the most adequate, being favored in EC8-3.

Subsequently, the methodology of calculation of seismic evaluation suggested by EC8-3 is presented and illustrated in a case study, where it was concluded that the bridge analyzed did not verify the seismic safety, so it was necessary to proceed with the design of a structural reinforcement intervention.

1 Introduction

The registration of the first regulation in Portugal related to the design and execution of concrete structures appeared in the year 1918. With the evolution of knowledge of concrete structures behavior, it became imperative that existing standards and regulations be updated. In this context, the regulations of 1935, 1967 and 1983 appeared. At the end of the decade of 2000 the Portuguese standards of structural Eurocodes were published.

Eurocodes, which aim to promote greater safety of structures when compared to old regulations, have only been implemented in recent years, so many existing structures do not satisfy the current design requirements. The Eurocodes, which have a philosophy of constant updating, reflect the knowledge resulting from the study and research carried out over the last decades.

In recent years, concern has been expressed about perceiving the level of security of existing structures. In this way, European standards considered it necessary to develop guidance documents whose theme would be the assessment and retrofitting of existing structures. This, launched in 2005, falls under part 3 of Eurocode 8 and its application has demonstrated the existence of serious deficiencies in the capacity of resistant structures of the old quake.

However, this norm does not contemplate the analysis of bridges, dealing only with buildings. The relevance of this type of structures, promoted an update of the document that includes prescriptions for bridges, whose publication will occur soon.

The study developed in this dissertation aims to present the concepts of the new version of the EC8-3, in order to illustrate the methodology of seismic evaluation of bridges based on European regulation.

2 Seismic effects in bridges

The present chapter aims briefly to illustrate how poor consideration of the seismic action level, construction or design errors can have consequences on structural behavior, particularly for bridges, and which are the most common collapses.

In general, reinforced concrete bridges, have a relatively simple structure, composed of a deck that is supported by columns. In this way, the design of the deck is usually conditioned by the gravitational loads, while the columns are by the horizontal loads.

As one of the objectives of this thesis is to demonstrate the application of EC8-3, relevance will be given to the most common problems that existing bridges present when exposed to a seismic action, which majority have origin in the resistance of the piers to this horizontal action. The following are presented some of most common collapses

- Damages in piers: The fact that the bridges present less redundancy makes the analysis of the seismic behavior much simpler when compared to the behavior of the buildings. Although in a smaller number, the formation and location of plastic hinges is a much more predictable process. However, the less redundancy of this type of structures implies greater care in the design and detailing, requiring a greater level of reliability. Damage in piers can occur for several reasons: Reduced resistance to shear, design defects like short piers where problems arise due to poor ductility and dispensing of longitudinal reinforcement in inadequate sections.
- Damage in bearings and abutments: Usually associated to a careless estimation of seismic displacements. So, it is very important that in the design or analysis of a structure the stiffness defined for the elements be close to what they actually present. This is a parameter with a great influence in the displacements presented by a structure. The fact that the displacements presented are higher than those considered in the design can lead to the rupture of support devices. On the other hand, the high displacements of the deck can lead to shocks with abutments which can promote structural damages.
- Damages in foundations: Associated to the liquefaction of the soil, promoted by the seismic excitation, which can lead to rupture of a foundation by rotation. This question has little relevance for the purposes of this dissertation.

3 Strengthening Alternatives in Reinforced Concrete Bridges

There are several structural intervention alternatives that aim to prevent the collapses mentioned above.

From the previous chapter we can see that when a seismic occurs most of the collapses in bridges, have origin in:

- Piers due to:
  - insufficient transversal reinforcement;
  - reduced ductility of the elements;
  - defects in reinforcement detailing or execution.
- Bearings, abutments and deck due to an incorrect estimate of the seismic displacements.

For this dissertation it is interesting to deepen the first point, related to piers, because these elements are subject to greater requirements during the seismic excitation. In this way, the analysis presented below will fall on in the piers, and in case of a collapse, the viable alternatives to make them efficient in the seismic response.

The safety check of an element consists to compare the resistance with the effects induced by the action. In this way, if an intervention is necessary, it can act on the resistance of the element (increasing it) or on the level of the effects (reducing them). Both approaches will be presented in this chapter.

As will be explained below, the EC8-3 define that the structural evaluation should be realized in terms of deformation (to ductile mechanisms) as well in strength (to fragile mechanisms).

Relatively to the rupture by reduced ductility of the element, we are faced with a particular situation, because even if the elements guarantee resistance to the requesting action, occurs a premature collapse due to the reduced deformation capacity.

3.1 The question of ductility and its relevance on bridges

The intensity of an earthquake is translated by acceleration of the ground, which promote displacements in the structures, and subsequently structural stresses in the elements, that in the case of a seismic action, can be particularly high. If we design the structure for these stresses, we will further a linear behavior, which has the following disadvantages:

- The high value of the forces leads to large sections and reinforcement, which is aesthetically and economically harmful;
- The seismic action has an unpredictable nature, so the values used in the design may be lower than those that will actually occur;
- Safety is guaranteed up to the elastic design values, however, if they are exceeded, a fragile and sudden collapse may occur.

With the knowledge developed in last years, it was realized that it would be possible to exploit the resistant capacity of structures in the non-linear domain, and that with certain precautions, this behavior is not synonymous of structural insecurity. For the seismic action, which is nothing more than the a displacement induced to the structure, the exploration of non-linear behavior, besides being possible and safe, is advantageous and advisable.

In this way, instead of design the structures to resist to forces resulting from the seismic displacements, we can use lower stress levels, although, it is necessary to guarantee, with a constant resistence, deformation capacity to reach de seismic displacement. This capability is defined as ductility.

The Figure 3.1 illustrates the linear and non-linear behavior of the structures and the influence of increasing ductility.

The design assuming a non-linear behavior allows to overcome the disadvantages previously presented. For a lower level action effects the sections can be more economical, and on the other hand, the ductility guarantees a reserve of deformation, which allows to face seismic of greater intensity than those foreseen in the design.

However, there are also disadvantages:

- Promote a non-linear behavior of the structure, will carry to a higher level of structural damage;
- Greater sensitivity to errors, which implies greater control in the design, detailing and execution of the reinforcements in the dissipative zones.

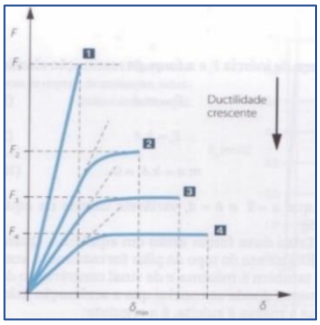


Figure 3.1. Force-displacement diagrams of a pillar to various ductility levels

The bridges are usually simple structures, which results in less redundancy of the structural system, that is, a smaller number of plastic hinges involved in the collapse mechanism. So, it is necessary to be more cautious in the exploration of ductility, especially in the design to allow a good behavior of the plastic hinges.

3.2 Reinforcement by increase of resistance of the elements

The seismic action defined in EC8 is, in some areas, more demanding than the presented in the RSA, so the structures built in the 70's/80's may be unsafe to high intensity seisms. A possible approach to solve this problem is to strengthen the structural elements. There are three alternatives of strengthening that allow to increase the resistance of the elements, making them efficient to actual regulation actions:

- Concrete jacketing;
- Steel jacketing;
- FRP plating and wrapping.

Some of these alternatives have the capacity to increase concrete confinement, which is favorable in the exploration of ductility.

3.3 Strengthening by reduce the effect of action on the elements

Sometimes an intervention by increasing the resistance of bridge piers, is not adequate. If the level of resistance to be guaranteed in the piers has very high, it may lead to the impracticability of section strengthening. Another fact is that some strengthening's do not promote an increase of ductility. Therefore, a viable alternative to make structures safe, without increasing the resistance of the elements, is promote the reduction of action effects, which can be reached using dissipative equipment or seismic isolation.

3.4 Resistance models according to EC8-3 for assessment of existing reinforced concrete structures

3.4.1 Ductile mechanisms: combined bending and axial load

The EC8-3 philosophy refers that for the mechanisms associated to bending, structural evaluation must be performed in terms of deformations. The deformation capacity of an element is defined by the chord rotation.

The maximum rotation, θ_u, of an element is composed to the sum of two parts. An elastic part, corresponding to the rotation which occurs the element yielding, θ_y; and a plastic part, corresponding to the rotation developed after the yield (formation of the plastic hinge), θ_{pl}

For columns of rectangular section, the EC8-3 recommends the following semi-empirical expression to calculate yield rotation, θ_y,

$$\theta_y = \emptyset_y \frac{L_v + a_v z}{3} + 0{,}0019\left(1 + \frac{h}{1{,}6 L_v}\right) + \frac{\emptyset_y d_{bL} f_y}{8\sqrt{f_c}} \tag{3.1}$$

Where $\emptyset_y$ is the curvature exhibited by the end section at yielding; L_v is the shear span; h is the height of the section; a_v is the translation factor of the bending diagram; z is the distance between the internal forces of the section; f_y and f_c are the mean value of steel yield strength and the mean concrete compressive strength, in MPa d_{bL} is the (mean) diameter of the tension reinforcement.

This expression, based on physical models and calibrated by experimental tests, provides values approximated to reality.

For the ultimate chord rotation, the EC8-3 recommends two expressions. One, with empirical basis, allows to obtain the plastic part of rotation, θ_u^{pl}, and it must be added to θ_y:

$$\theta_u^{pl} = \kappa_{conform} \cdot \kappa_{axial} \cdot \kappa_{concrete} \cdot \kappa_{shearspan} \cdot \kappa_{confinement} \cdot \theta_{u\,0}^{pl} \tag{3.2}$$

Where $\theta_{u\,0}^{pl}$ is the basic value of plastic chord rotation capacity of an element, assuming: i) the element is detailed for ductility; ii) concrete strength equal to 25 MPa; iii) $L_v/h = 2{,}5$ at the section of maximum moment; iv) zero axial force; and v) symmetric reinforcement concentrated at the ends section; with these assumptions, $\theta_{u\,0}^{pl}$ should be taken equal to: 0,039 rad if the element is a beam or a column with section consisting of rectangular parts; 0,023 rad if the element is a rectangular wall; 0,027 rad if the element is a wall with a T, I, H, C or box section.

Then, several factors k are applied to $\theta_{u\,0}^{pl}$, to adjust the base value of the rotation, to the specific case of the element under analysis.

If the previous expression is not applicable, the EC8-3 advises another one based on physical models and applicable to any section, that results in estimation of the ultimate rotation, θ_u:

$$\theta_u = \theta_y + (\emptyset_u - \emptyset_y) L_{pl} \left(1 - \frac{0{,}5 L_{pl}}{L_v}\right) + \Delta\theta_{u,slip} \tag{3.3}$$

Where $\emptyset_u$ is the ultimate curvature at the end section; L_{pl} is the plastic hinge length, whose value must be multiplied by 1.3 if the detailing do not comply with the regulation; $\Delta\theta_{u,slip}$ is the post-yield fixed-end rotation due to yield penetration in the anchorage zone beyond the yielding end of the element.

The values returned by the expression (3.3) are more conservative than those obtained by the empirical expression (3.2).

3.4.2 Brittle members and mechanisms: shear

The evaluation of elements subject to bending, whose mechanisms are ductile, is carried out in capacity rotation. This type of analysis is very expedient because it allows to compare the displacements imposed by the seismic action with the elements capacity to accommodate them.

However, in the shear evaluation the analysis based on rotations is no longer appropriate, because the mechanisms of rupture are fragile, so the analyses should be realized in terms of forces.

Thus, the calculation of shear resistance depends on the stage that the shear rupture occurs. If the element stays in an elastic regime, the shear resistance should be calculated using EC2-1:

$$V_{Rd}^{EC2-1} = min\left(máx\left(V_{Rd,s}; V_{Rd,c}\right); V_{Rd,max}\right) \quad (3.4)$$

On the other hand, if the rupture occurs in a plastic regime, the shear resistance should be calculated using EC8-3. The norm suggests an expression, which considers the degradation of shear strength when the element is in cyclic plastic regime. This degradation is materialized by the plastic ductility factor, $\mu_{\Delta}^{pl} = \left(\theta_{pl}/\theta_{y}\right)$. The following expression is only usable where the element is in a plastic regime.

$$V_R = \frac{h-x}{2L_v} min(N; 0{,}55A_c f_c) + \left(1 - 0{,}05 min\left(5; \mu_{\Delta}^{pl}\right)\right) \cdot [V_c + V_w] \quad (3.5)$$

Where V_c and V_w represent the contributions of the concrete compression zone and the web reinforcement to the shear strength, calculated by $V_{Rd,s}$ expression but considering θ=45º. V_c should be calculated by the following expression:

$$V_c = 0{,}16 max(0{,}5; 100\rho_{tot}) \cdot \left(1 - 0{,}16 min\left(5; \frac{L_v}{h}\right)\right) \sqrt{f_c} A_c \quad (3.6)$$

Finally, the plastic shear strength is obtained by:

$$V_{Rd}^{EC8-3} = min\left(V_R; V_{Rd,max}\right) \quad (3.7)$$

3.5 Safety verification in linear analysis

At the beginning, safety check analyzes the rotation induced by the seism in the sections of formation of plastic hinges, θ_{Ed}. If it is inferior to the yield rotation, it does not occur ductile rupture, so there is no plastic hinge formation. However, it is necessary to verify the safety to fragile mechanisms. In an elastic regime, the shear resistance is calculated by EC2-1 (V_{Rd}^{EC2-1}).

On the other hand, if the induced rotation is superior to the yield rotation, plastic hinge formation occurs in the critical zone, so the pier behavior is in a plastic regime. In this way, the shear resistance is calculated by EC8-3 (V_{Rd}^{EC8-3}). However, it is necessary to verify if in these cases there is no shear premature collapse at the time of plastic hinge formation, so the elastic shear resistance must be compared with the shear stress at the yielding ($V_y = M_y/L_v$).

3.6 Verification of Limit States

The future version of EC8-3 presents a different philosophy when compared with the old one. Previously the knowledge level was materialized in a confidence factor (CF) used to reduce the calculation tensions of the materials, and indirectly reducing the resistance capacities of the elements. For the verification of limit states, a safety factor is applied to the resistance capacities, which value depend if the element is primary or secondary. In the new version, the resistance capacities calculation is carried out using mean material properties, and only afterwards the values obtained are divided by partial factors, γ_{Rd}, that reflect the uncertainty respecting to the resistances, and are a function of the knowledge level (KL) and of the limit state to check.

4 Seismic assessment of a viaduct

In this chapter the methodology of evaluation of the new version of the EC8-3 is applied to a practical case of a viaduct.

4.1 Description of the viaduct

The viaduct consists in two end spans (28m) and 9 intermediate spans (35m), presenting a total development of 371m and a width of 30,32m.

The deck consists in a ribbed slab with two ribs of 1.60m high and variable width between 2.5m at the base and 3.0m at the slab connection. The deck supports in two piers by alignment, with uniform height of 10m and a section of 0.70x2.0 (m).

Regarding the structural design, the connection between the deck and the abutments is materialized by supports that allow longitudinal displacements and impede the transversal ones. At the top of the piers, the supports are fixed, inhibiting bending transmission between the deck and the piers.

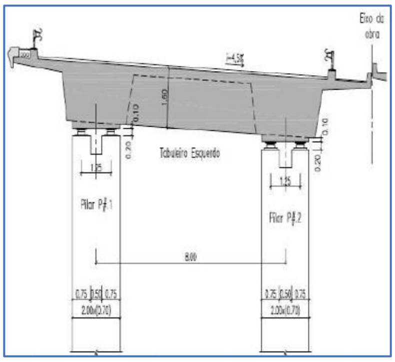
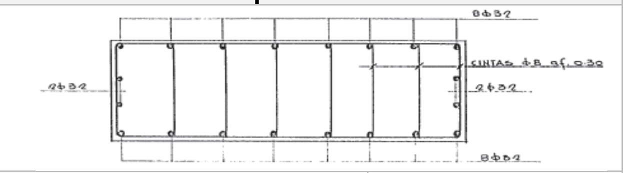
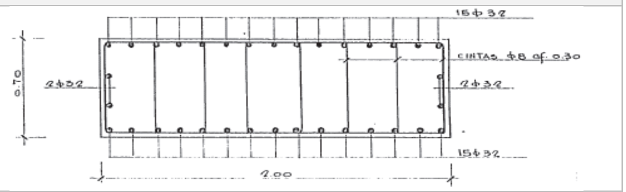


Figure 4.1. Cross section of the viaduct in the alignment of the piers

The analysis will fall globally on the piers, since these are the structural elements that most contribute to the seismic response of the viaduct. To calculate the resistance capacities of these elements it is necessary to analyze the dimensions and the reinforcement detailing.

Table 4.1. Dimensions and reinforcement details of the piers

Top Section	
	
Longitudinal Reinforcement	20Ø32
Transversal Reinforcement	Ø8//0.30
Base Section	
	
Longitudinal Reinforcement	34Ø32
Transversal Reinforcement	Ø8//0.30

4.1.1 Representative values of material properties

The resistance capacities should be calculated considering the mean properties of the materials. These should be obtained by using experimental tests and other sources of information. However, if no tests are performed the EC8-3 prescribes some considerations to obtain the mean tensions. For the case of concrete, should be use (4.1):

$$f_c = f_{ck} + 8MPa \tag{4.1}$$

For the case study, the mean stress of the concrete, f_c, was obtained by (4.1). For steel the mean yield stress, f_y, was conservatively taken equal to 500MPa.

4.2 Effective stiffness of the principal members

The stiffness of the structural elements is a parameter that influences considerably the seismic response of a structure, so its definition in the analysis model must be carried out with rigor.

In the design of new structures, which is usually performed using linear analyzes, the EC8-1 allows a simplified approach that considers 50% of non-cracked stiffness for value of effective stiffness. In general, the value obtained by this way is higher than the real one, however, the EC8-1 methodology is based on forces, and a higher stiffness value leads to higher forces, so this is a conservative analysis.

In the seismic evaluation based in displacements, higher values of stiffness are associated with an estimate of displacements by lack, which is against safety. Thus, EC8-3 prescribes that in elements whose non-linear behavior is expected, the effective stiffness must be obtained by (4.2). For elements with linear behavior can be used the non-cracked stiffness.

$$EI_{eff} = \frac{M_y \, L_v}{3\theta_y} \tag{4.2}$$

Where M_y e θ_y are the yielding bending and rotation.

In viaducts it is expected that the piers present a non-linear behavior in the response to a seism, so should be modeled considering the effective stiffness. For the decks, is expected a non-linear behavior, so the non-cracked stiffness can be used.

For the case study were reached the following values of effective stiffness: $EI_{eff}^{Long.} = 0{,}41 \cdot EI_I$ e $EI_{eff}^{Trans.} = 0{,}25 \cdot EI_I$. Note that the values obtained are considerably lower than those allowed by EC8-1.

4.3 Evaluation of the rotation capacity

As prescribed in EC8-2, the total displacement to be considered in the seismic design situation, d_{Ed}, for which the structure should be verified, is obtained by:

$$d_{Ed} = d_E + d_G + \Psi_2 d_T \tag{4.3}$$

Where d_E is the design seismic displacement; d_G the long-term displacement; d_T thermal movements displacement; and Ψ_2 combination factor for permanent actions ($= 0{,}5$).

The displacements associated to imposed actions and temperature variations, $d_G + \Psi_2 d_T$, were considered using the equivalent temperature concept, ΔT_{equi}, whose value was 36^oC.

Thus, the value of d_{Ed}, to be considered in the evaluation must be obtained by adding seismic action effects with those associated to ΔT_{equi}. The induced rotation is obtained by: $\theta_{Ed} = d_{Ed}/L$.

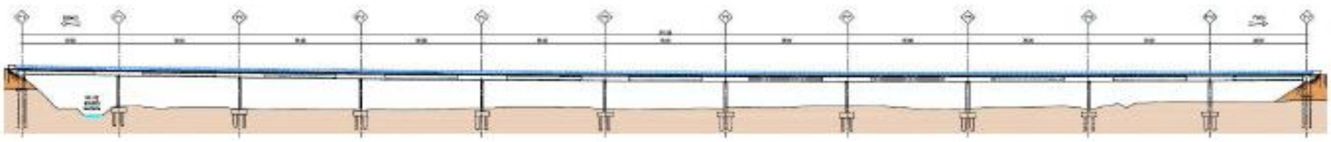


Figure 4.2. Longitudinal section of the viaduct

In order to realize the safety verification of the piers in terms of rotation, it was necessary to calculate the resistance capacities using the expressions (3.1) and (3.2). In the following figures are illustrated the safety check for both directions of seismic action in Faro.

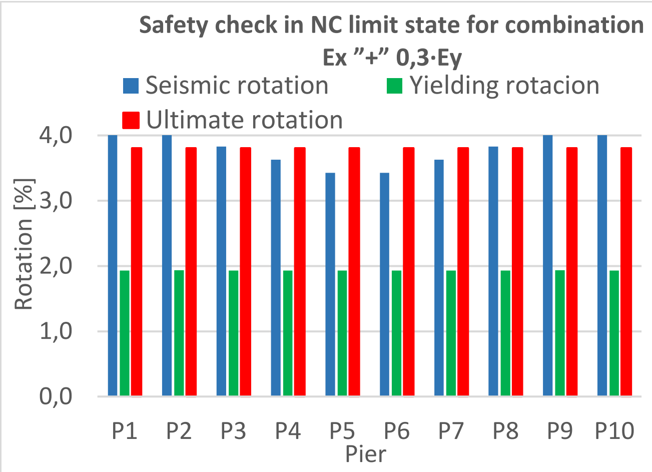


Figure 4.3. Rotation safety check in NC limit state for combination Ex ”+” 0,3·Ey

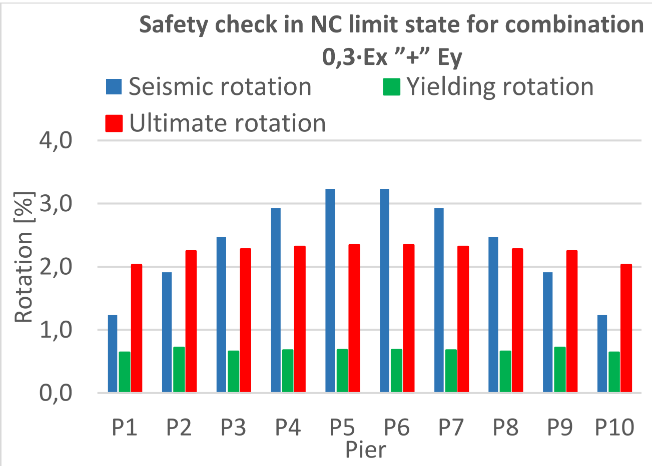


Figure 4.4. Rotation safety check in NC limit state for combination 0,3·Ex ”+” Ey

None of the piers verified simultaneously the safety of the NC limit state for the two seismic combinations.

4.4 Evaluation of the shear strength

To know if the seismic promotes a nonlinear behavior of the piers, it is necessary to verify if the induced rotation is superior to the yield rotation. As can be seen in previous figures, all the piers present plastic hinge formation capacity. However, the plastic hinge only appears if there is no premature shear failure, ie, case $V_y = M_y/L_v \le V_{Rd}^{EC2-1}$. If the conditions for formation of plastic hinge are satisfied, the verification should be performed considering the shear stress, V_{Ed}, calculated according to the Capacity Design, and the plastic shear resistance ($V_{Ed} \le V_{Rd}^{EC8-3}$).

After calculating the shear resistances, according to the (3.7) and (3.10), it is necessary to evaluate the possibility of a premature shear rupture, that prevents a formation of a plastic hinge at the base of piers. It was verified the impossibility of forming a plastic hinge on the piers 1, 2, 9 and 10 in transversal direction. For the remaining piers, where exists capacity to form plastic hinges, it is still necessary to verify the safety to the fragile mechanisms in plastic regime.

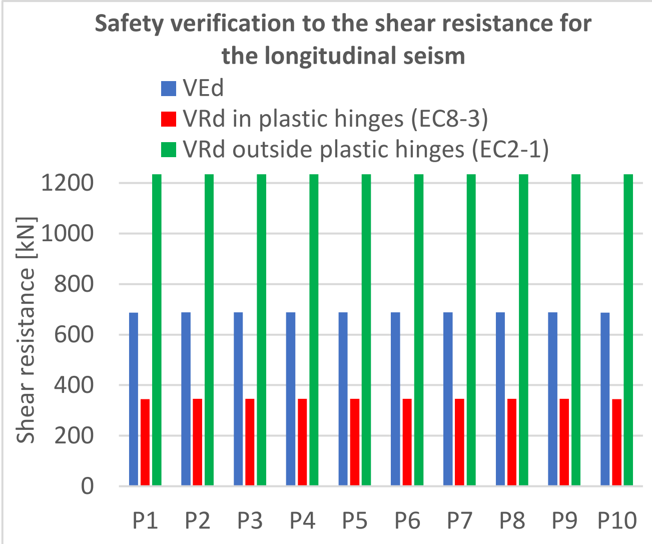


Figure 4.5. Safety verification to the shear resistance for the longitudinal seism

From the figures in this section, can be concluded that none of the piers verify the safety to plastic shear resistance.

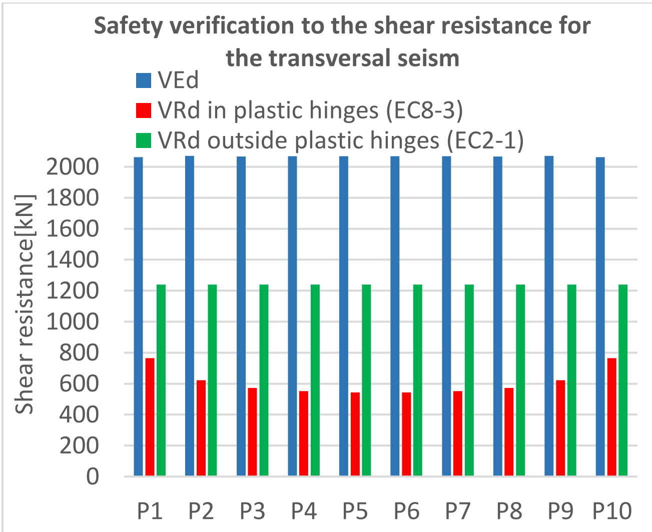


Figure 4.6. Safety verification to the shear resistance for the transversal seism

It is important to observe that transversely the piers did not verify safety outside the critical zone. This fact means that reinforcement is not only necessary in the base, but in the whole element.

5 Comparison between the structural assessment by the current and the future version of EC8-3

The future version of EC8-3 presents some differences comparatively to the current one, namely in the philosophy of calculating resistance capacities and the process to verify limit states. For this reason, it was found convenient to perform the calculation of piers capacity in accordance with the current version and compare them with those obtained previously. This analysis has as main objective realize in practical terms the significance of the changes made between the two versions of the norm. The figures bellow show that the future version provides lower capacities which reveals a more conservative philosophy.

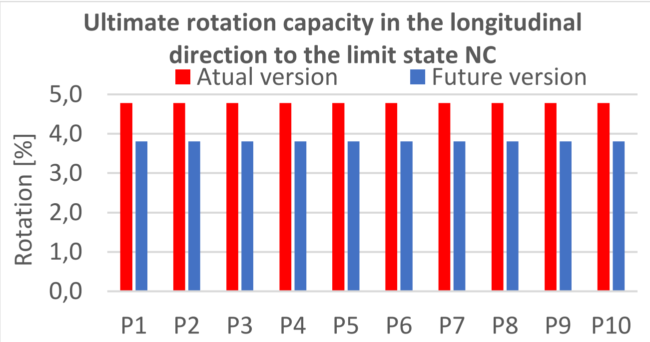


Figure 5.1. Ultimate rotation capacity of the piers in the longitudinal direction to the limit state NC

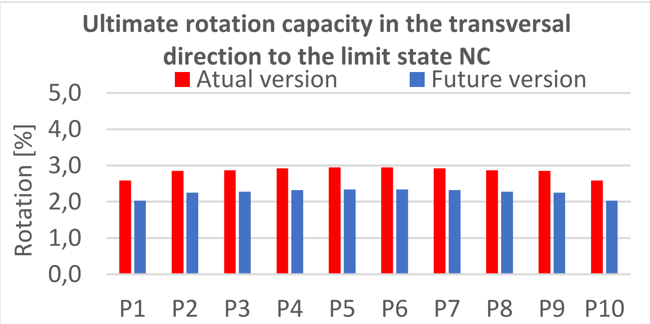


Figure 5.2. Ultimate rotation capacity of the piers in the transversal direction to the limit state NC

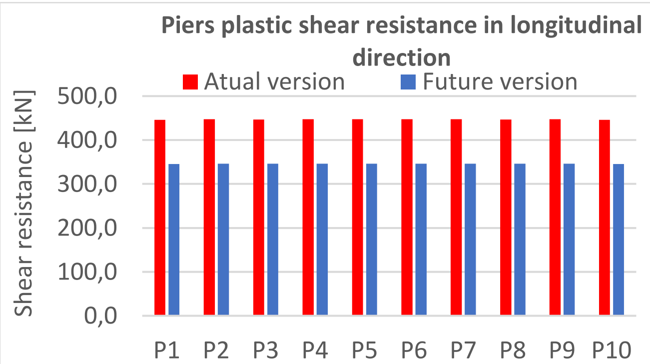


Figure 5.3. Piers plastic shear resistance in longitudinal direction

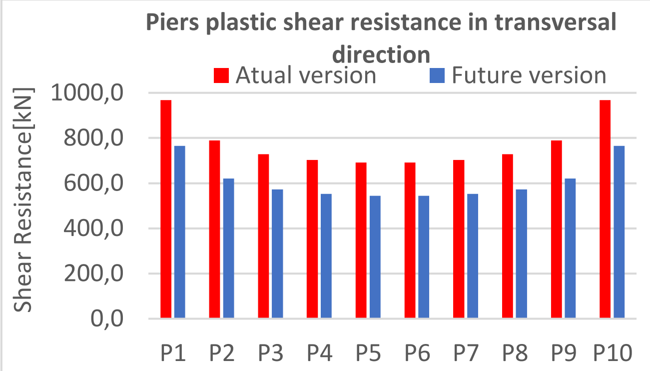


Figure 5.4. Piers plastic shear resistance in transversal direction

6 Structural intervention

As presented in the previous chapter, the case study does not verify the safety requirements, which leads to a structural intervention. For this effect, was applied seismic isolation devices in the piers top section. For this strengthening alternative EC8-3 defines that should be followed the rules of EC8-2 (7).

When a seism occurs, the viaduct behavior is similar to an inverted pendulum. So, for seismic action, , it can be said that the seismic behavior is mostly conditioned by piers inertia, height and mass of the structure. In viaducts the decks mass is the one who contributes with greater relevance to the displacements induced by the seismic action.

Seismic isolation devices promote a horizontal discontinuity, in this case between the deck and the piers, allowing the relative movement release between the two elements. In this way, it is possible to reduce the energy, reflected in stresses and displacements, that the seismic action induces in structural elements. In this type of solution, must be ensured the elastic behavior of the all elements.

It was necessary to make changes to the model used in the structural evaluation of the viaduct. Since the analysis will be performed according to EC8-2, the level of seismic action to be considered, a_g, should be defined by increasing the reference seismic action, a_{gR}, by the coefficient of importance, γ_I, which in Portugal for structures of class III is 1,45.

Still about seismic action, it is of all the convenience to consider that the isolators allow to exploit damping between 10 and 15%. For this reason, was adopted an equivalent response spectrum that presents spectral values corresponding to a damping of 10% in vibration modes which the isolation system dominates the structural behavior and damping of 5% in the remaining spectrum. show the equivalent elastic response spectrum of accelerations for seismic type 1:

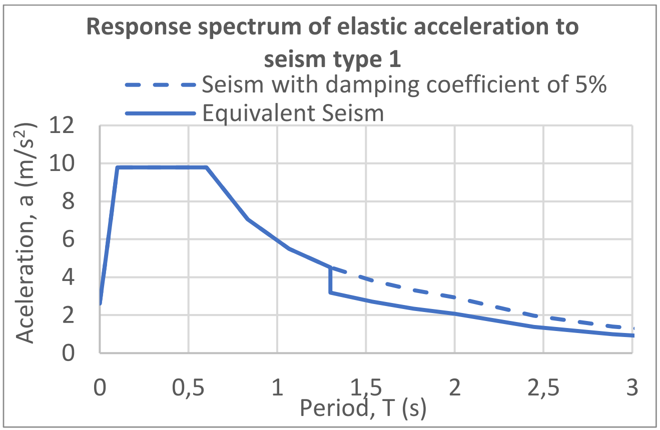


Figure 6.1. Response spectrum of elastic acceleration to seism 1

To modeling the isolation system, it was used link elements (seismic springs).

6.1 Safety verifications and selection of the seismic isolation device

The selection of a seismic isolation involves a sequence of safety checks. At first, it's necessary to select an isolator capable to support the axial forces.

Each device has a displacement limit, $d_{máx}$, that when exceded leads to collapse. So, after choosing an alternative that has capacity to support the axial loads it's necessary to evaluate if $d_{Ed} \leq d_{máx}$; d_{Ed}, must be calculated according the expression (4.3) but the displacement d_E, must be amplified by $\gamma_{IS}(= 1{,}5)$.

After a few attempts, was selected the seismic isolator SI-N 800/180 (FIP Industriale's catalogue):

$d_{máx}$ $[mm]$	N_{QP} $[kN]$	N_{ELU} $[kN]$	K_e $[kN/mm]$	K_v $[kN/mm]$
350	6790	14990	2,23	2186

Table 6.1. SI-N800/180 properties

For the seismic and fundamental combination, the maximum axial force is respectively 5808,6 and 9139,6kN, both below than the maximum values. Then, the lateral and vertical stiffness were introduced into the model links and was evaluated the device integrity as shown below.

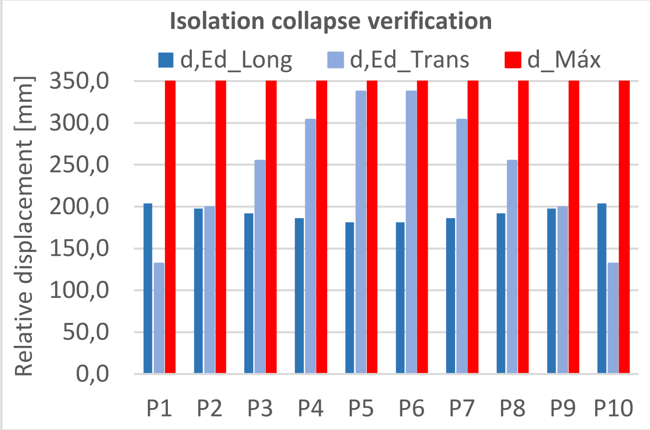


Figure 6.2 Isolation collapse verification.

After verification of isolator integrity, is necessary to analyze the structural elements safety. For this, it's necessary to verify if the piers remain in elastic phase: $M_{Ed} \leq M_y$; M_{Ed} is obtained by summing the seismic moment with the moment of second order effects. $M_{2^a ordem}$ is calculated considering the displacement at the top of the pillar, δ_{Ed} and the eccentricity promoted by the device distortion, e_{IS}. Figures above illustrate that all the piers remain in elastic phase.

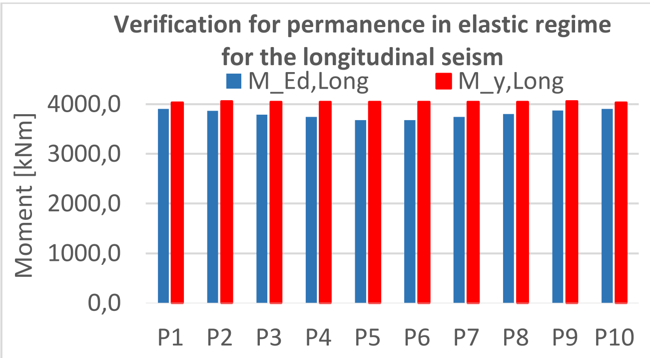


Figure 6.3. Verification for permanence in elastic regime for the longitudinal seism

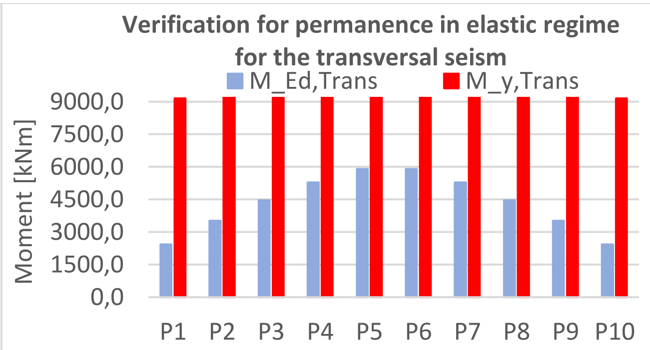


Figure 6.4. Verification for permanence in elastic regime for the transversal seism

Finally, it remains the analyze of the shear safety. For this, the value of V_{Ed} is obtained by equilibrium of M_{Ed} and should be compared with elastic shear resistance,

V_{Rd}^{EC2-1}. As illustrated bellow does not occur shear collapse in any pier.

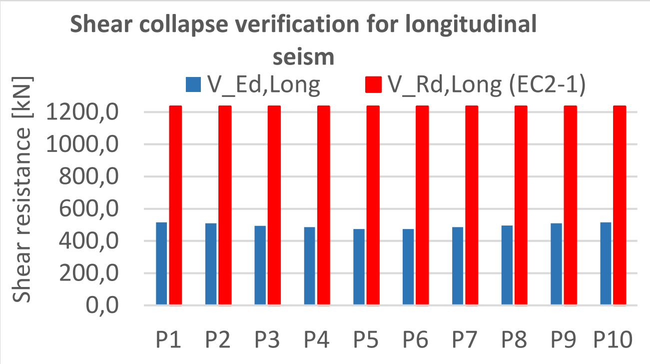


Figure 6.5. Longitudinal shear collapse verification

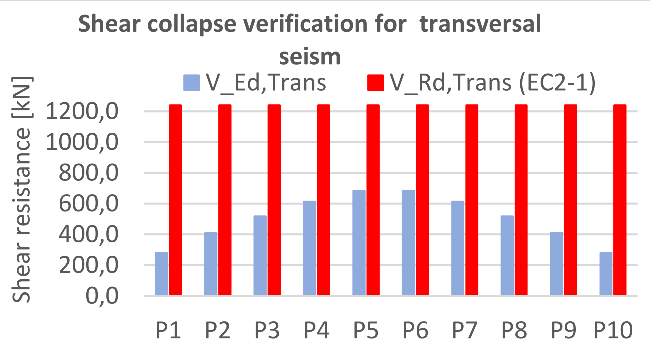


Figure 6.6. Transversal shear collapse verification

7 Conclusions

The main objective of this thesis was to present the methodology analysis proposed by the future version of EC8-3, applicable to a real case of a viaduct.

At the beginning, are presented the most common damages in bridges associated to seismic actions and explained the reasons for their occurrence.

Next, the various reinforcement alternatives were presented, and it was mentioned that only a few allow to explore the ductility of the element. This property has major relevance for the design, so it is dedicated a section to explain the favorable influence of this parameter in structures behavior.

In the evaluation of the viaduct, has been done relevance to the calculation of the effective stiffness of the piers, this property influences the displacements and stresses, reason why an uncared estimate can lead to a deficient seismic evaluation of the structure, calling into question the reliability safety verification results. Next to evaluation of the viaduct, was made a comparison between the capacities obtained according to the current and future version of EC-3, concluding that the values provided by the second are more conservative.

8 References

[1] A. Costa, Estruturas de Betão (Capítulo 10). ORION, 2013;

[2] Guerreiro L., “Engenharia Sísmica de Pontes: Danos em Pontes devido à acção sísmica”, 2010;

[3] CEN EN1992-1-1, “Eurocódigo 2: Projeto de estruturas de betão Parte 1-1: Regras gerais e regras para edifícios.” Bruxelas. 2010;

[4] CEN, EN 1998-1, “Eurocódigo 8: Projecto de estruturas para resistência aos sismos Parte 1: Regras gerais, ações sísmicas e regras para edifícios.”, 2010.

[5] CEN, EN1998-2: “Eurocode 8: Design of structures for earthquake resistance-Part 2: Bridges,”, Brussels. 2005.

[6] CEN, EN1998-3: “Eurocódigo 8: Projeto de estruturas para resistência aos sismos – Parte 3: Avaliação e reabilitação de edifícios.”, 2017;

[7] CEN, EN1998-3: “Eurocode 8: Design of structures for earthquake resistence – Part 3: Assessment and retrofitting of buildings and bridges”, Brussels. 2018-05-22;