How Effective Are EC8 and Recommended AASHTO-LRFD Criteria for Regular Seismic Behavior of Ductile Bridges With Unequal Height Piers?

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SUMMARY:
The seismic design and response prediction of *irregular* bridges supported on piers of unequal heights represent a challenging problem that is yet not effectively addressed by seismic code provisions worldwide. This paper presents an investigation of the seismic response of a few schemes of a three-span case-study continuous bridge featuring two unequal piers with relative heights of 0.5, 0.64, 0.75 and 0.86, respectively. The target is to check the validity of EC8 and *newly proposed* AASHTO-LRFD provisions for regular seismic behavior of ductile bridges dimensioned per force-based design procedures. It is demonstrated that regularity criteria in both provisions fall short of providing *absolute* regular behavior of the case-study bridges. A new design criterion is hence introduced (and verified for achieving *simultaneous* collapse of piers) for bridges with piers of unequal heights but with same cross-section dimensions as may be dictated by some practical considerations.

Keywords: EC8, AASHTO-LRFD, Regular Seismic Behavior, Ductile Bridges, Unequal Height Piers.

1. INTRODUCTION

A particularly challenging problem worth tackling is the seismic design and response prediction of bridges supported on piers of unequal height – so called *irregular* bridges, a commonly adopted solution when crossing steep-sided river valleys. In case where the cross-sections of the piers are identical, the shorter piers resist higher level of inertia forces than the taller piers. The shorter piers are subjected to increased ductility demand and consequently damage tends to localize in these relatively stiffer piers. Design approaches endorsed by codes shall be hence oriented to guarantee *balanced* damage and ultimately *near-simultaneous* collapse of all piers of a given bridge at a given *extreme* target demand irrespective of their different heights. A broad study is performed throughout the present research to verify codes regulations and recommendations for seismic regularity of bridges. The investigation is performed on case-study three-span continuous concrete box girder bridges resting on two unequal piers with relative heights of 0.5, 0.64, 0.75 and 0.86, respectively. Static pushover and time history (under incrementally scaled-up *actual* records) nonlinear inelastic analyses are performed using OpenSees analysis package to check the validity of EC8 (EN 1998-2 2005) and *newly proposed* AASHTO-LRFD (Imbsen 2006) conditions for regular seismic behavior of ductile bridges dimensioned per force-based design procedures. Finally, a new design criterion is introduced and verified for regular seismic behavior of ductile bridges designed per force-based approaches. It is accordingly promoted for the case of unequal height piers but with same cross-section dimensions as may be typically dictated by some compelling aesthetical and practical considerations.

2. RESEARCH THRESHOLD AND BASIC CONCEPTS

Force-based design (FBD) methods constitute the basis of conventional seismic design in nearly all seismic codes and standards worldwide. Although current FBD is considerably improved compared to procedures used in earlier years, there are still some fundamental problems with the procedure, particularly when applied to bridges with unequal height piers. In FBD, the initial *elastic* stiffness of different piers is estimated and used to distribute the total required strength (i.e., the base shear). This step can result in undesirable and illogical results as a consequence to this elastic force distribution. First, it is recalled that the yield curvature of a concrete section is independent of the strength. Hence, the yield displacements of the piers will be proportional to H^2 regardless of the strength allocated to them. If one of the two piers of a three-span continuous bridge has a height 2.5 times that of the other, then its yield displacement will be 6.25 times that of the other. Clearly the ductility demands of the two piers will be grossly different, and assessment of a characteristic structural ductility factor, and hence of a behavior factor q (or R) will be problematic. Second, the shortest pier will be allocated much higher flexural reinforcement amount than the longer pier which will increase its elastic stiffness even further. A re-design should be thus carried out with the improved estimates of pier stiffness, which would result in a still higher percentage of total "elastic" seismic resisting force being allocated to the shorter pier. Allocating a large proportion of the total seismic design force to the short pier increases its vulnerability to shear failure. Further, the displacement capacity of heavily reinforced piers is reduced as the reinforcement ratio increases, and hence the initial stiffness approach will tend to reduce displacement capacity of the bridge as a whole.

Many codes are currently dealing with the idea of bridge seismic regularity and the issue is handled by each code from a different perspective. According to EC8 (EN 1998-2 2005) the concept of regularity of bridges focused on piers' relative strength (namely, moment capacity and demand). Bridges are considered *regular* when the following condition is satisfied:

$$\rho = r_{max} / r_{min} \leq \rho_o \tag{2.1}$$

where r_{min} = minimum (r_i) and r_{max} = maximum (r_i) for all ductile piers i; r_i is the local force reduction factor, $r_i = q\ (M_{Ed,i}/M_{Rd,i})$; q is the behavior factor; $M_{Ed,i}$ and $M_{Rd,i}$ are the maximum value of design moment (i.e., demand) and the maximum value of design flexural resistance, respectively, under the seismic load combinations at the intended location of plastic hinge of ductile pier i; ρ_o is defined as a limit to ensure that sequential yielding of the ductile piers shall not cause unacceptably high ductility demands on one pier relative to the other. Code recommended value for ρ_o is 2. One or more ductile piers may be exempted from the above calculation of r_{min} and r_{max}, if the sum of their shear contributions does not exceed 20% of the total seismic shear in the direction under consideration.

On the other hand, in the US practice [AASHTO-LRFD *newly proposed* guide specifications (Imbsen 2006; SCDOT 2008)], recommended code provisions relate regularity criteria to the piers effective stiffness and such criteria are proposed for high seismic zones. Provisions set limits to the ratio between the minimum and maximum effective stiffness of the piers of a given bridge. The value of these limits varies according to the piers position within the bridge length. For two *adjacent* piers this value equals 0.75, while for *any* two piers within the bridge (i.e., piers that are not located at two adjacent axes) this value is 0.5. These recommendations exclude the consideration of the abutments.

3. MODELING AND DESIGN OF THE CASE-STUDY BRIDGE

A case-study bridge with unequal height piers is selected to verify the aforementioned regularity criteria pre-set by codes (Guirguis 2011 and Guirguis and Mehanny 2012). The case study bridge is a continuous pre-stressed concrete box girder bridge with general arrangement and dimensions as depicted in Fig. 3.1. The bridge deck rests on two piers through fixed pot bearings, while at abutments, sliding pot bearings are installed. Piers are designed and then tried with different dimensions and heights as will be demonstrated later. A few design schemes have been executed for the case-study bridge to encompass a wide range for the ratio between the heights of the two piers. For all schemes a long pier that is 14 meters high is considered. The short pier is selected to have a height of 7, 9, 10.5 and 12 meters thus resulting in relative heights between the two unequal piers of 0.5, 0.64, 0.75 and 0.86, respectively, for investigated schemes.

The structural analysis platform OpenSees is used to determine structural response of the case-study bridges. Concrete01 uni-axial material object with degraded linear unloading/reloading stiffness and with no tensile strength is used. It considers the effect of confinement of the concrete section due to stirrups. Stress and strain limits of the confined and un-confined concrete are as per EN 1992-1-1 (2005) and are illustrated in Guirguis (2011). Accordingly, the compressive strength of the confined area is 35 MPa, the strain at reaching the maximum strength of the confined area is 3% and the ultimate strain of the confined area is 14%. Steel01 uni-axial bilinear material object with kinematic hardening is used for reinforcing bars. Its behavior is characterized by an initial elastic portion of the stress-strain relationship, with a modulus of elasticity Es = 200GPa, up to the yield stress *f*y, followed by a strain hardening region. *f*y of the reinforcing steel is 400MPa and the hardening modulus is 4MPa. Force-based elements featuring distributed plasticity are adopted to model the piers. The pre-stressed deck is modeled using elastic frame elements and uncracked section stiffness as per EN 1998-2 (2005). Piers are assumed fixed to their foundations – i.e., soil-structure interaction is ignored – and released for the moment in the longitudinal direction of the deck at their tops. Piers cross-sections are modeled as fiber sections that are divided into two distinct regions: (1) an unconfined region; and (2) a highly confined region, each represented by stress-strain properties that are function of the different levels of confinement. Abutments at both ends of the bridge are modeled as roller supports. Bridge masses are lumped at the level of the deck and are assigned in the longitudinal direction of the bridge. Lumped masses include dead loads in addition to 20% of the bridge live loads according to EC8. 5% constant modal damping is assumed for all modes.

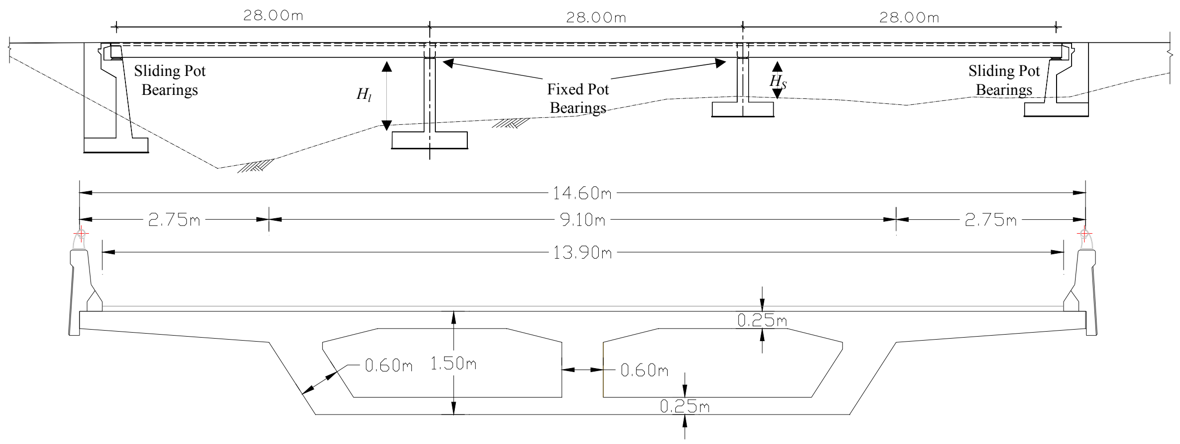

Figure 3.1. General arrangement and typical cross-section for the case-study bridge.

Dimensioning of the piers starts by pre-selecting cross-section dimensions to satisfy gravity design and appropriate slenderness ratios to avoid buckling. Consequently, a response spectrum analysis is performed for the bridge under the effect of a response spectrum (Type 1) constructed according to EN 1998-1 (2005) with ground acceleration 0.3g (assuming moderate to high seismicity), soil type B, and considering a high importance factor of 1.3. The design further assumes a behavior factor q = 3.5 for the design of piers [EN 1998-2 (2005)]. An effective inertia for the piers is taken as 50% of the pier gross inertia according to a parametric sensitivity study in Guirguis (2011). This assumption is supported by charts in Imbsen (2006) for typical bridge piers' cross-sections. The main concept followed for the piers design is to guarantee a factor of safety (FOS) against flexure of exactly 1.0 relying on the generated "axial force-bending moment" interaction diagram for each pier, and the design values retrieved from the *elastic* response spectrum analysis after being reduced by the q value. Loops of analyses are performed to design the piers (i.e., to determine piers cross-section dimensions and reinforcement) according to this *somehow optimal* (FOS =1.0) design concept. Table 3.1 shows a summary of the main dimensions of piers for different investigated schemes. It is however worth noting that design-resulting reinforcement ratios that are less than 1% (satisfying EC8 while violating AASHTO minimum reinforcement ratio) were deliberately kept as a starting point for unconstrained investigation purposes and to guarantee a theoretical design FOS of exactly 1.0. The process will subsequently generate versions of each case-study bridge by incrementally increasing these design-resulting reinforcement ratios of the long pier. The seismic response of each bridge version is hence monitored up to collapse under the effect of incrementally increased seismic loads searching for the optimal version (i.e., combination of cross-section dimensions and reinforcement ratios of unequal piers) featuring simultaneous collapse of both piers.

Table 3.1. Summary of piers characteristics for different investigated schemes optimally designed per EC8.

Ratio Between Piers' Height	Long Pier				Short Pier			
	Height (m)	Width (m)	Depth* (m)	$\rho_{steel,l}$ (%)	Height (m)	Width (m)	Depth* (m)	$\rho_{steel,s}$ (%)
0.50	14	3.50	1.85	0.48	7	3.50	1.00	1.46
0.64	14	3.50	1.85	0.40	9	3.50	1.50	1.10
0.75	14	3.50	1.85	0.50	10.5	3.50	1.60	0.96
0.85	14	3.50	1.50	0.53	12	3.50	1.50	0.94

* Dimension measured in the longitudinal direction of the bridge deck.

4. PROPOSED CRITERIA AND METHODOLOGY TO VERIFY REGULAR BEHAVIOR

Section fiber analysis (SFA) is the method used herein to determine piers' cross-section strength and stiffness. SFA of the cross-section of each bridge pier is performed under the axial force P corresponding to the seismic design combination. OpenSees generates stress-strain curves for each fiber within the cross-section for the different materials (concrete and/or steel). Moment-curvature (M-Φ) curve is also plotted from which the moment resistance, M_R, curvature resistance, Φ_R, and the effective stiffness, K_{eff}, are directly extracted. M_R is the ultimate (*maximum*) moment achieved and Φ_R is the curvature that corresponds to this ultimate moment [SCDOT 2008]. K_{eff} is also calculated from M-Φ diagram as the ratio between the moment at 75% of the maximum moment and the corresponding curvature [El-Tawil and Deierlein 1999].

Time history inelastic analysis (THIA) is hence performed to predict design forces and displacements under a given actual seismic attack. The current research adopts a few (namely *seven*) recorded ground motions that pertain to a bin characterized by Large Magnitude (LM), M>6.5, Small Distance from fault (SR), R<30km, records identified as LMSR records. This bin is taken out from the large database of records gathered and used in Medina (2002) where all specific details are reported. These records were also used in other studies [e.g., Chenouda and Ayoub 2009, Mehanny 2009, Mehanny and Ayoub 2008]. THIA is performed for all investigated permutations of pier cross-section dimensions and reinforcement ratios. Each accelerogram is scaled up *gradually* until the bridge reaches a failure mechanism according to the failure criterion introduced later in the paper. In other words, THIA loops stop once the curvature demand Φ_E retrieved from the hysteretic loops at one of the bridge piers' bases reaches the curvature resistance Φ_R determined from SFA. At this level of the scaled accelerogram the corresponding relevant straining actions at the piers' bases – including the moment demand M_E – and other deformation demands are recorded to verify codes' regularity provisions.

A set of proposed steps are accordingly activated to manipulate non-linear analysis (THIA, pushover and SFA) results in order to verify the abovementioned EC8 regularity conditions. EC8 r-value in Eqn. 2.1 is instead estimated in terms of curvatures (i.e., deformations) – as a substitute for moments (i.e., strength) – as a ratio between Φ_E and Φ_R. Φ_R corresponds to the maximum moment capacity as derived from SFA, and Φ_E is the maximum curvature demand on the ductile pier derived from M-Φ hysteresis loops (output from THIA) that typically refers to an assumed failure state. This *newly* defined r is therefore somehow an *indicator* for failure from a deformation perspective once the curvature resistance of the critical section is reached. The corresponding ρ (Eqn. 2.1) takes accordingly the notation, ρ_{Φ}, referring to deformation – instead of strength – data. On the other hand, newly proposed AASHTO seismic regulations [Imbsen 2006] put a condition on piers relative effective stiffness intended to also guarantee regular response. The effective stiffness of the bridge piers could be computed from SFA output (M-Φ curve) as previously illustrated. From the above, it could be noted that EC8 look into the regularity condition from the sole perspective of strength (through moments as per Eqn. 2.1), while *newly* proposed AASHTO provisions extend this view point to encompass both force and deformation through stiffness values. However, AASHTO regulations looked only into the *elastic* range. It is believed more effective to look at the both sides of the coin: i.e., strength and deformation but further in the *inelastic* range. A hierarchy of analyses is developed according to the following sequence to verify bridge regularity codes' provisions:

1. Elastic response spectrum analysis (ERSA) is first performed to get the straining actions in pre-selected pier cross-sections and reduce them by q. Then, interaction diagrams are generated in order to check suitable dimensions and steel reinforcement ratio that entail a FOS of exactly 1.0. The ratio of the cross-section moment capacity M_{Rd} to the moment demand M_{Ed} (retrieved from ERSA after being reduced by factor q) of 1.0 at the design axial force level distinguishes an *optimal* elastic design. This situation leads to a value of r of 3.5 for both bridge piers. This r is equal to q considered *a priori* for design. Consequently, ρ per Eqn. 2.1 equals 1.0.

2. SFA is then performed for the piers' cross-sections that result from ERSA and the previous design phase. M_R, Φ_R, and K_{eff} are hence determined for each pier as previously illustrated.

3. A series of THIAs is then performed for each case-study bridge version under incrementally scaled-up *real* ground records until failure of the first pier (according to a *curvature-based* failure criterion). The cross section maximum curvature demand Φ_E and moment demand M_E are retrieved as discussed before. r could be re-calculated according to THIA results as the ratio between Φ_E and Φ_R. ρ_Φ can be then re-computed as the ratio between r_{max} to r_{min} for the two unequal height piers.

4. The bridge longer pier reinforcement ratio is afterwards increased which results in increasing the long pier cross-section moment capacity, and hence increasing the pier cross-section design (axial-flexure) FOS. Consequently, r in EC8 sense for long pier will be reduced by the value of the increase of FOS for long pier and it will be simply classified as r_{min}. On the other hand, r_{max} is for the shorter pier as its value is still equal to 3.5 since the FOS is yet 1.0 for the short pier's cross-section with no change in its reinforcement ratio. ρ as defined per EC8 has therefore coincidentally the same numerical value of FOS retrieved for the longer pier. As an example, if FOS retrieved for the long pier is 2, r_{min} is 1.75 and r_{max} thus pertaining to the shorter pier is 3.5. Hence, ρ (= r_{max}/r_{min}) is 2 which is simply the value of FOS.

5. For each increase in FOS of the cross-section, SFA is re-performed to get new curvature and moment resistances. THIA is also re-performed for the bridge with piers' new reinforcement ratios to retrieve updated maximum curvature and moment demands for each pier at the instant of failure. The incremental increase in reinforcement ratio in all investigated schemes is gradually applied to the long pier up to a limit resulting in a *preset maximum* (for practical purposes) FOS of 5.

In the current research, the pier is considered to have reached a failure state when its curvature demand from THIA reaches its curvature capacity identified from SFA. Hence, only flexure failure is considered while shear failure and buckling are not expected to occur. Generally speaking, bridge piers are considered to have reached their curvature capacity when either of the following two cases occurs: 1) the outermost confined concrete fibers of the pier cross-section reach the maximum compressive strain limit; or 2) the steel bars of the pier cross-section reach maximum tension strain limit. Looking from a *global* perspective, the scenario considered for bridge failure in the context of the current research is as follows. Bridge short pier reaches its curvature capacity while the long pier has not yet exceeded its moment capacity. That means the plastic hinge has formed in the short pier and has already exhausted all its ductility capacity while the long pier is still able to carry extra moment (i.e., attract extra load due to redistribution). In other words, a full plastic hinge has not yet formed in the long pier. Although the long pier is still functional, this situation is considered as a failure state since the full ductility capacity of the short pier has been already fully exhausted.

5. VERIFYING CODE-DESIGNED CASE-STUDY BRIDGES FOR REGULAR BEHAVIOR

In order to validate codes' design provisions for regular seismic behavior, cross-section dimensions of the two piers along with the reinforcement ratio of the shorter pier resulting from the design step introduced above were kept constant, while the reinforcement ratio of the longer pier is gradually (incrementally) increased. Accordingly, various versions of the long pier with different FOS are generated. For each combination of reinforcement ratios for long and short piers including the code-designed versions (referring to Table 3.1) of the case-study bridges, an inelastic pushover analysis is performed to determine the ductility demand on each of the two piers at the instant of failure. Note that ρ_{Φ} = 1 reveals occurrence of "*simultaneous*" or "*synchronized*" failure of both piers irrespective of the irregularity (i.e., irrespective of the different height of piers). It has been recognized from the pushover results compiled in Guirguis (2011) that ρ (Eqn. 2.1) enforced through ERSA, and inherent to EC8 seismic design procedure, does not coincide with the pushover analysis-retrieved ρ_{Φ} values. For investigated ratios of 0.86, 0.75 and 0.64 between piers' heights, the minimum achieved value of ρ_{Φ} (≈1.0) corresponds to ρ with values of 2.2, 2.6 and 3.8, respectively. It is worth mentioning that ($\rho_{steel,l}$, $\rho_{steel,s}$) for the long and short piers for these bridge schemes are (1.8%,0.94%), (1.9%,0.96%) and (2.8%,1.1%) at these specific states, respectively. It has been also recognized that the above reported ρ values corresponding to minimum ρ_{Φ} increase as the ratio between heights of short to long piers decreases. On the other hand, for the case-study bridge with relative heights of piers of 0.5 – delineating a severe *irregular* geometry, ρ_{Φ} reaches a minimum value of 1.74 as the analysis stops at ρ equals 5 associated with $\rho_{steel,l}$ = 4.21%. It has been decided not to increase $\rho_{steel,l}$ beyond this already large value since attaining ρ_{Φ} =1 for such *largely* irregular bridge scheme with a ratio of 0.5 between piers height by further increasing the reinforcement ratio is not likely to occur except perhaps at un-practically high reinforcement ratios for the long pier. Furthermore, it could be accepted that EC8 regularity criteria may not be literally applied to the case-study bridges with $H_s/H_l \leq 0.5$ since the elastic (i.e., design) shear contribution of their longer pier is below the 20% of the total seismic shear in the direction under consideration set by EN 1998-2 (2005) – item 4.1.8(3) as an upper limit to exempt the pier from the regularity criteria prescribed by Eqn. 2.1. It is further noticed that despite respecting an EC8 pre-specified ρ value that is less than 2.0 throughout the design process, the piers with different heights do not *simultaneously* fail in the sense introduced above.

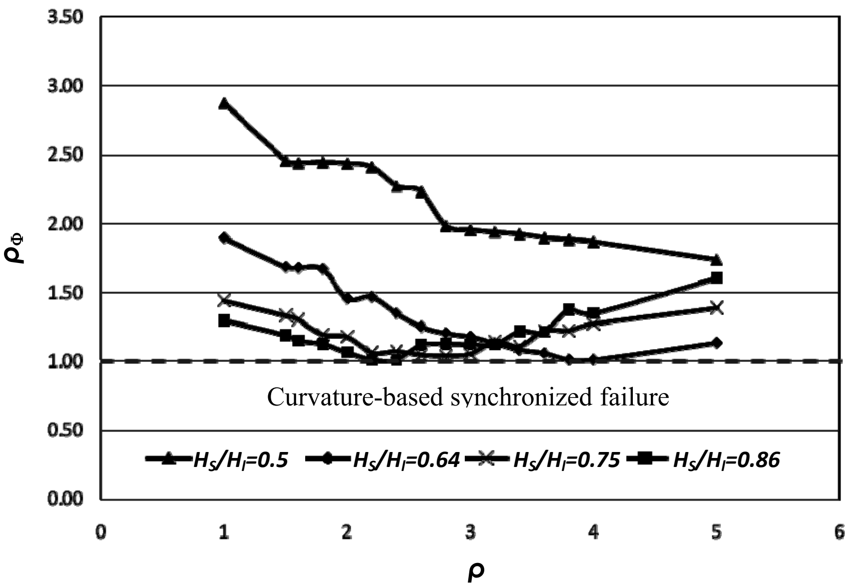

Figure 5.1. ρ versus "THIA-retrieved" ρ_{Φ} for all case-study bridges (LP89G03 record).

THIAs are hence performed for each investigated bridge scheme under the selected set of seven actual records. Accelerograms used are incrementally scaled-up until the bridge reaches failure. THIA-retrieved ρ_Φ values for different ratios between piers height are found to be less than 2 except for the investigated bridges with a ratio of 0.5 between piers height. The associated ρ as per Eqn. 2.1 ranges from 1 to 5. THIA retrieved results show that satisfying EC8 regularity condition of $\rho \leq 2$ does not necessarily result in a simultaneous failure of both unequal height piers. However, such code criterion may guarantee a *somehow regular* response with failure in both piers occurring not very distant apart except for a low – and hence severe – ratio between piers height (0.5 and below). It may be further noted that the optimal value of ρ_Φ (namely, 1) that has been retrieved from analysis corresponds to high ρ (and hence to high reinforcement ratios for the bridge long pier), and that such ρ increases with the decrease of the ratio between piers height. In more specific terms, for ratios between piers height of 0.64, 0.75 and 0.86, a ρ_Φ of 1 corresponds to ρ of about 4, 3, and 2, respectively, when consulting *average* results of all adopted ground records. Note that these results correspond to ($\rho_{steel,l}$, $\rho_{steel,s}$) for the long and short piers of about (2.95%,1.1%), (2.33%,0.96%) and (1.7%,0.94%), respectively. Fig. 5.1 shows sample curves depicting the relationship between ρ (Eqn. 2.1) and THIA-retrieved ρ_Φ for all case-study bridge versions under the effect of incrementally scaled-up LP89GO3 record up to failure.

To brief, the increase of the reinforcement of the long pier (and hence the increase of ρ) results in an increase of the effective stiffness of the pier, and accordingly its curvature capacity decreases. This will reduce gradually the difference between the curvature demand and capacity which entails a direct reduction in ρ_Φ until it scores 1 (corresponding to synchronized piers failure). Such observation retrieved from the inelastic analysis (either pushover or THIA) contradicts the conventional results of the linear elastic analysis typically used for code-design purposes where higher moments are located at the shorter pier's base thus always demanding higher reinforcement ratio for this short pier relative to the long one. It should however be noted that for bridges with small differences in heights of piers (i.e., for ratios between piers height of 0.75 and above), ρ_Φ decreases as the reinforcement ratio of the long pier increases only up to a moderate reinforcement ratio of about 1.5 to 2.5%. This is associated with failure in the short pier preceding that in the long one. ρ_Φ then reverses trend and increases with a further increase in the reinforcement ratio of the long pier. Such increase in ρ_Φ corresponds to a failure of the long pier prior to failure (in terms of curvature) of the short one. Such reversed response is different from that observed for the case-study bridges with lower ratios of the heights of the short to long piers (i.e., below 0.75) where the failure of the short pier always precedes failure of the long pier for all investigated reinforcement ratios for the long pier (up to $\rho_{steel,l}$ = 4.2% tried herein).

Finally, looking into effective stiffness, $K_{eff,s}/K_{eff,l}$ (condition for regularity as introduced by *newly proposed* AASHTO recommendations) may be more effective to verify a somehow *regular* seismic response of bridges with unequal height piers than ρ (equivalent criterion recommended by EC8). For instance, for the ratio between piers height of 0.5, 0.64, 0.75 and 0.86, ranges of analysis-retrieved ρ_Φ that correspond to $K_{eff,s}/K_{eff,l}$ larger than 0.75 (code preset lower limit for regular behavior) are approximately 1.9 to 2.2 , 1.2 to 1.5, 1.1 to 1.4 and 1.0 to 1.2, respectively [Guirguis 2011].

6. PROPOSED CRITERION FOR REGULAR BEHAVIOR OF A SPECIAL BRIDGE TYPE

In a complementary effort, a study is performed where both unequal piers of all investigated bridge schemes have same cross-section dimensions as is likely the case for architectural reasons. Firstly, cross-section dimensions for both piers and initial estimate for reinforcement ratio of each pier [Table 6.1] are determined through ERSA. Reinforcement ratio for each pier is computed in such a way that a FOS for flexure of exactly 1.0 is achieved. Secondly, a series of pushover analyses is performed and the reinforcement ratio of the bridge long pier is incrementally increased up to the state (and for a specific $\rho_{steel,l}$) at which synchronized failure occurs in both piers. Such event usually coincides with a sudden drop in the base shear curve that occurs at a certain deck displacement. As a final step, THIA is performed on the bridge with the combination of piers reinforcement ratios that caused synchronized failure during the pushover analysis in order to verify similarly reaching a *simultaneous* failure mechanism during THIA. In this particular study restrictions on the maximum and minimum reinforcement ratios set by codes have been violated to keep a theoretical perspective to the investigation.

Table 6.1. Summary of piers characteristics for bridges designed with same pier cross-section dimensions.

Ratio Between Piers' Height	Long Pier				Short Pier			
	Height (m)	Width (m)	Depth* (m)	$\rho_{steel,l}$ (%)	Height (m)	Width (m)	Depth* (m)	$\rho_{steel,s}$ (%)
0.50	14	3.50	1.50	0.10	7	3.50	1.50	1.61
0.64	14	3.50	1.50	0.21	9	3.50	1.50	1.18
0.75	14	3.50	1.50	0.35	10.5	3.50	1.50	1.00
0.85	14	3.50	1.50	0.53	12	3.50	1.50	1.00

* Dimension measured in the longitudinal direction of the bridge deck.

From the pushover analysis results compiled in the current research (Guirguis 2011) it has been observed that increasing the reinforcement ratio in the bridge long pier leads to an increase in the pier stiffness, followed by an increase in the base shear demand on that pier along with a decrease of its ductility capacity. Accordingly, ρ_{Φ} decreases with that increase in $\rho_{steel,l}$ until it reaches a value of 1.0. This declares an *optimal* design for the bridge in question where curvature demand attains curvature capacity simultaneously for both unequal height piers with same cross-section dimensions. It was noticed that ρ_{Φ} = 1.0 occurs for a ratio $\rho_{steel,l}$ / $\rho_{steel,s}$ = 18.63. Note that this "optimal" situation from a *synchronized* "flexure" failure point of view is based on an unpractical and exaggerated reinforcement ratio (30%) in the long pier that violates all relevant restrictions in design codes and that may even question the validity of the analysis results. It is however decided to present (but *skeptically* consider) this extremely large reinforcement ratio to test the applicability of the hypothesis demonstrated herein for other investigated bridge schemes. THIA is then performed for this particular case considering these achieved $\rho_{steel,l}$ and $\rho_{steel,s}$ values – with all reservations stated above – at which synchronized failure occurs for both bridge piers. For each THIA, the record is gradually scaled-up until failure occurs in both bridge piers. Average of corresponding THIA-retrieved ρ_{Φ} values for all selected (seven) ground records is approximately 1.0 (Guirguis and Mehanny 2012), which means synchronized failure of both piers. Results of THIA hence coincide with those of the pushover analysis at synchronized failure of both piers that occurs for $\rho_{steel,l}$ = 30%, while $\rho_{steel,s}$ = 1.61% (original as-designed reinforcement ratio as per Table 6.1). However, one should still keep in mind that this 30% reinforcement ratio is very theoretical and practically impossible. Finally, it is worth highlighting that from both the pushover and the time history analyses the ratio $\rho_{steel,l}$ / $\rho_{steel,s}$ at synchronized failure is 18.63 for the case-study bridge with a ratio of 0.5 between the height of its piers. $\rho_{steel,l}$ / $\rho_{steel,s}$ is found to be equal to $(H_l / H_s)^{4.2}$ or, simply, $(H_l / H_s)^4$.

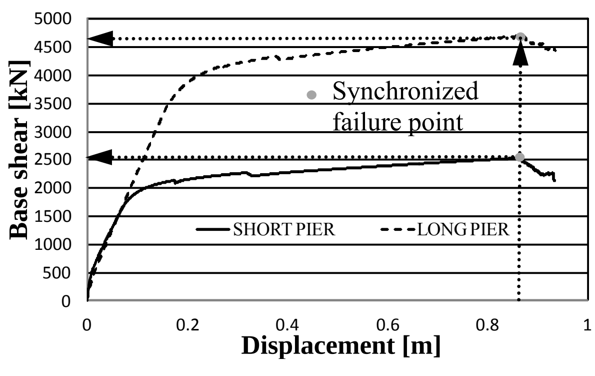

Figure 6.1. Base shear-Displacement curve showing synchronized failure of bridge piers for ratio between piers heights of 0.75 – pushover analysis results.

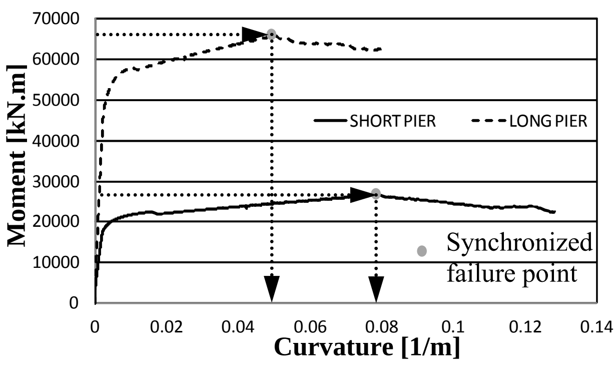

Figure 6.2. Moment-Curvature curve showing synchronized failure of bridge piers for case of ratio between piers heights of 0.75 – pushover analysis results.

Benefiting from previous results for the case-study bridge with the ratio of 0.5 between piers height, although achieved relying on unpractical $\rho_{steel,l}$ = 30%, the reinforcement ratio of the bridge long pier,

$\rho_{steel,l}$, is increased so that the ratio $\rho_{steel,l}$ / $\rho_{steel,s}$ is forced to be equal to $(H_l / H_s)^{4.2}$ for all other investigated bridge schemes. In other words, $\rho_{steel,l}$ is bumped up from values reported in Table 6.1 to assume values of 7.55%, 3.34% and 1.90% for investigated bridges with relative height between piers of 0.64, 0.75 and 0.86, respectively, while the short pier reinforcement ratio, $\rho_{steel,s}$, is kept constant as resulting from the original design (Table 6.1). Sample detailed results presented below pertain to the bridge with the ratio between piers height of 0.75. Referring to Fig. 6.1, a drop is observed in "base shear-displacement" curves of both piers at a displacement of the bridge deck of 0.87m as retrieved from pushover analysis. Verifying results at the piers cross-section level associated with this monitored drop in the pushover curve, it has been confirmed that both unequal height piers have reached a *synchronized* curvature-based failure mechanism at this specific displacement of the bridge deck. The corresponding bending moments at the base of the bridge piers – i.e., at location of highest curvature demand – are 26040 kN.m and 65630 kN.m for short and long pier, respectively. The curvature demands associated with the recorded deck displacement at *synchronized* failure of both piers are 0.082/m and 0.050/m for short and long pier, respectively, as shown in Fig. 6.2. It has been verified that these monitored demand values are equal to the curvature capacity values estimated for the piers cross-section relying on SFAs. THIA is hence performed for the above bridge with $\rho_{steel,l}$ and $\rho_{steel,s}$ of 3.34% and 1%, respectively. Fig. 6.3 gives sample THIA results for one selected record (LP89HDA) showing the relationship between Sa(T1) and the maximum retrieved curvature demand on each pier. Detailed THIA results (Guirguis and Mehanny, 2012) at the instant of *synchronized* failure of both piers for this bridge show that the average of all retrieved ρ_Φ values under the set of the seven selected records is 1.05 (i.e., ≈ 1.0). That means a *synchronized* failure of both piers occurs as expected – and as tailored – when the long pier has a reinforcement ratio, $\rho_{steel,l}$ of 3.34% and the short pier has $\rho_{steel,s}$ of 1%. Note that $\rho_{steel,l}/\rho_{steel,s}$ = 3.34 is equal to $(H_l/H_s)^{4.2}$, i.e., $(14/10.5)^{4.2}$. Same trend of results has been detected for all other investigated bridge versions thus confirming the relationship promoted herein between $(\rho_{steel,l}/\rho_{steel,s})$ and $(H_l/H_s)^{4.2}$ which ensures *synchronized* failure of both piers relying on either pushover or THIA. Comprehensive sets of results are included in Guirguis 2011.

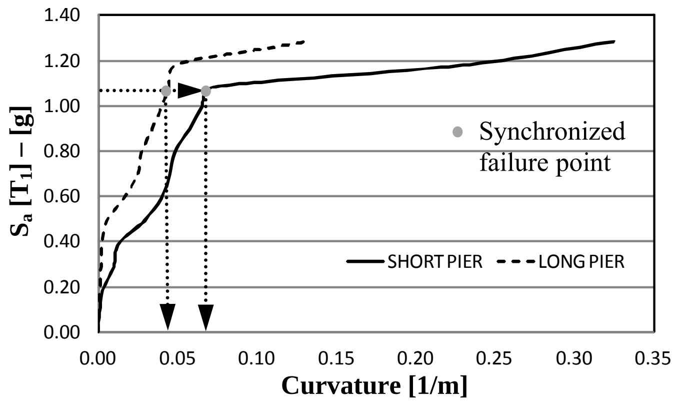

Figure 6.3. $S_a[T_1]$ versus Curvature for bridge long and short piers, case of ratio between piers heights of 0.75 (LP89HDA record).

7. CONCLUSIONS

Main conclusions of the present research are summarized along two different fronts:

1. *Investigating codes recommendations for regularity*: From a deformation perspective, *curvature-based* ρ_Φ values for different case-study bridges with different ratios between piers' height are found to be less than the value of 2 pre-specified by EC8 as the upper bound for the *strength/moment-based* "design" regularity parameter ρ (except for the case of the ratio between piers height of 0.5), but are still *generally* larger than 1 associated with synchronized failure of unequal height piers. It should be noted that ρ equals 1 corresponds to the case where the piers cross-section dimensions and reinforcement are *optimally* designed (from a strength perspective)

according to EC8 FBD recommendations. It has been shown that satisfying EC8 regularity condition of $\rho < 2$ does not necessarily result in a simultaneous failure of both unequal piers. However, it may guarantee a *somehow regular* seismic response with failure in both piers not very far apart except for a low ratio between piers height (0.5 and below). From another perspective, results obtained for the ratio $K_{eff,s}/K_{eff,l}$ (condition of regularity introduced by AASHTO) may intuitively appear to be more effective than ρ (criterion recommended by EC8) as discussed in the body of the manuscript. It has been further concluded that the reinforcement ratio for the longer pier satisfying *newly proposed* AASHTO recommendations to maintain regularity is more "compatible" with the *curvature-based* regularity criteria promoted herein than the reinforcement ratio computed for the longer pier to satisfy EC8 recommendations for regularity. The word "compatible" here means that computed ρ_Φ for bridge piers satisfying AASHTO regularity conditions is closer to 1 than computed ρ_Φ for bridge piers designed to satisfy EC8 regularity conditions.

2. *Investigating case study bridges but with piers having same cross-section dimensions*: The present investigation promotes that for these special bridges, the ratio $\rho_{steel,l} / \rho_{steel,s}$ shall be equal to H_l / H_s raised to the power of approximately 4.2. This criterion is found to cause synchronized failure (i.e., $\rho_\Phi = 1$) in both bridge piers for almost all case-study bridges. The power of 4.2 offers a reasonable value that "works well" for a wide range of unequal height piers for bridges investigated in this research. The word "works well" means leading to a *synchronized* failure of unequal piers where failure is estimated according to some deformation demand parameter (specifically *curvature*). It should however be noted that this value of 4.2 is not a "magic" value that fits all cases. The presented study nonetheless provides a methodology (rather than a "one size fits all" value) that may be followed to determine appropriate values compatible with other bridges with unequal height piers but with structural systems different from the system investigated herein. Such methodology, based on repeated nonlinear analyses rather than on a set of empirical equations, ensures getting *synchronized* failure of all unequal piers of bridges designed through conventional linear elastic analysis – and FBD procedures endorsed by modern seismic provisions – when responding in-elastically to actual severe seismic events.

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