Seismic Performance of Novel Resilient Hinges for Columns and Application on Irregular Bridges

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ABSTRACT

Bridges are important components of the transportation network that should maintain mobility and accessibility even after severe earthquakes. The current design philosophy of earthquake-resistant bridges requires the disastrous seismic energy to be dissipated in hinges that are formed in the piers, whilst the deck should remain essentially elastic. However, post-earthquake restoration of damaged piers is challenging, time-consuming, and causes traffic disruptions. In this context, this paper proposes a novel resilient hinge, which is cost-effective and has minimal damage during earthquakes. The resilient hinge is a versatile substructure that dissipates energy through the yielding of easily replaceable steel bars, thus offering rapid restoration times, whilst it is designed to have recentering capabilities, as a number of steel bars remain primarily elastic. Numerical models of single-column piers with the proposed hinge were studied and compared against conventional reinforced concrete piers to investigate the efficiency of the design. It was found that the piers with resilient hinges exhibit a significant reduction in residual drifts when compared to the ones of the conventional piers. Application of the proposed philosophy in irregular bridge models enables a more rational and even distribution of ductility requirements along the bridge piers.

keywords: bridge; resilience; hinge; pier drifts; recentering; irregular

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INTRODUCTION

Bridges are structures which are key components of the transportation system, especially during emergency planning and response. The majority of current design guidelines (AASHTO, 2012; EN 1998-2, 2005) are based on strength limits, and thus damage is allowed, whilst minimizing the risk of bridge collapse to ensure their safety when subjected to earthquake excitations. However, in recent years it has been recognized the importance of moving towards a new performance level approach, in which controlled damage is attained through a displacement-based seismic design approach (Priestley, 2000). Yet, conventional designs prevail in current code designs, and hence restoration of damaged bridge piers can be challenging.

The traditional design approach for earthquake resistant bridges considers the energy dissipation capacity of ductile reinforced concrete columns; which exhibit hysteretic dissipation at localized regions where plastic hinges are developed. Following capacity design concepts, an intended configuration of plastic hinge formation within the structural components is conceived as the collapse mechanism. Under seismic excitation, flexural hinges will efficiently dissipate energy through hysteresis, allowing for the reduction of seismic forces.

Hence, during the inelastic response of the structure, the energy dissipation capacity is highly dependent on the hysteretic dissipation at the hinges. Several models or rules of increasing complexity exist in the international literature for the representation of the hysteretic behavior of structural components (e.g. bilinear, Takeda, Ramberg Osgood, flag-shape, etc.), each of which may prove accurate for specific components, materials or systems.

It has been observed that although generally successful in avoiding collapse, this approach has usually resulted in severely damaged (**Fig. 1a**) structures during past earthquakes (EERI,

1995); whose repair is not economically feasible and total or partial demolition is generally required (Kam, et al., 2011).

This post-earthquake damage situation has raised several concerns regarding the suitability of the plastic hinge design approach (Rodgers, et al., 2015) (Pampanin, 2012). Also, the economical investment involved in important infrastructure assets like bridges has led to a growing demand for damage-free structures. Additionally, the expectations of the end-user society have increased over the years and an urgent shift from collapse-avoidance to damage-avoidance is required in order to minimize downtimes and economic losses. Hence, the achievement of resilient structures has become one of the biggest priorities for policy makers, owners and designers (Wilkinson, et al., 2014); as structures are expected to withstand large earthquakes, yet remaining functional, ensuring long life and sustainability.

In this context, new technologies are being developed to promote alternative approaches to the plastic hinge model. These strategies aim to provide alternative energy dissipation mechanisms that will reduce residual drifts and damages, while allowing cost-effective and expeditious replacements and retrofitting. The work by researchers such as Priestley (1996) introduced initial attempts of resilient components; an example being the ductile joints for precast walls, which are used in the building industry as a substitute for conventional shear walls (Priestley, 1989). In this system (Sritharan, et al., 2015), the introduction of unbonded post-tensioned tendons would allow the precast walls to undergo a rocking response over the foundation, with lift-off and subsequent re-grounding on the existing gap (**Fig. 1b**). A continuation of the controlled rocking mechanism described was developed for bridge piers. Hybrid systems (Palermo, et al., 2004) that combine self-centering capacity with supplementary damping by energy dissipaters will result in a so-called flag-shape hysteretic behavior. Amongst others, controlled rocking has been studied experimentally by Sakai & Mahin (2004) and by Ou, et al. (2007), who used post-tensioning to minimize the residual

drifts of piers. Additional work on rocking isolation using steel frames was conducted by Eatherton, et al. (2014).

Other novel design schemes make use of super-elastic materials such as shape memory alloys, whose properties make them suitable for use as dissipaters and self-centering systems. Tests carried out by Saiidi (2012) have successfully replaced traditional mild steel reinforcement by SMA bars over the plastic hinge zone, showing large reductions in residual drift response.

This paper will investigate the efficiency of a novel resilient hinge (RH), which is to show adequate energy dissipation, reducing the catastrophic effect of earthquakes on structures, while providing recentering capabilities to minimize the drifts of bridge piers. The RH is also suitable for tuning the stiffness and period through adjustments to the properties of the rebars, aiding to control bridge irregularities by distributing uniformly the seismic demands across piers and hence rationalizing the seismic design of the bridge. Additionally, the RH is to allow for ease of inspection, maintenance and replacement of damaged components.

The effectiveness of the RH should be assessed based on a whole-life cost-benefit study that considers the cost of the installation and components, the maintenance cost and the retrofitting/repair cost after an earthquake, when compared to conventional concrete design.

Indicatively, the cost for retrofitting a small size bridge pier with traditional methods was estimated (Banerjee, et al., 2014) at \$56,000, excluding the cost of bridge closure and labor.

Thus, a resilient bridge design scheme that adapts to higher earthquake requirements could provide significant cost-savings, as repair costs for earthquake-resistant piers will have a great impact on the total cost of the bridge pier.

The analyses described in the following sections showed that the RH is able to sustain 0.50g earthquakes with minimal residual pier drifts of the order of 0.02%, as opposed to larger permanent drifts of conventional concrete hinge (CH) models.

DESCRIPTION OF THE RESILIENT HINGE

The resilient hinge is used at the positions where the bending moments of the piers are maximized, i.e. at the pier top and bottom. It links cast in situ or precast pier segments to the deck and/or to the footing of spread foundations or pile caps as shown in **Fig. 2**.

The resilient hinge (RH) has three main components: (a) the top metallic shim, which is a steel bearing plate connected to the upper part of the RC column. Adequate anchorage means of the upper plate to the pier should be provided, based on capacity design principles, as this plate receives great forces that are transferred in the concrete part of the column. The top plate has a curved recess that will be supported by a steel column. This recess allows the column to rotate, but restricts the horizontal relative displacements of the column to the top plate. It also transfers the vertical axial and shear forces to the steel column, whilst it allows rocking of the pier. Friction effects between the steel column and the recess was not part of this study, however it is acknowledged that potential friction effects should be taken into account for achieving recentering capabilities. The top plate is also used to anchor the bars, whilst room is provided for inspection and replacement of the bars. The concrete column rotations are controlled by tensile forces developed within the bars. (b) The second part of the resilient hinge is the bottom bearing plate, which is a steel shim attached to the footing; it also includes the steel column, which projects upwards and supports the appropriately shaped recess formed at the top plate. The steel column transfers vertical axial and shear forces of the column, which are carried to the footing of the foundation. As for the top cover, the bottom steel bearing plate houses the bar connections that control the movement of the hinge. (c) The resilient hinge restrains the bending moments through the replaceable steel rebars, which

connect the top and bottom plates of the RH and can be made of either ordinary strength steel (OSS) or high strength steel (HSS), such as B500C rebars or low-relaxation high strength bars that can be either unstressed or partially prestressed to offer recentering. In this paper high strength prestressing steel rods of 1030MPa yield limit were considered. The rebars are placed in the perimeter of the pier cross section, hence for a given relative pier-foundation rotation, the bars have variable eccentricities with respect to the centroid axis of the pier, shown in **Fig. 2**. The relative rotations of the bottom and top plate of the RH are resisted by tensile forces only that are developed in the bars. These tensile forces comprise the bending capacity and the stiffness of the RH. Thus, both the bending capacity and the stiffness of the RH are fully controlled by the rebar design, i.e. the material, the diameter, the length and the boundary conditions of the rebars.

Possible compression of the rebars was avoided at the rebar connection to the steel bearing plate anchored to the concrete. The connection would be consistent with a bolted joint in which tensile movement is restrained, while rebar compression that would lead to possible bar buckling is curtailed.

An important feature of the RH is that for a given rotation of the RH (i.e. pier-to-foundation rotation), which corresponds to a design target displacement of the bridge, the bars at a greater distance from the bending axis are designed to yield, as the larger the eccentricity of the bar with respect the axis of the pier the larger the axial strain of the bars. On the other hand, the bars that have smaller lever arms are designed to remain elastic and hence provide recentering to the pier. Adequacy of this recentering mechanism was assessed based on the numerical models built in this research that included P-delta effects. However, it is recognized that additional prestressed high-strength rods might be required in some cases should the friction between the metallic column and the recess be significant and the P-delta effects cause large overturning moments.

Moreover, the RH arrangement considers the space needed for jacks to reposition the pier after an earthquake, allowing for recentering and elimination of permanent drifts. Permanent pier drifts are expected to be much lower, and pier shortening (Espinoza & Mahin, 2012) (and subsequent leveling) avoided.

In this paper, the design of the RH for a circular column is given, although similar designs and geometrical configurations of the RH may be used for different pier cross sections, as the main function of the RH is to transfer the loads of the pier to the foundation in a fully controllable manner.

Features and Mechanical Properties of the Resilient Hinge

Fundamental features of the resilient hinge (RH) include the rapid replacement of the yielded bars, which offer dissipation; the recentering capabilities, provided by the bars that remain elastic; and the predefined stiffness and ductility of the column for a given pier-to-foundation rotation.

The bending moment capacity of the RH is defined by the material properties, i.e. Young's modulus, strength, and geometry, i.e. diameter and length, of the rebars and their lever arms. Based on capacity design principles (Eurocode 8-1), the bending moment capacity of the RH was chosen to be at least 1.35 times smaller than the bending moment capacity of the RC column, to provide a damage-free column.

The axial and shear capacity of the RH is defined by the appropriate selection of the steel column dimensions. The design presented here aimed to provide a steel column that has larger axial capacity than the one of the reinforced concrete column. The shear capacity of the RH is attained by the inclusion of the recess, which prevents any horizontal movements of the pier base. Appropriate design of the column was conducted to ensure that the column is able to transfer axial and shear forces.

NUMERICAL MODELING OF BRIDGE PIERS WITH RESILIENT HINGES

The efficiency of the RH is investigated using numerical models. The components of the RH will be modeled using SAP2000NL ver. 17, and the results are compared to the ones of the conventional concrete hinge considered in current design codes (EN 1998-2). Maximum and residual drifts, available ductility and effective period of the individual piers and the bridge are examined to compare the performance of the RH versus the performance of conventional reinforced concrete bridge piers.

Materials and geometry

Two piers P1 and P2 will be considered in the study. Their geometry and their FE models are depicted in **Fig. 3**. The pier columns are 6m and 12m high and have a circular cross section with a diameter of 2.00m and 2.50m respectively, corresponding to typically encountered bridges.

Fig. 3 also shows the backbone curves used to model the idealized elasto-plastic behavior of the concrete plastic hinges. Both yield and ultimate rotations and moments were computed based on the bilinearized approach suggested in EC8 Annex E, which assumes zero post-elastic stiffness up to the maximum allowable rotation for concrete crushing.

The materials of the piers are C30/37 (EN 1992-1-1) for concrete, i.e. cylindrical and cubic compressive strength of 30MPa and 37MPa respectively, and B500c for reinforcing steel conforming to BS 4449:2005.

For pier P1, the model considered the 6m high column using line elements of constant cross section. Moreover, the tributary mass from the adjacent spans corresponding to the dead and superimposed dead loads, plus the mass of the pier cap, was calculated equal to 866.46tons, with the center of gravity of this mass at 1.30 m above the pier cap. Hence a total pier height of 9.3m was effectively used. The longitudinal reinforcement of the pier was 48 rebars of

diameter 32mm, corresponding to a longitudinal reinforcement ratio of 1.2%, and for transverse reinforcement 16mm spirals were used with a spacing of 100mm.

The same design approach was followed for pier P2 that had an effective height of 15.3m. The longitudinal reinforcement was 68 bars of 32mm diameter and transverse reinforcement of 16mm spirals spaced at 100mm. Details of both cross sections are shown in **Fig. 4**.

Modeling of the Conventional RC Column

The effective flexural stiffness was considered for the pier in order to account for column cracking prior to yielding in ductile members. Values are summarized in **Table 1**.

For the concrete hinge, a lumped plasticity model using a nonlinear rotational spring joint at its base was used as per Kappos, et al. (2012). The hinge rotation capacity is based on the moment-curvature diagrams for the column section, for a given axial loading. The moment-rotation curves for P1 and P2 are shown in **Fig. 5**.

The elasto-plastic behavior depicted in **Fig. 5** was based on inelastic static analysis of the reinforced concrete column, up to concrete crushing failure of the outer column fiber. Subsequently, the nonlinear dynamic time-history analysis of the column was carried out using the preceding curves and introducing hysteretic damping based on Takeda rule hysteresis type.

The confinement of C30/37 concrete has been taken into account using the Mander Confined and Unconfined models (Mander, et al., 1988) to represent the stress-strain behavior of the inner core and outer cover concrete respectively. The ductility capacity summarized in **Table 2** shows computed values above 3.

Modeling of the Resilient Hinge

The resilient hinge comprises a number of perimetric bars that have predefined lever arms, i.e. eccentricities from the axis of the pier.

Fig. 6 shows the steel column represented by a beam element pinned at its base. The horizontal radial beams are rigid arms. The rebars were modeled by tension-only nonlinear spring elements, whose force-displacement relationship is derived from the stress-strain curve depicted in **Fig. 6**. It is also noted that any potential yielding and residual elongation of the bar when the column is subjected to seismic loads was taken into account. Thus, any seismic action that causes yielding of the bars causes softening of the resilient hinge. The constitutive moment–rotation ($M-\theta$) relationship for the resilient hinge can be expressed by the equations and graphs depicted in **Fig. 6**. The elastic rotational stiffness of the concrete column part shall be considered in addition to the hinge stiffness.

Based on capacity design principles, the bending moment capacity of the RH was designed to be 1.35 times smaller than the one of the RC column. Thus the design of the bars (strength, length and diameter) was based on the above aim. The design of the bars of P1 resulted in 0.75m long bars while P2 bars were 1m long. The bars were arranged in the perimeter of the column and the specific locations of each individual bar are modeled. Aiming to limit the total number of the replaceable rebars of the resilient hinge, as means to facilitate replacements, quick restoration times and reduce labor, a diameter of 50mm of high strength steel was used. Balance between number of bars, diameter, and length is achieved by the design to provide adequate energy dissipation, maintenance and self-centering capacity of the RH.

The bars are high strength steel Y1030H with a breaking stress of 1030MPa (i.e. 2022kN for 50mm dia bars) and yielding stress 835MPa (1690kN). No prestressing of the bars was considered at this stage, even though it is recognized that prestressing with low-relaxation steel rods/bars might be required should friction effects and overturning moments be significant.

The properties of these bars are shown in **Table 3**, which are specified according to EN 10138-4 (2000) Table 1. The modulus of elasticity is 205GPa, and the maximum elongation at maximum force is $\epsilon_{uk}=3.5\%$.

The stress-strain curve for the 50mm-dia Y1030H bars is shown in **Fig. 7**.

The force-displacement diagrams for each bar were introduced in the link element properties for each model, with negligible compression stiffness and a Takeda type hysteresis model (CSI, 2014).

The bar configuration has circular pattern of 26 rebars in total for P1 and 30 rebars for P2. These are modeled by 26 and 30 nonlinear link elements, which are connected to the center of the column by means of radial beam elements of infinite stiffness, i.e. stiff zones. The rebar arrangement plan view is shown in **Fig. 8**. The distance from the rebar face to the section edge is 100mm, so the radius for the location of the rebars is 0.875m and 1.125m for P1 and P2 respectively. Measures for the protection of these bars against corrosion can be considered, as for example the RH can be sealed with a protective shell (not shown herein).

Selection of seismic input motion

Nonlinear dynamic time history analysis was performed for a set of appropriately selected accelerograms. All input motions were compatible with the elastic design peak ground acceleration and response spectrum of Eurocode 8-1. Artificial ground motions were generated using the commercial software SeismoArtif (Seismosoft, 2013). A total of seven accelerograms (EQ1 to EQ7) were used for the analysis.

A PGA of 0.50g was chosen for the study and the elastic design response spectrum shown in **Fig. 9** was computed following the guidelines from EN1998-1, for a ground type B and a 5% viscous damping ratio. This relatively high seismic action was chosen to enable the

assessment of the behavior of the proposed RH design under significant dynamic effects that introduce important rotations and thus yielding to the components of the RH.

RESULTS AND DISCUSSION

The numerical results of the conventional and unconventional columns that have either a traditional reinforced concrete design and detailing or the proposed resilient hinge detailing were compared. Bridge irregularity and potential solutions with the use of resilient hinges are also discussed in the following section.

Response of the bars of the Resilient Hinge

Fig. 10 illustrates the axial force force-time history of four rebars (#14, #17, #19 and #20) when pier 1 (P1) was subjected to the seismic motion of accelerogram EQ1. The four bars labeled as #14, #17, #19 and #20 in **Fig. 8a**, represent bars that will undergo substantial yielding (#14, #17), bars with moderate yielding (#19) and a bar that remains elastic respectively (#20); the lever arms of these bars are 0.875m, 0.655m, 0.310m and 0.105m respectively.

When the hinge rotates under the effect of bending moments, the bars that have the largest distance from the axis of the pier exhibit the largest displacements and axial loads that cause yielding of the bars. A maximum required local displacement ductility of 4.08 was obtained for bar #14 at peak response under EQ1. On the other hand bars #19 and #20 exhibited either minimal ductility requirements or remained essentially elastic.

The bars that yield provide energy dissipation and carry large part of the tensile forces (of the order of 1800kN) and bending moment of the pier, i.e. force (1800kN) times the lever arm (0.875m), when the seismic event causes maximum pier drifts (1.96%). On the other hand, the bars that remain essentially elastic (e.g. #20) have smaller forces, with minimal permanent deformations and thus have the ability to recenter the pier.

Comparison of the performance of the traditional concrete hinge to the resilient hinge

The comparison (**Fig. 11**) of the response of the reinforced concrete (RC) and the resilient hinge (RH), based on static nonlinear analysis (pushover) shows that the yield drift does not vary significantly, although the RH shows ductility capacity that is up to 23-33% larger. It is noted that the elastic-perfectly plastic behavior assumed for the RC column differs from the post-elastic stiffness of the Y1030H bars, which is accounted for in the RH modeling. From the figure it is also observed that lower lateral force for yielding and failure of the RH is obtained compared to that of the concrete hinge (CH). This will ensure the intended damage within the rebars and avoidance of damage to the reinforced concrete part of the column. The results are summarized in **Table 4**. The yielding bending moment of the CH is 1.70 and 2.08 times the yield moment in the RH, for P1 and P2 respectively.

In **Fig. 12** the drift-time histories have been superimposed for EQ1 and EQ2, showing the variation in response for the top column displacement.

The large differences in response between the RC hinge and the RH shown in **Fig. 12** evidenced that the pier behavior under seismic excitation is significantly influenced by the dynamic characteristics of the structure, i.e. the fundamental period of oscillation and the frequency content and fundamental period of the ground input motion. From **Fig. 12** it can also be observed that although the response varies, the general trend is that the piers with RH exhibit larger displacements over the conventional pier and are able to recover the deflections and exhibit negligible permanent drift after the earthquake, based on the numerical modeling. Both maximum and residual drifts have been recorded for all seven accelerograms and results are summarized in **Table 5**. The areas of the table highlighted with grey provide a direct comparison between the maximum and mean residual drifts for the RC and the RH.

On average, the circular arrangement of 26 bars for P1 will reduce the residual drift up to 96%, i.e. from 0.44% to 0.02%, with respect to the concrete conventional column, leading to negligible residual drifts. Also, the maximum drift of the pier with RH is increased by up to 25% during earthquake, due to the fact that the pier with the RH is more flexible thus offering a controllable rocking isolation scheme.

It is recognized though that large displacements of the deck might increase the possibility of pounding and unseating and thus additional measures against these effects should be considered (Shrestha, et al., 2014).

From the results for P2, it is deduced that the RH shows a greater benefit in stiff columns that experience high ductility requirements. It is observed P2 exhibits larger flexibility, whilst ductility demands are lower. In turn, damage observed is lower – compare 0.24% residual drift in P2 versus 0.44% in P1 – and the impact of the RH reduced. Nonetheless, the effect of the RH is similar to P1, as the residual drift has been again significantly reduced by up to 90%, i.e. from 0.24% to 0.02%. The peak drift of the pier with the RH was found to be larger by up to 19%. Hence, the permanent deflection is reduced more efficiently in P1, whilst the maximum deflection is larger in P2

The energy dissipation capacity can be investigated by plotting hysteresis loops of the hinge, as shown in **Fig. 13**.

Energy dissipation can be quantified based on the area enclosed within the moment-rotation curves, shown in **Fig. 13**. The hysteresis loops of the RH are bounding larger areas than the ones of the CH, but they are also shorter in size and the CH has larger values for the yielding moment and the loading stiffness. The values for P1 show a total energy dissipation of 38713kJ for the RC (EQ1), whereas the RH computed value was 29695kJ, indicating therefore a decrease in energy dissipated of approximately 23%, which is an acknowledged

effect in rocking isolation of piers (Rodgers, et al., 2015). In general, the pinched shape of the RH, with the loops trying to pass always through the axis origin, is a good indication of the self-centering low-damage ability of the RH. With regard to response of P2, the hysteresis loops are thinner, as the ductility demand is lower. Again, a reduction of the dissipated energy of 34% was observed in comparison to the traditional RC column, as values computed during EQ1 were 37430kJ and 24760kJ for CH and RH respectively. Nevertheless, the same pinching loops of the RH are observed compared to the slenderer loops which include damage to the CH.

Based on the results from **Table 5**, a summary of the average ductility demand and effective period of vibration is shown in **Table 6**. The effective period of vibration at maximum ductility is computed as opposed to the elastic period, based on the equations 6-4 and 6-10 outlined in FEMA 440 (2005). The RH is more flexible than the CH, with a fundamental effective period for the expected level of drift around 46% and 56% larger than the CH on average. Moreover, ductility demands also increase on average by 13% and 29%, what indicates that the resilient hinge is expected to exhibit a more ductile behavior than the plastic hinge of the concrete.

RATIONALIZATION OF PIER DESIGN IN IRREGULAR BRIDGES BASED ON ADJUSTMENTS OF THE RESILIENT HINGE

Irregular bridges can present asymmetric configurations or, more commonly, shorter piers at the center spans. This configuration is expected to concentrate the seismic demand on the short and stiff intermediate piers, potentially leading to poor performance and premature failure.

A multi-span bridge model which considered an irregular pier height distribution was analyzed. The resilient hinge rationale was applied to examine the efficiency of the RH in

reducing the concentration of ductility demands on short and stiff piers. The bridge models incorporate two effects that are usually challenging during the seismic design and analysis i.e. the superstructure continuity and variable pier stiffnesses, due to significantly different heights. In continuous superstructures, for a given uniform transverse displacement of the deck, shorter piers at the center attract larger base shear.

In this context, three models have been analyzed to investigate the possible advantages of altering RH parameters so that the global response of the bridge produces global ductility demand in accordance to local ductility demands at each pier. The bridge configuration studied is depicted in **Fig. 14**:

- CH: conventional concrete hinge, as for the single-pier analysis.
- RH1: resilient hinge, as for the single-pier analysis.
- RH2: resilient hinge design tuned based on engineering judgment. Center pier includes a 52% increase in bars stiffness; end piers consider 58% more flexible bars. This variation of RH stiffness was achieved by modifying the length of the bars for each pier hinge. This new bar arrangement was selected as an optimum solution to improve ductility demand uniformity across piers.

The mean transverse displacements of the deck for the THA of the seven EQs are depicted in **Fig. 15**.

The analyses showed that the RH models have larger displacements than the ones with CH. Also, RH1 and RH2 models show approximately the same deflected shape, with the RH2 (tuned design) exhibiting slightly larger deflection in P1 and smaller for P2 than RH1 results. Given the uneven distribution of pier height, much larger drifts are to be expected in center pier, considering that the deflected shape is curved in plan view, but pier heights are inverted with respect to a regular case.

Fig. 16a shows that drift at the center pier is 2.30%, 3.18% and 3.16% for the CH, RH1 and RH2 respectively. This represents 2.21, 2.22 and 2.08 times the drift at the end piers for each case; i.e., pier drifts are more than double the ones of the central pier. This drift concentration will in turn lead to ductility demand concentrations, as evidenced by **Fig. 16b**, in which the CH and RH1 show a well-defined peak in local ductility. In particular, ductility demand at the center pier for the CH is 2.81 times larger than for the end pier, which exhibits a mostly elastic behavior. For the RH1 the ratio is improved to 1.90 times larger, but still there is large concentration of ductility demand.

Nevertheless, the tuning carried out for RH2 shows that a more rationalized and uniform ductility behavior can be obtained, as evidenced in the graph which shows that ductility variation across piers remains within 20% difference. Hence, it has been proved that varying certain parameters of the RH design –particularly rebar length – a more rationalized seismic design of irregular bridges is possible, as well as adjustment of the dynamic properties of the bridge, in case larger seismic loads are prescribed by codes in the future.

CONCLUSIONS

A series of numerical models were analyzed to investigate the seismic performance of bridge piers designed with innovative resilient hinges (RH). These models were compared to monolithic concrete piers designed based on the traditional plastic hinge (CH) concept, in order to evaluate their performances in terms of residual drifts, ductility requirements and energy dissipation. Two circular reinforced concrete piers were analyzed, having typical heights (9.3 and 15.3 meters) and diameters (2.00m and 2.50m). Nonlinear dynamic time history analysis was carried out based on seven synthetic earthquake records compatible to the Eurocode elastic response spectrum. The study came up with the following conclusions.

Analyses of seven earthquake excitations showed that the maximum drift of the RH is larger by 25% (P1) and 18% (P2) on average, in comparison to the drift of the CH, due to the increased flexibility of piers with RH and due to lower energy dissipation capacity of the RH. However, the residual drifts of the RH were found to be reduced on average by 93% in comparison to the residual drifts of the CH, which indicates that negligible permanent drifts are to be expected in piers designed with the proposed RH.

The aforementioned reduction of the residual drift was achieved by the recentering capabilities of the RH. Specifically, the rebar arrangement led certain bars to remain elastic and therefore recovering its original position and self-centering the pier. This was also evidenced by the hysteretic loops at the base of the column with RH, which showed a pinched shape, passing symmetrically through the moment-rotation axis origin. Additional means for recentering can be provided on the basis of low-relaxation prestressing bars that will compensate for potential residual drifts due to large overturning moments and/or friction effects within the RH recess.

Effective fundamental periods of vibration for the piers were also computed, showing that the flexibility of the RH ($T_{eff}=2.08s$ and $2.35s$ for P1 and P2) was greater than the CH ($T_{eff}=1.42s$ and $1.51s$ for P1 and P2), therefore increasing the expected maximum drifts, but providing means of base isolation, as the period was shifted. Additionally, versatility of the stiffness and capacity of the RH is provided by changing the design of the bars, which essentially control the rocking of the pier.

Analyses on continuous bridges showed that, by tuning the RH stiffness, a uniform seismic demand across piers was achieved and thus reducing the differences in ductility demands for the different piers. The rationalized design succeeded in reducing the ratio of maximum to minimum ductility demands across piers by 57% on average, when compared to the conventional RC bridge case.

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520 **Table 1.** Effective stiffness computation for cracked column

Pier	$I_{\text{cracked}} \text{ (m}^4\text{)}$	$I_{\text{gross}} \text{ (m}^4\text{)}$	$I_{\text{eff}} = 0.08I_{\text{gross}} + I_{\text{cracked}} \text{ (m}^4\text{)}$	$m = I_{\text{eff}}/I_{\text{gross}}$
P1	0.233	0.775	0.295	0.380
P2	0.538	1.893	0.689	0.364

521

522 **Table 2.** Ductility capacity for the concrete hinge

Pier	Yield Drift, D_y (%)	Ultimate Drift, D_u (%)	Ductility $\mu_c = D_u/D_y$
P1	0.63	2.29	3.64
P2	0.80	2.81	3.53

523

524 **Table 3.** Y1030H steel rebars properties to EN 10138-4

Steel Name	Diameter d (mm)	Tensile Strength R_m (MPa)	Cross- sectional area S_n (mm ²)	Characteristic value of maximum force F_m (kN)	Characteristic value of 0.1% proof force $F_{p0.1}$ (kN)
Y1030H	50	1030	1964	2022	1640

525

526 **Table 4.** Comparison of the available ductility and yield forces/moments for the conventional
527 reinforced concrete pier and the pier with the resilient hinge

	P1		P2	
	concrete hinge, $D=2.00\text{m}$	resilient hinge, 26 bars	concrete hinge, $D=2.50\text{m}$	resilient hinge, 30 bars
drift at yield D_y (%)	0.63	0.70	0.80	0.73
ultimate drift, D_u (%)	2.29	3.12	2.81	3.42
ductility capacity $\mu_c = D_u/D_y$	3.64	4.49	3.53	4.69
yield moment, M_y (kN.m)	19,754	11,639	35,547	17,104
base shear, V_y (kN)	2,124	1,251	2,323	1,118

Table 5. Comparison of the maximum and residual drifts between the conventional and the resilient pier designs

P1, h=9.30m						
	concrete hinge, d=2.00m			resilient hinge, circular pattern, 26 bars		
	residual drift (%)	maximum drift (%)	residual/maximum drift ratio	residual drift (%)	maximum drift (%)	residual/maximum drift ratio
EQ1	0.26	1.59	0.16	0.01	1.96	0.00
EQ2	0.39	1.62	0.24	0.04	2.77	0.02
EQ3	0.44	1.79	0.25	0.01	1.92	0.00
EQ4	0.18	1.50	0.12	0.02	2.34	0.01
EQ5	0.79	2.32	0.34	0.01	1.90	0.00
EQ6	0.41	1.82	0.23	0.05	2.63	0.02
EQ7	0.60	1.86	0.32	0.00	2.12	0.00
Max Value	0.79	2.32	0.34	0.05	2.77	0.02
Mean Value	0.44	1.79	0.25	0.02	2.23	0.01

P2, h=15.30m						
	concrete hinge, d=2.50m			resilient hinge, circular pattern, 30 bars		
	residual drift (%)	maximum drift (%)	residual/maximum drift ratio	residual drift (%)	maximum drift (%)	residual/maximum drift ratio
EQ1	0.24	1.40	0.17	0.02	1.59	0.01
EQ2	0.34	1.56	0.22	0.05	1.97	0.03
EQ3	0.01	1.33	0.01	0.03	1.75	0.02
EQ4	0.33	1.48	0.23	0.01	1.66	0.01
EQ5	0.26	1.44	0.18	0.01	1.66	0.01
EQ6	0.29	1.46	0.20	0.03	1.70	0.02
EQ7	0.18	1.46	0.12	0.02	1.64	0.01
Max Value	0.34	1.56	0.22	0.05	1.97	0.03
Mean Value	0.24	1.45	0.16	0.02	1.71	0.01

535 **Table 6.** Average ductility demand and effective period of the pier from dynamic THA

	P1		P2	
	concrete hinge, D=2.00M	resilient hinge, circular Pattern, 26 bars	concrete hinge, D=2.50M	resilient hinge, circular Pattern, 30 bars
peak drift, D_{peak} (%)	1.79	2.23	1.45	1.71
ductility demand, μ_d	2.84	3.20	1.81	2.34
effective period, T_{eff} (sec)	1.42	2.08	1.51	2.35

536

a)

b)

a)

b)