EARTHQUAKE RESPONSE OF BRIDGE WITH UNEQUAL PIER HEIGHTS

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Abstract: Bridges are one of the most critical components of any transport infrastructure network, and their serviceability during earthquakes is vital to ensure the safety of society. To be able to overcome bridge failure, code committees started to focus on different methods to design bridges under the effect of seismic forces. One of the challenges associated with Eurocode 8 and AASHTO-LRFD, which is not effectively addressed by code committees, is to withstand the failure of irregular bridges with unequal pier heights from seismic forces.

EC8 currently uses “moment demand-to-moment capacity” ratios to insure simultaneous failure of piers on bridges supported by piers of unequal heights, while AASHTO-LRFD relies on the relative effective stiffness of the piers. These regulations are not entirely valid, especially for bridges with piers of relative height of 0.5 or less, where in the case of earthquakes, they can experience a combination of shear and flexure failure modes. In this case, the shorter piers often result in a brittle shear failure, while longer piers are most likely to fail due to flexure.

This study aims to evaluate the adequacy of EC8 and AASHTO-LRFD design procedures for regular seismic behavior, by modeling various irregular bridges using shear-critical fiber-based beam-column elements. In this work the behavior of several irregular monolithic and bearing-type bridges is investigated. In addition, different methods of regularizing the bridge performance are examined in order to balance damage, with the ultimate aim of obtaining a simultaneous or near-simultaneous failure for all piers irrespective of the different heights and failure modes experienced.

Introduction

Bridges are increasingly developed to span greater distances and serve as critical components of any infrastructure network; the safety of such structures is of great interest since their failure could lead to catastrophic disasters. Therefore bridges have to be designed in a manner that ensures any damage is controllable, and only occur in the place and time that designer intended. One of the challenges associated with Eurocode 8 is the effects of unequal pier heights in the failure of irregular bridges, which is not effectively addressed by code committees. Despite the well-known advantages of code criteria for regularity of bridges, previous studies on seismic response of bridges with unequal pier height reveals that the codes criteria may somehow guarantee simultaneous failure, however such criteria may not take into account low span to height ratio particularly ratios lower than 0.5. This is one of the challenging areas that this research project seeks to address in order to verify the accuracy of code guidelines.

In this paper, the Finite Element Analysis program ($FEAP_{pv}$) is used in order to simulate the nonlinear response of the structure [1]. The finite element model is based on a Timoshenko fiber beam-column formulation, in which the concrete constitutive laws are accounted for at the fiber level [2]. In this model, concrete is

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modelled as an orthotropic material in which the principal directions of total stresses are assumed to coincide with the principal directions of total strains, thus changing the direction continuously during the loading. The bi-directional shear mechanism at each concrete fiber is modelled by assuming the strain field of the section as given by the superposition of the classical plane section hypothesis with a uniform distribution over the cross-section for the shear strain field.

Eurocode (EC8)

The criteria for regular and irregular seismic behavior of ductile bridges addressed in EC8 focused on piers moment capacity and demands; the seismic behavior of bridges is considered regular in the horizontal direction when the following conditions are satisfied [3]:

$$\rho = \frac{r_{max}}{r_{min}} \leq \rho_{\circ} \tag{1}$$

$$r_i = q\,\frac{M_{Ed,i}}{M_{Rd,i}} \tag{2}$$

In equations (1) and (2); r_i is the local force reduction value for ductile member i, therefore r_{max} and r_{min} are the (r_i) maximum and minimum value, q is the behaviour factor, $M_{Ed,i}$ is the maximum value of design moment or moment demand of ductile member (i) at the intended plastic hinge location, and $M_{Rd,i}$ is the maximum value of design flexure resistance at the same location. If the value of ρ is less than $\rho_{\circ}$ (which is the recommended value to prevent high ductility demand on one pier) the bridge is considered as regular. The recommended value of $\rho_{\circ}$ is 2 [3].
According to EC8 if the bridge is not satisfying the regularity condition, it shall be considered as an irregular bridge, and should be either designed by using a reduced value of q [3]:

$$q_r = q\,\frac{\rho_{\circ}}{\rho} \geq 1 \tag{3}$$

or should be designed using the result from non-linear analyses in accordance with EC8 [3].

AASHTO-LRFD Code

In the AASHTO-LRFD code the regularity condition is related to effective stiffness; such condition is used for high seismic zones only. On the other hand EC8 relates the regularity to the bridge pier flexure strength. The following condition recommends a set of limited value for ratio between the effective stiffness. This effective stiffness is between two columns within a bent or any two bent within a frame [4].
In order to satisfy the condition for regularity to take place, the following conditions are applied for any two bents within a frame or any two columns within a bent [4]:

$$\frac{K_i^e}{K_j^e} \geq 0.5 \tag{4}$$

$$\frac{K_i^e\, m_j}{K_j^e m_i} \geq 0.5 \tag{5}$$

The following conditions take into account adjacent bents within a frame or adjacent columns within a bent in order to satisfy the condition for regularity to take place [4]:

$$\frac{K_i^e}{K_j^e} \geq 0.75 \tag{6}$$

$$\frac{K_i^e\, m_j}{K_j^e m_i} \geq 0.75 \tag{7}$$

This work presents the analysis of irregular bridges for both monolithic and bearing-types as described in the following section. Analysis of bridges with flexural piers was conducted in [5].

In the above equations, K_i^e and K_j^e are the smaller and larger effective bent or column stiffness, and m_i and m_j are the tributary mass of column or bent [5].

Monolithic-type Bridge Modeling

The first case study bridge is a three-span bridge, with spans 28.0+28.0+28.0m (Figs. 1-2), and the second case study bridge is a four-span bridge (Figs. 3-4), with spans 40.0+50.0+50.0+40.0, and both are simply supported by two abutments at both ends, which allows for free sliding and rotation about both horizontal axes. The bridge superstructure is assumed to be rigid. Both case studies are chosen to have monolithic-type connections between piers and the superstructure; the deck is assumed to be a cast-in-place concrete, voided slab, rigidly connected to the piers. The bridge piers consist of single circular columns with diameter of 1.2 to 2 metres and 50mm cover.

There were two different scenarios for each case study. For the first case study with three-span, the first scenario is a bridge a with pier ratio of 0.5 and pier heights of 14 and 7 metres while the second scenario is a bridge with pier height ratio of 0.3 with pier heights of 14 and 5 meters. For the second case study, the first scenario considers a bridge with pier ratio of 0.5 and heights of 10, 8 and 12 meters which makes the pier height ratio for the longer and shorter pier to be 0.5, while the pier height ratio between the longer and medium pier to be 0.75 .The second scenario assumed a ratio of 0.3 with pier heights of 16, 6 and 12 meters. The pier height ratio of the latter case study was 0.37 (between the longer and shorter pier).It should be noted that bridge piers are designed with the same cross-section and a diameter of 1.2 metres.

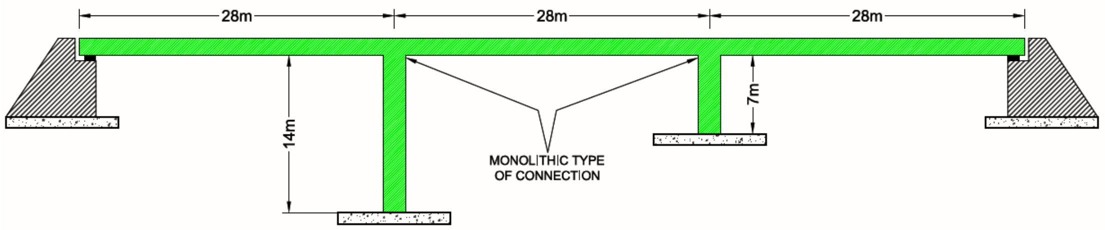

Figure 1. Bridge arrangement for the first scenario of the bridge with two piers

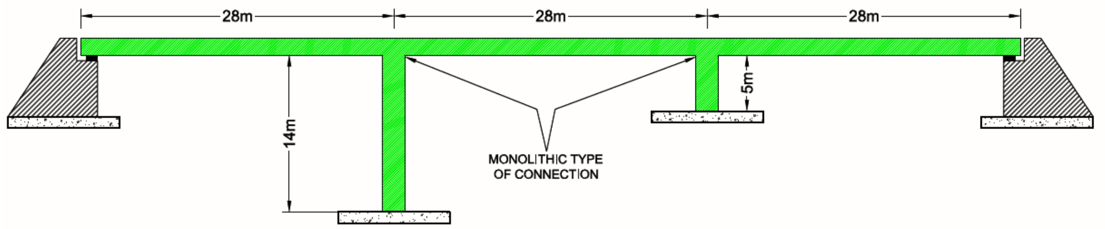

Figure 2. Bridge arrangement for the second scenario of the bridge with two piers

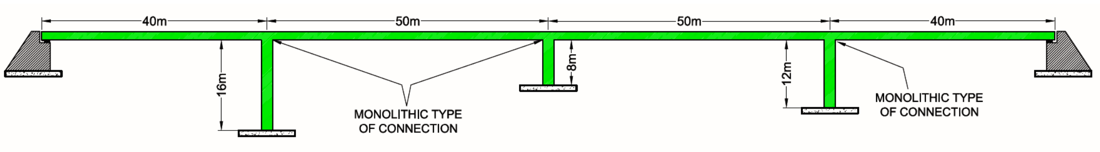

Figure 3. Bridge arrangement for the first scenario of bridge with three piers

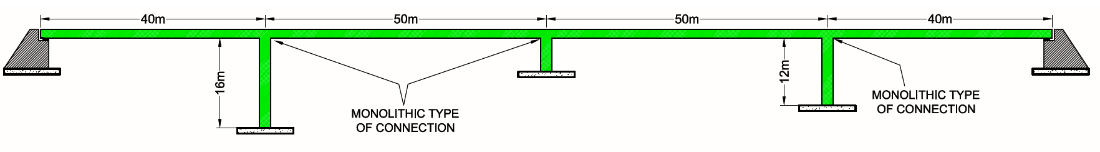

Figure 4. Bridge arrangement for the second scenario of bridge with three piers

Bearing-type Bridge Modeling

The case-study bridge used is a continuous pre-stressed concrete box girder bridge with three equal span lengths of 28.0 m, making a total length of 84.0 m. The bridge is straight in plan and is supported by two abutments at both ends. The deck rests on the piers of the bridge using bearings.

The pier heights for the first Bearing-type case-study bridge are 14.0 m and 7.0 m (pier height ratio of 0.5) and the pier heights of the second case-study bridge are 14.0 m and 3.5 m (pier height ratio of 0.25). The piers are circular in shape and sized with 0.6 m diameter, initially with one layer of 37 longitudinal bars. The transverse reinforcements initially consist of 32 mm bars, with a spacing of 400 mm. Both longitudinal and transverse reinforcements are applied to both piers of the bridge irrespective of their heights.

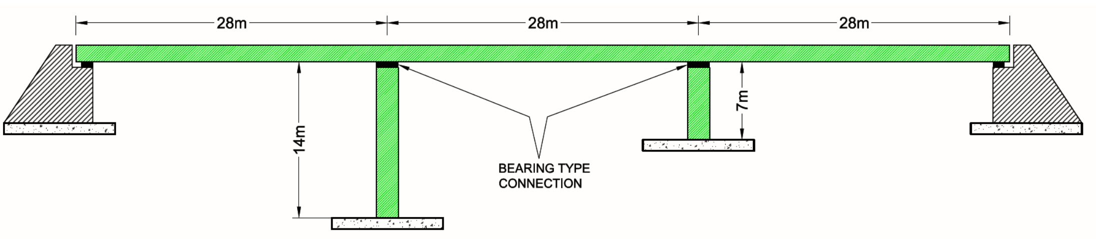

Figure 5. Bridge arrangement for the third scenario of bridge with bearing connection

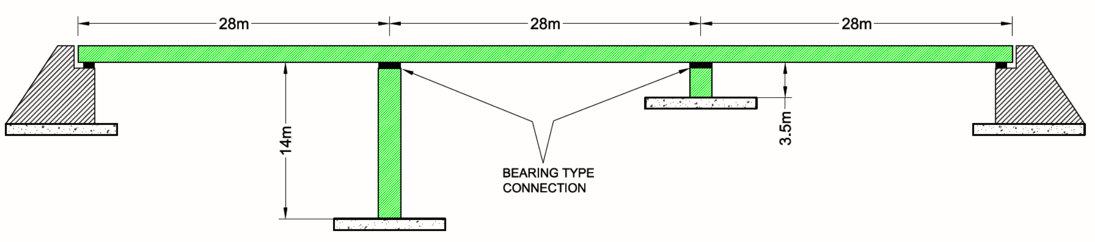

Figure 6. Bridge arrangement for the fourth scenario of bridge with bearing connection

Combination of Bearing and Monolithic-type Connection

To find another solution that could be used for regularizing pier performance, the connection between the pier and superstructure is investigated. A combination of monolithic and bearing-type connection on each bridge structure is looked at. Using the original geometry and reinforcements of the bridge highlighted in the first scenario of the first case study to show the behavior of the piers when either one of the piers are changed to a bearing connection.

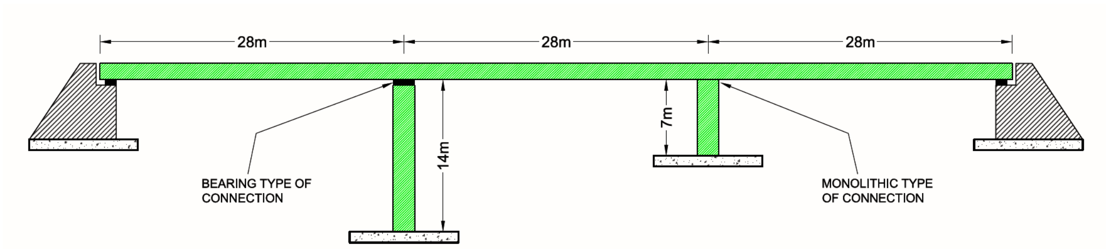

Figure 7. Bridge arrangement for the first scenario of bridge with combination of connection

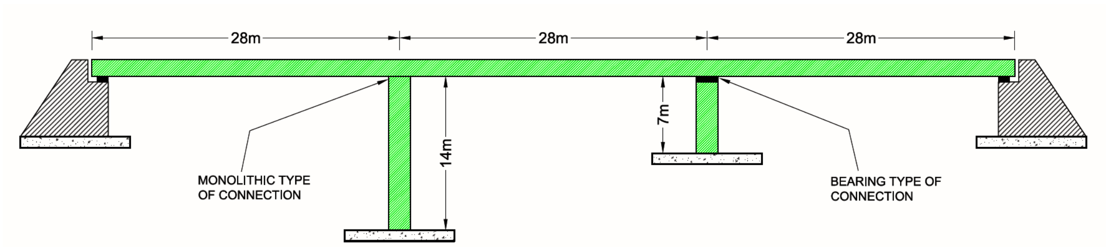

Figure 8. Bridge arrangement for the first scenario of bridge with combination of connection

Proposed criteria to verify EC8 regularity conditions

The proposed criteria to link the non-linear analysis output in order to verify EC8 regularity condition in this research is to evaluate the ρ factor, which is calculated from equation (1) in terms of lateral displacement instead of moments [6]. Therefore, the ratio between maximum and minimum displacement of piers up to the failure point can be taken as ρ_D. The ρ_D factor is generally used to judge the regularity of the bridge and the value of exactly 1 means all piers within a given bridge fail at the same point, which reveals that the bridge have a homogenous design and reaches failure simultaneously, irrespective of the difference between pier heights.

Investigating Irregularity Conditions for case study bridges

Three-span monolithic-type bridge with pier height ratio 0.5: A pushover analysis was performed for this case study with pier height ratio of 0.5 [7]; the reinforcement ratio for the short and long piers were 1.7% and 2.63%, respectively [8]. The transverse reinforcement for the initial design was assumed to be the same and consists of 16mm diameter bars with the required spacing for both piers of 100mm. As a result, the peak lateral load for the 7m and 14m piers are 1421 kN at 73.2mm and 994 kN at 292mm, respectively (Figure 9, Table 1). Therefore the displacement ratio is; $\rho_D = 4$ while, the corresponding value from Eurocode is ($\rho = 1.75$), which is less than the recommended value for $\rho = 2$ in order to guarantee regular response; however according to the results from the pushover analysis, it is clear that here the bridge is irregular.

In order to synchronize the failure, the transverse reinforcement of the short pier is changed to 10mm diameter at 245mm spacing while the transverse reinforcement of the long pier is kept the same as the initial assumption. These changes in the shorter pier affected the maximum lateral load and displacement to be more closely similar to the longer pier. The ρ_D factor reached a value of 1, which is the value corresponding to simultaneous failure of both piers.

Table 1. Detail from initial design for the bridge piers with ratio of 0.5.

	First Pier. 14m	Second Pier. 7m
Maximum Lateral Force (N)	9.9475×10^{5}	1.4217×10^{6}
Maximum Displacement (mm)	2.9224×10^{2}	7.3217×10

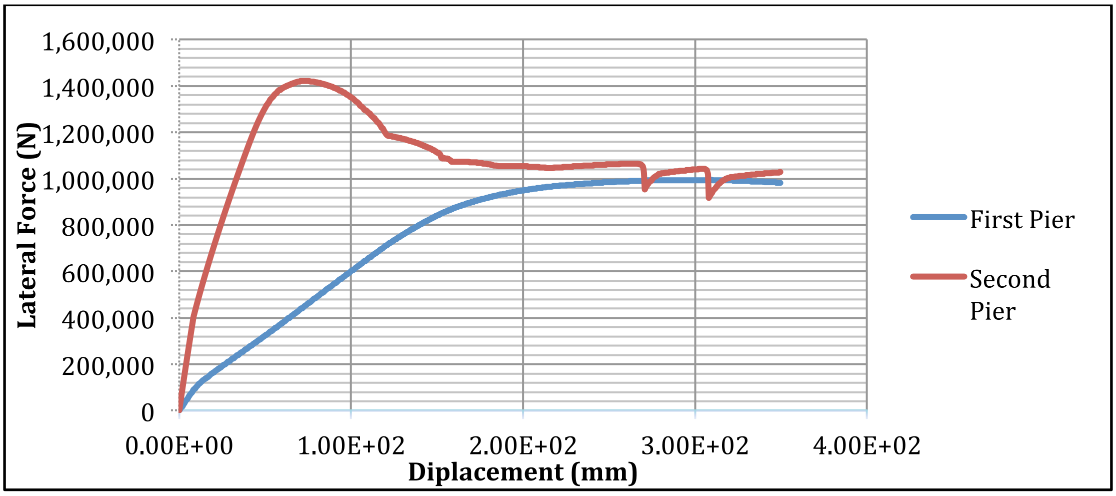

Figure 9. Shear-displacement curve showing unsynchronised failure of bridge with pier height ratio of 0.5

Three-span monolithic-type bridge with pier height ratio 0.35: The results from pushover analysis again showed unsynchronised failure of the bridge piers. In this bridge, the short pier fails with a shear capacity of 1656.4kN and 43mm displacement, while the failure occurs in the long pier with a shear capacity of 996.2kN and 290mm displacement (Table 2). These values remain nearly constant for the long pier. Consequently, the displacement ratio between the long and short pier is ($\rho_D = 6.74$), while the corresponding EC8 regularity ρ factor equals 1.59. It is clear that the flexural strength of the pier is much greater in the shorter pier. The shorter pier showed signs of a high stiffness and quickly experienced brittle shear failure as expected. The results also show that the longer pier of 14m gives a larger displacement at its maximum load, indicating that the ductility is greater in the longer pier.
In order to synchronize the failure, the transverse reinforcement of the short pier is changed to 10mm diameter at 270mm. When ρ_D is ≈1, the shear capacity of the short pier is 1232.7kN and displacement is 307.6mm, while the displacement for the long pier at failure of bridge is 319mm. The corresponding EC8 regularity ρ factor at this point equals 1.25. According to EC8, it is clear that the regularity factor ρ remains nearly constant for different transverse reinforcement ratios of the short pier. This is due to the fact that EC8 uses “moment demand-to-moment capacity” ratios to guarantee simultaneous failure of piers on bridges, while here the bridge is regularized by change in the shear capacity of the short pier to make it more ductile.

Table 2. Detail from initial design for the bridge piers with ratio of 0.35.

	First Pier. 14m	Second Pier. 5m
Maximum Lateral Force (N)	9.9621×10^{5}	1.6564×10^{6}
Maximum Displacement (mm)	2.9086×10^{2}	4.7309×10

Four-span monolithic-type bridge with pier height ratio 0.5: The finite element results again clearly showed unsynchronised failure of all piers within the bridge. The shear capacities for the pier with 16 metre, 8 metre and 12 metre heights are; 3624.7kN, 4390.4kN and 4077.4kN, respectively. Displacements at the failure points are; 307.4mm, 142.7mm and 207.2mm for the piers with 16m, 8m and 12m heights, respectively (Table 3). For this particular bridge, a transverse reinforcement with diameter of 20mm and 100mm spacing were assumed for all three piers. For the purpose of this research the stirrups were kept constant for the long pier, while gradually decreased for the medium and short piers in order to have synchronised failure for all piers. When ρ_D is ≈1 the shear capacity of the short pier is 3688kN, while the initial shear capacity was 4433kN, therefore, from the strength point of view, there is 745kN reduction in the capacity, but on the other hand, ductility is increased in the short pier.

Table 3. Detail from initial design for the bridge with 16, 8 and 12 metre pier heights.

	First Pier. 16m	Second Pier.8m	Third Pier. 12m
Maximum Lateral Force (N)	3.6247×10^6	4.3904×10^6	4.0774×10^6
Maximum Displacement (mm)	3.0742×10^2	1.4275×10^2	2.0725×10^2

Four-span monolithic-type bridge with pier height ratio 0.37: Results of the finite element analysis of the originally-designed bridge are shown in Table (4). In this case study it has been further noted that the optimum value of ($\rho_D \approx 1$), corresponds to a small stirrups value in the short pier, which decreases with increase of spacing between transverse bars. However, when ($\rho_D \approx 1$), the shear capacity of the pier reduces to 3816.9kN; the pier became more ductile and the displacement increased to 342mm.

From a pushover analysis, it has been noted that for the case study bridges, the ρ-factor pre-specified by EC8 suggested that the bridge is regular, while the corresponding ρ_D value retrieved to judge the regularity is higher than 1. The gap between ρ_D value and ρ-factor even increases as the difference between pier heights increases. In other word, despite the EC8 pre-specified ρ-factor, it does not guarantee simultaneous failure of bridges with different pier height. As the shear capacity changes in the shorter pier in order to have ($\rho_D \approx 1$), the EC8 ρ-factor stayed nearly constant.

Table 4. Detail from initial design for the bridge with 16, 6 and 12 metre pier heights.

	First Pier. 16m	Second Pier. 6m	Third Pier. 12m
Maximum Lateral Force (N)	3.6254×10^6	4.8046×10^6	4.0839×10^6
Maximum Displacement (mm)	3.0054×10^2	1.1270×10^2	2.0975×10^2

Three-span bearing-type bridge with pier height ratio 0.5: The peak lateral load for the 7 m pier is about 1330 kN and the peak lateral load for the 14 m pier is about 480 kN (Fig. 6). The transverse reinforcement of the 7m pier is changed to 6 mm bars at 400 mm spacing while the longitudinal reinforcement of the 7 m pier is adjusted to a layer of 37@24 mm bars. These changes in the shorter pier affected the maximum lateral load and displacement to be more closely similar to the longer pier.

By reducing the longitudinal reinforcement of the shorter pier to allow a smaller transverse reinforcement, the behaviour of the overall structure became more regular in the sense that both piers fail at a closer time. It is visible that the shorter pier continues to fail sooner than the longer pier. It is logical to believe that reducing the size of the transverse reinforcement or increasing the spacing will create a simultaneous failure, but it is not recommended for the transverse reinforcing bars to be reduced to any lower than 6 mm in diameter. The spacing also should not generally exceed 400 mm.

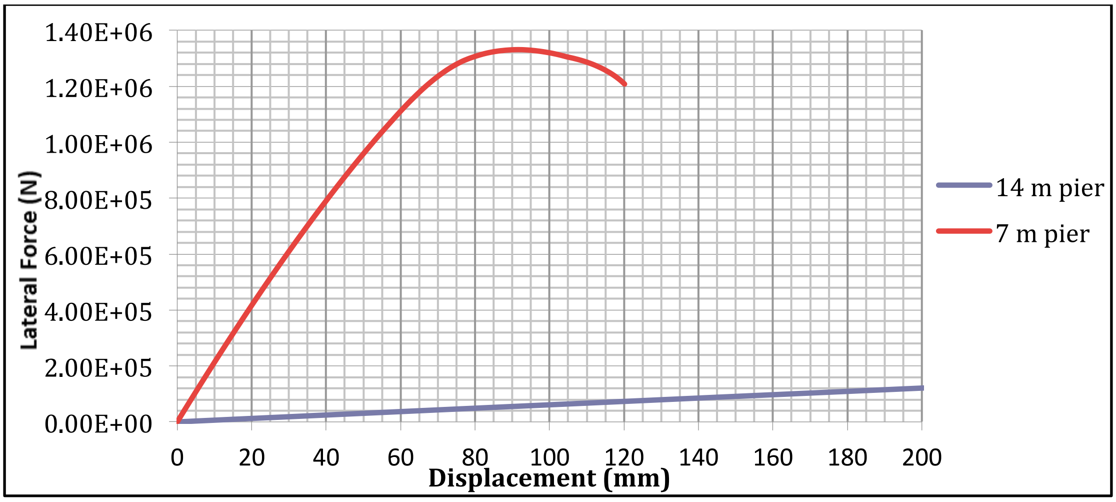

Figure 10. Shear-displacement curve showing unsynchronised failure of bearing-type bridge with pier height ratio of 0.5.

To analyze a bridge with piers of relative height of 0.25, the short pier is reduced to 3m in length. As predicted, the 3.5 m pier is extremely stiff with the original reinforcements, and both transverse and longitudinal reinforcements must be lowered in order to attempt at having the two piers fail simultaneously. In comparison to the previous models of higher pier height ratio, the performance of the pier is generally less ductile and experiences failure at a lower displacement but takes a slightly higher force before failure. It is clear that using a small transverse reinforcement of 6 mm bars at 400 mm spacing is less effective as the pier gets shorter. Therefore, it is sensible to suggest that attempting to change the reinforcement of the 14 m pier is a solution to getting the longer pier to fail at the same maximum displacement as the shorter pier. It is known from testing the previous models that reducing the longitudinal reinforcement will decrease the maximum load, and reducing the transverse reinforcement will cause the longer pier to be even more flexible resulting in a later failure.
However, when increasing and decreasing the transverse reinforcement of the longer pier from 32 mm to 40 mm and 18 mm, there are little changes to the failure point, and therefore it is not a suitable method for making both piers fail simultaneously.

Three-span bridge with combination of bearing and monolithic connection and pier height ratio of 0.5: To find another solution that could be used for regularising pier performance, the connection between the pier and superstructure is further investigated [9]. For this section of the study, a combination of monolithic and bearing-type connection on each bridge structure is looked at. Using the original geometry and reinforcements of the bridges (pier height ratio of 0.5 and 0.3) highlighted previously, the longer 14 m pier is connected monolithically to the deck. The lateral load experienced by both piers of the bridge becomes very similar, at around 1000 kN. This is a reduction in peak lateral load for the shorter pier, and a significant increase in peak lateral load for the longer pier. The displacement at which the shorter pier fails has also increased slightly while the displacement at which the longer pier fails has decreased. Creating a rigid connection in the 14 m pier therefore decreases its ductility and increases its flexural strength.
The transverse reinforcement for the 7 m pier is reduced to steel bars of 8 mm in diameter. The results shown indicates that the 7 m pier continues to shows shear behaviour and reaches yielding at around 300 kN but the failure points of both piers are near simultaneous. Unlike the bridge models previously analysed with bearing connections on top of both piers, the ultimate load of the shorter pier is lower than

the 14 m pier due to the high flexural strength generated by its rigid connection to the bridge deck. Reducing the transverse reinforcement for the shorter pier reduced the maximum load from about 1000 kN to about 850 kN.
A reduction in the shorter pier to 4.2 m, as expected, shows an increase in stiffness compared to bridges with height ratio of 0.5. Due to its short length, the increase in maximum force is now just above 1000 kN which is very similar to the maximum force of the longer pier. The maximum displacement for both piers is also similar at roughly 930 mm before failure. Both piers of the bridge show unique behaviour but have similar levels of ductility with these chosen reinforcements. The bridge appears to be regular in design and both piers will fail almost simultaneously.

Conclusion

1. It has been concluded that EC8 conditions of regularity are not entirely valid and satisfying EC8 design procedures for regular seismic behaviour does not necessarily result in synchronised failure of bridge piers with unequal height (with relative height of 0.5 or less).
2. Bridges with unequal pier with relative height of 0.5 or less have combinations of flexure and shear failure modes. In this case, the shorter piers often result in brittle shear failure and this limits its ductility capacity, while the longer piers are most likely to fail in a ductile flexural mode. As a conclusion, the shorter pier needs to be designed for higher ductility capacity in order to achieve a regularization condition.
3. Bridges with piers of unequal height could have a synchronised failure by having suitable arrangements of transverse reinforcement. Furthermore it has been noted that, by increasing the spacing between transverse bars (decrease the confinement), the transverse reinforcing steel will yield resulting in a more ductile behaviour.
4. Introducing a monolithic connection in bearing-type bridges for the long pier is advantageous as it significantly increases its flexural strength allowing a more even load distribution with the shorter pier, and therefore a more regular design for the bridge.

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