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DESIGN FOR SEISMIC UPGRADING OF EXISTING RC FRAMES BY FRICTION DAMPERS

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SUMMARY: *Nowadays, many buildings with RC framed structure need to be seismically upgraded. The insertion of steel braces equipped with friction dampers within the framed structure is a promising seismic upgrading technique. In fact, steel braces and friction dampers reduce the storey drift demand providing additional lateral stiffness and energy dissipation. Furthermore, friction dampers cap the forces transmitted by braces avoiding that the upgrading system overload the existing structure. In this paper a design procedure of the bracing-friction damper system is formulated. The design procedure is applied to a case study frame considering different combinations of the design parameters. The analysis of the seismic response of the bare and rehabilitated frames provides information on the effectiveness of the upgrading technique and proper setting of the design parameters.*

KEYWORDS: Existing buildings, RC framed structures, seismic upgrading, friction dampers, design procedure.

1 Introduction

Many existing buildings with RC framed structure were designed without considering seismic provisions. In order to mitigate the seismic response of buildings and prevent the structural collapse, one of the most practical methods is the introduction of passive control devices [Constantinou and Symas, 1993, Soong and Dargush, 1997]. Passive energy dissipaters are able to reduce the plastic deformation of structural members by absorbing part of the input energy. Passive energy absorbing devices can be permanent or disposable: devices belonging to the first type remain permanently in the structure, even though they might need some readjustment after earthquake action, while those belonging to the second type need to be replaced after a certain number of earthquakes [Monir and Zeynali, 2013]. Friction dampers are permanent devices, which dissipate seismic energy by friction mechanisms developed on sliding shear surfaces. The slip resistance of such devices depends on the bolt preload force and the roughness of the sliding surface, while the maximum slip distance is determined by the length of the slotted holes [Grigorian *et al*., 1993]. A type of steel braces equipped with frictional devices was proposed by Pall [Pall and Marsh, 1982]. They consist of series of steel plates clamped together and treated to get a reliable friction behaviour. The friction joint is endowed with slotted holes, which let the steel plates slip at a predetermined load. Sliding can activate when the brace is both in tension and compression, provided that the brace is designed not to buckle in compression up to the slip load value. The slip load can be tuned to achieve an optimum seismic response of the structure. This type of device was used in the past in Canada for new buildings and seismic retrofit of existing steel structures [Chang *et al*., 2006, Verganelakis and Pall, 2004, Tirca *et al*., 2007]. Monir and Zeynal proposed a modified friction damper applied at the intersection of X-shaped diagonal braces, having friction hinges with only rotational movements [Monir and Zeynali, 2013]. Nastri *et al.* [Nastri *et al.,* 2019]

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suggested the insertion of friction dampers into beam-to-column joints to optimize the seismic performance of steel Moment Resisting Frame. Hu and Noh [Hu and Noh, 2015], instead, proposed a self-centring friction damper to reduce the residual interstorey drifts that are likely to occur when conventional friction dampers are used.

The introduction of energy dissipation devices, such as friction dampers, buckling restrained braces or steel plate shear walls, is a valid retrofit approach to improve the seismic response also of existing r.c. buildings, as demonstrated by numerical analyses or practical applications already realized [Totter *et al.*, 2018; De Domenico *et al.*, 2019]. Indeed, these devices act as energy fuses and thus protect the structural members from the seismic action. In the past, several researches investigated on optimisation techniques to find the optimal location and quantity of dampers to be introduced in the structure [Singh and Moreschi, 2001, Zhang and Soong, 1992] and to study the effect of supplemental damping on seismic response of structures [Lin and Chopra, 2001]. To provide a cost-effective retrofit design of steel structures, Tirca *et al.* [Tirca *et al.*, 2010] presented a design procedure based on minimising the difference between the total energy input and the energy dissipated by friction. Further design applications are provided by Pall and Pall [Pall and Pall, 2004] following the guidelines of FEMA365/367. The optimal slip load and the best placement of friction devices are investigated by Lee [Lee, 2008] so that the considered performance indexes, such as interstorey drifts or acceleration, are minimized based on numerical analyses.

Based on this background, embedding steel braces equipped with friction dampers in RC framed structures is expected to be an effective technique for seismic upgrading of existing buildings. Indeed, a bracing-friction damper system, if properly sized, can provide the RC frame with additional lateral stiffness and dissipation capacity. Furthermore, it can avoid drift concentration at a few storeys and promote a favourable and dissipative collapse mechanism. Finally, the heightwise distribution of the size of steel braces and friction dampers can be optimized, so that the drift demand is determined at every storey according to the drift capacity of the RC frame.

In this paper, a displacement-based design method for seismic upgrading of existing RC frames by steel braces and friction dampers is presented. According to this method, the design is controlled by the design storey drift Δ_d and the storey drift Δ_{act} corresponding to the activation of the friction dampers. First, the cross section of braces is determined at each storey so that the drift demand of the upgraded frame is lower than the drift capacity of the bare frame at a predetermined limit state. Second, the force corresponding to the activation of the friction damper and the maximum stroke are determined. As an example, the proposed design method is applied to retrofit a frame that does not satisfy the minimum requirements stipulated in EuroCode 8 (EC8) [CEN, 2004] for RC framed structures in occurrence of strong ground motions. The case study is a RC frame designed to sustain gravity loads only according to the provisions stipulated in the old Italian building code. The design procedure is applied considering different criteria to assign the values of the activation drift Δ_{act}. The effectiveness of the upgrading technique should be validated by refined numerical models and analyses [Lima *et al.*, 2017; Montuori *et al.*, 2019]. Hence, fibre modelling and nonlinear dynamic analysis are adopted to simulate the seismic response of the upgraded frame and the obtained seismic performance is compared to that of the bare RC frame. The results are summarised in terms of storey drift demand and storey drift demand-to-capacity ratio. The comparison of the obtained results in terms of size of the upgrading system and seismic performance of the upgraded frame shows the best criterion to assign Δ_{act}.

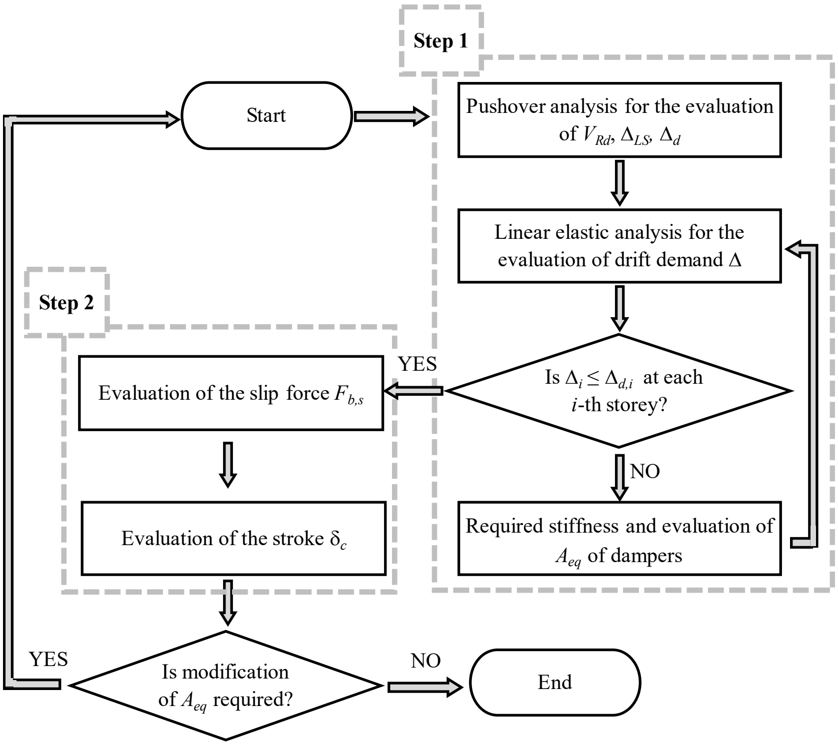


Figure 1 – Flow chart of the design procedure

2 The design method

The bracing-friction damper system used for seismic upgrading consists of a steel brace connected in series with a friction damper (for instance, the one presented in [Pall and Pall, 1996]). The design method should provide, at each storey of the frame to be upgraded, slip load and stroke of the friction damper, and axial stiffness of the brace. The proposed design method is iterative and consists of two main steps: Step 1 yields to the determination of the axial stiffness of the bracing-friction damper system $k_{b,ax}$, while Step 2 leads to the evaluation of the slip load $F_{b,s}$ (Figure 1).

In the first step, the axial stiffness of bracings is evaluated so that the drift demand of the RC frame Δ is reduced below the design storey drift Δ_d. This latter is assumed equal to a prefixed rate of the drift capacity Δ_{LS}, which is the storey drift corresponding to the target Limit State. The drift capacity is evaluated at both the Near Collapse (NC) and Significant Damage (SD) limit states in terms of chord rotation capacity, according to the provisions of EC8 [CEN, 2005]. The value of Δ_{LS} depends on characteristics of the RC frame (mechanical properties of materials, size and detailing of members, etc.), but also on the axial force ratio of the columns, and it is determined by a pushover analysis. In particular, at each storey, the drift capacity is evaluated at each step of the pushover analysis as the column chord rotation capacity times the interstorey height and is compared to the corresponding drift demand. When the drift demand at one storey equals the capacity, the pushover analysis is stopped and the drift of each storey is assumed as the corresponding Δ_{LS}. Then, the storey lateral strength V_{Rd} is determined by the pushover analysis at the attainment of the target limit state and is evaluated as the summation of the lateral strength of the bare RC frame and the lateral strength provided by the inserted braces. At the first iteration, the lateral strength is given only by the existing frame. The evaluation of the drift demand Δ is based on the equal displacement rule. Particularly, the drift Δ_{el} is first determined at each storey by an elastic analysis with PGA corresponding to the assumed seismic excitation level, and then modified following the rule by Vidic *et al.* [Vidic *et al.,* 1994]. Further, the drift Δ_{el} is modified to take into account that the contribution to the horizontal drift given by the axial deformation of columns is overestimated. In fact, during the ground motion, the axial force of each steel brace has an upper limit equal to the slip force of the damper, which is lower than the axial force determined by the elastic analysis. Hence, axial force and axial deformation of the columns are overestimated by the elastic analysis. Further details regarding the evaluation of the contribution due to the axial deformation of columns are illustrated in a similar design procedure proposed by the authors for seismic upgrading by means of buckling restrained braces [Barbagallo *et al.,* 2017]. If the drift demand Δ is larger than the design storey drift Δ_d, the insertion of braces is needed to provide the frame with the required stiffness. Hence, the cross sectional area of the braces is determined at each storey.

In the second step, slip force and stroke of the friction dampers are evaluated at each storey. The slip load $F_{b,s}$ to be assigned to the friction dampers is determined in order to (i) provide sufficient additional dissipation capacity and (ii) limit the force transmitted by the steel braces to the existing RC frame. The value of $F_{b,s}$ is determined assuming that the damper should activate when a prefixed storey drift, named activation storey drift Δ_{act}, is attained. Since the response of the structure is elastic until damper activation, the slip force can be derived from the axial force of the brace N_b evaluated by the elastic analysis performed in Step 1

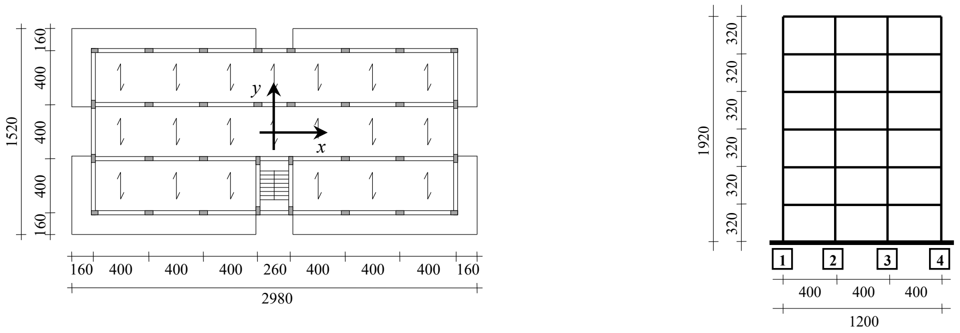
$$F_{b,s} = N_b \frac{\Delta_{act}}{\Delta_{el}} \tag{1}$$

In this paper, two criteria to assign the activation storey drift are explored. The first criterion assumes the same Δ_{act} at all storeys. This drift should be small in order to activate the dampers before the main structure exhibits damage. According to the second criterion, Δ_{act} is given as a fraction of the storey drift capacity Δ_{LS}. This fraction should be small so that the dampers start to protect the structure well before the drift capacity is attained. Furthermore, the assumed fraction is equal at all storeys, which leads to Δ_{act} variable along the height of the structure, because Δ_{LS} generally presents a variable heightwise distribution. The brace designed in Step 1 should be able to transfer the slip force. Hence, once $F_{b,s}$ is determined, it should be verified that the buckling resistance of the brace be larger than $F_{b,s}$. Finally, the stroke of the damper is calculated as the difference between the elongation of the brace δ_b demanded by the earthquake and the one corresponding to the activation of the damper $\delta_{b,s}$. The elongation demand δ_b is obtained by the results of the elastic analysis modified according to the rule by Vidic *et al.* [Vidic *et al.,* 1994], while $\delta_{b,s}$ is calculated by the following equation

$$\delta_{b,s} = \frac{F_{b,s}}{k_{b,ax}} \tag{2}$$

3 Case study

The developed design method has been applied to retrofit a RC frame designed by the old Italian codes [Italian Ministry of Public Works, 1971 and 1974] considering gravity loads only. Various combinations of the design parameters are considered.

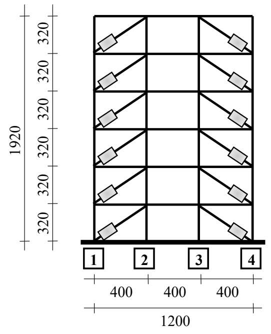


(a) (b)

Member cross-sections

Storey	Columns		Beams
	1 and 4	2 and 3	
6th	30x30	30x30	30x60
5th	30x30	30x30	30x60
4th	30x30	30x30	30x60
3rd	35x30	30x40	30x60
2nd	40x30	30x50	30x60
1st	50x30	30x60	30x60

(c)

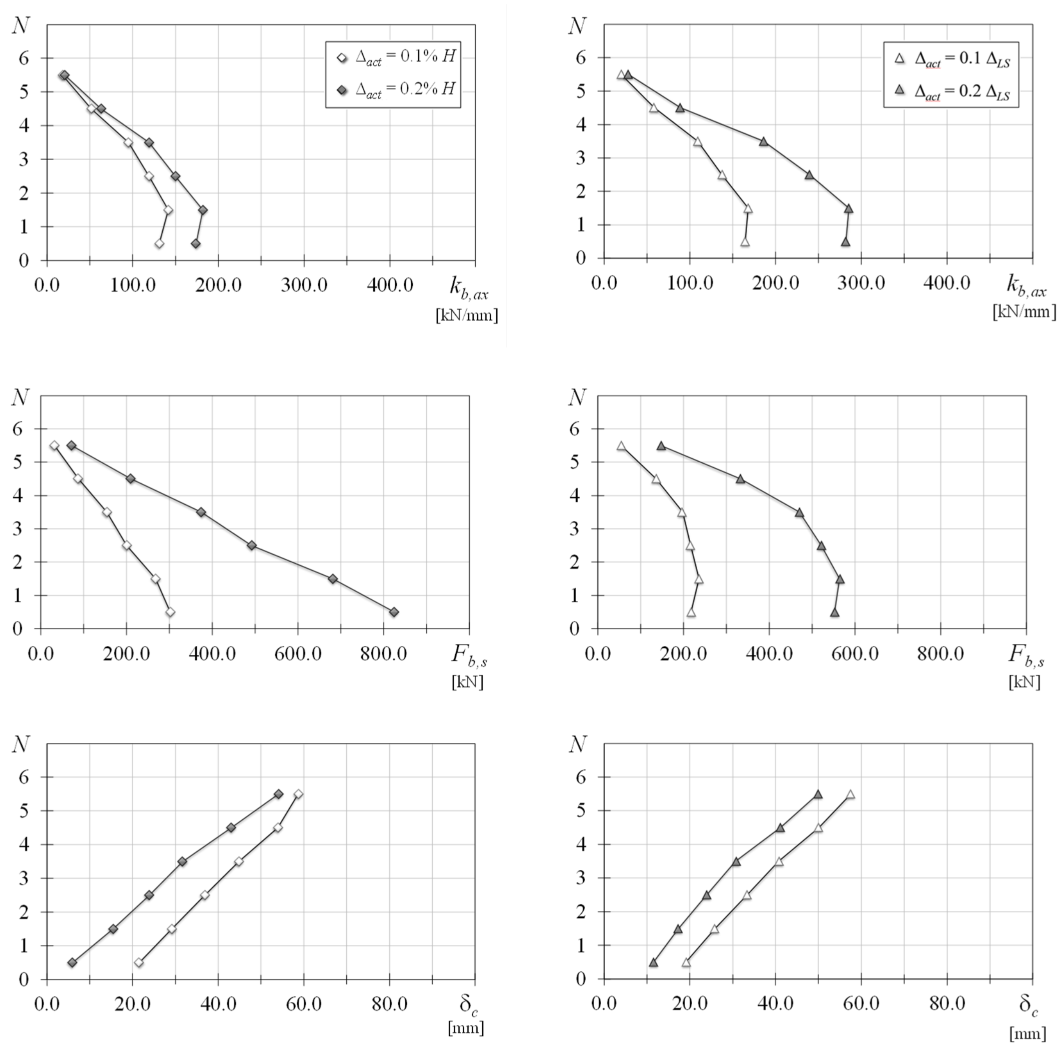


(d)

Figure 2 - *Analysed system: (a) plan layout of the building, (b) structural scheme of the bare frame, (c) RC member cross-sections and (d) structural scheme of the upgraded frame.*

3.1 Case study frame

The frame to be retrofitted is taken from a six-storey building with RC framed structure designed for gravity load according to the old Italian building code. The structure is symmetric in-plan with respect to the y-axis and presents four frames arranged along x-axis and four frames along y-axis (Figure 2(a)). Dead and live gravity loads are determined considering the nominal (characteristic) values given in [Italian Ministry of Public Works, 1996]. Size of cross-sections and area of steel reinforcements of beams and columns are determined by the allowable stress method applied according to the code in force in 1974 [Italian Ministry of Public Works, 1974]. Beams are modelled as continuous beams and are designed to sustain bending moment and shear force. Supports simulate the presence of columns, hence the numerical model is integrated considering partial rotational restrains on the supported cross-section in order to simulate the flexural stiffness of the columns. Same cross-sections and steel reinforcements of beams are adopted at all floors. Columns are designed to resist axial forces only, which are evaluated according to the tributary area concept. Since the axial force decreases along the height of the building, cross-section and steel reinforcement of the column are reduced accordingly. The minimum requirements stipulated by the aforementioned regulations for cross-section area and steel reinforcement of beams and columns are taken into account. For design, it is assumed that concrete with characteristic compressive cubic strength R_{ck} equal to 25 MPa (corresponding to characteristic cylinder strength f_{ck} equal to 20 MPa) is used, while rebars are made of steel grade Feb38K with a characteristic yield strength f_{yk} = 375 MPa.



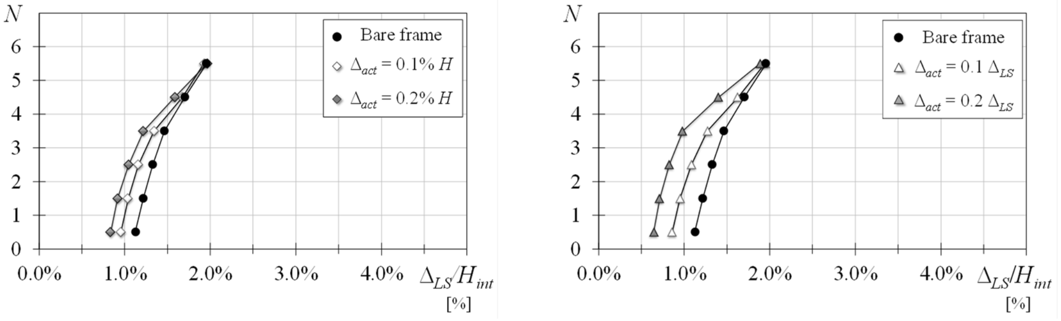
(a) (b)

Figure 3 - *Features of the bracing-friction damper system designed by: (a) constant Δ_{act}, (b) Δ_{act} equal to a fraction of Δ_{LS}.*

The building is more vulnerable to seismic force acting in y-direction because it presents low lateral stiffness and strength in this direction. Furthermore, due to the symmetry of the structure with respect to the y-direction, a two-dimensional numerical model can be used to predict the seismic response of the building in this direction. Hence, a plane frame is analysed as representative of the response of the building in y-direction. This is the outermost frame along the y-direction whose geometrical scheme is shown in Figure 2(b). Cross-section sizes of the members are reported in Figure 2(c). The location of the braces introduced by the retrofit design is shown in Figure 2(d).

3.2 Seismic upgrading of the case study frame

The proposed design method has been applied to size the bracing-friction damper system for the seismic upgrading of the case study frame.



(a) (b)

Figure 4 - *Drift capacity of columns: (a) constant Δ_{act}, (b) Δ_{act} equal to a fraction of Δ_{LS}.*

The elastic numerical model adopted for the determination of displacement and strength demands simulates beams and columns by De Saint Venant members, and braces by truss elements. Horizontal relative displacements of the nodes belonging to the same floor are restrained to simulate the rigid diaphragm effect due to the presence of the slab. The elastic response caused by earthquake excitation is determined by modal response spectrum analysis with SRSS combination method. The seismic input is given by the elastic spectrum proposed by the EC8 for soil type C and a reference peak ground acceleration set according to the Near Collapse (NC) limit state. The outermost frames are stiffer than those close to the staircase and provide a larger contribution to sustain seismic force. Hence, the floor mass assigned to these frames is equal to 30% of the total one (103 t). According to the National Annex issued in Italy [Italian Ministry of Public Works, 2012] for the application of EC8, for this limit state the minimum seismic excitation level is associated to the 5% probability of exceedance in 50 years and to a PGA equal to 0.45 g in high seismicity regions. The design storey drift is assumed equal to 0.6 Δ_{LS} [Barbagallo *et al.*, 2017]. The drift capacity Δ_{LS} of the RC frame is evaluated considering the provisions of EC8 for the evaluation of the ultimate chord rotation θ_{um}. The axial force ratio of the columns, which in turn determines the ultimate chord rotation θ_{um}, is evaluated by pushover analysis. A vertical distribution of lateral loads proportional to the first mode of vibration of the frame and a member-by-member modelling are adopted for pushover analysis. In this case, beams and columns are modelled by elastic members with rigid-plastic hinges assigned at their ends. Braces equipped with friction dampers are modelled by elastic-perfectly plastic truss elements with plastic resistance equal to the slip force. The design is performed considering the two criteria for the definition of the activation storey drift and two values of the activation storey drift. Specifically, when the same value of the activation drift is assumed at all storeys, Δ_{act} is equal to either 0.1% or 0.2% of the storey height *H*; when the activation drift is assumed as a fraction of storey drift capacity, Δ_{act} is assumed equal to either 10% or 20% of Δ_{LS}.

Figure 3 shows the axial stiffness of bracing-friction damper system, the slip force and the stroke of friction damper to be provided at each storey. The results are plotted in six figures organised in two columns; column (a) present the results obtained by the design with a constant value of Δ_{act}, while column (b) shows those obtained by assigning a value of Δ_{act} equal to a constant fraction of Δ_{LS}. In each figure, white and grey markers distinguish results obtained for small and large values of Δ_{act}. Regardless of the criterion adopted to set Δ_{act}, the required axial stiffness of the bracing-friction damper system increases with Δ_{act} and larger values are required at lower storeys. However, when Δ_{act} is assumed constant along the height of the frame, the required axial stiffness is smaller than that obtained by the other criterion (Δ_{act} equal to a fraction of Δ_{LS}). The distribution of the slip force assigned to the friction dampers depends on the criterion adopted to assign Δ_{act}: $F_{b,s}$ increases linearly from the top to the bottom of the frame when a constant Δ_{act} is adopted, while it varies more gradually along the height of the frame when Δ_{act} is assigned as a fraction of Δ_{LS}. If the slip forces obtained by the two criteria are compared ($\Delta_{act} = 0.1\%H$ vs $\Delta_{act} = 0.1\ \Delta_{LS}$ or $\Delta_{act} = 0.2\%H$ vs $\Delta_{act} = 0.2\ \Delta_{LS}$), because of the different distribution along the height of the frame, $F_{b,s}$ is larger in the upper storeys when constant Δ_{act} is adopted in design while the opposite occurs in the lower storeys. Regardless of the criterion adopted to assign Δ_{act}, the obtained slip force of friction dampers is larger for larger values of Δ_{act}. The assigned stroke δ_c increases with the storey with an almost linear distribution and tends to increase when lower values of Δ_{act} are assigned. The differences between the values of stroke assigned by the two design criteria are small at almost every storey, with the exception of the first storey of the frames designed by $\Delta_{act} = 0.2\%$ $\Delta_{act} = 0.2\ \Delta_{LS}$ where they become slightly more significant.

The introduction of bracing-friction dampers causes the reduction of the storey drift capacity of the RC columns. In fact, the reaction forces transmitted by the friction braces to the RC structure increase the axial force on columns, thus reducing their chord rotation capacity. Figure 4 shows the distribution of storey drift capacity of the RC columns along the height of the frame at the attainment of the NC limit state, for all the designs performed. Regardless of the design criterion, the drift capacity decreases compared to that of columns in the bare frame, especially at lower storeys, and larger values of Δ_{act} led to slightly lower drift capacities.

4 Effectiveness of the seismic upgrading intervention

To evaluate the performance of the rehabilitated frames, nonlinear dynamic analyses were performed and the results were compared to those of the bare frame. To this end, a numerical model of the bare frame and the upgraded frames was built using the software OpenSees [Mazzoni *et al.*, 2007].

4.1 Numerical model

A two-dimensional centreline numerical model with masses concentrated at floor levels is used to evaluate the nonlinear response of the outermost (analysed) frame. In keeping with the hypotheses assumed in Section 3.2 for the design of the seismic upgrading intervention, the floor mass assigned to the frame is equal to 30% of the total one (103 t). The nominal dead loads plus quasi-permanent live loads are assigned as initial gravity loads in the analysis. Uniformly distributed loads are applied to the beam elements and vertical forces are applied to the nodes to include the effects of the gravity loads not directly acting on the beams. A Rayleigh viscous damping is used and set at 5% for the first and the third mode of vibration. P-Δ effects are accounted for by a leaning column constituted by “elasticBeamColumn” elements pinned at their ends. The leaning column is loaded by vertical forces applied at floor levels, each one equal to the floor seismic weight (1010 kN). A “PDelta” transformation is applied to the elements of the leaning column, while a “Linear” transformation is assigned to all the other members. All the nodes of the same floor, included that of the leaning column, are constrained to have the same horizontal displacement in order to simulate the rigid diaphragm effect due to the concrete slab.

A member-by-member modelling is adopted for beams and columns. The "BeamWithHinges" element implemented in OpenSees is used. In particular, beams and columns are modelled as members constituted by an elastic element with plastic hinges of finite length at their ends. The length of the plastic hinge is equal to the depth of the cross-section. A fibre cross-section is assigned to each plastic hinge, where both concrete and steel components are considered. Concrete fibres have 5 mm depth and width equal to the width of the cross-section. Single steel fibres are used to model rebars. The Mander constitutive law (Concrete 04 uniaxial material) is assigned to concrete fibres. An elasto-plastic with strain kinematic hardening constitutive law (Steel 01 uniaxial material) is used for steel fibres. The average values derived from the characteristic values are used to quantify the strength in the constitutive law of the materials. Thus, assuming that the quality of concrete actually utilised for construction was lower than that considered in the design, the compressive strength of concrete and its Young modulus are equal to 20 MPa and 27085 MPa, respectively. The yield strength of steel and the strain hardening ratio are assumed equal to 400 MPa and 0.0066, respectively. A zero-length element is added at one end of each beam to connect the beam to the corresponding node restrained by the rigid deck. This element is characterized by a low axial stiffness and large shear and flexural stiffness. Hence, it acts as a release in the axial direction and allows the beam to deform axially. This expedient prevents the development of fictitious axial force, which occurs in RC beams modelled by fibre elements and connected to the rigid diaphragm [Barbagallo *et al.*, 2019]. The large shear and flexural stiffness transfer the shear force and bending moment. The panel zone of beam-to-column joints is not introduced in the numerical model.

In the case of the upgraded frames, the numerical model also includes the steel braces equipped with friction dampers, which are modelled as truss elements. An equivalent area is assigned to the cross-section of the truss to consider that the two components, steel brace and friction damper, have different cross-sections. An elastic-perfectly plastic uniaxial material is assigned to the truss element. The yield strength of this material is calculated as the ratio of the slip force of the damper to the equivalent area of the truss cross-section.

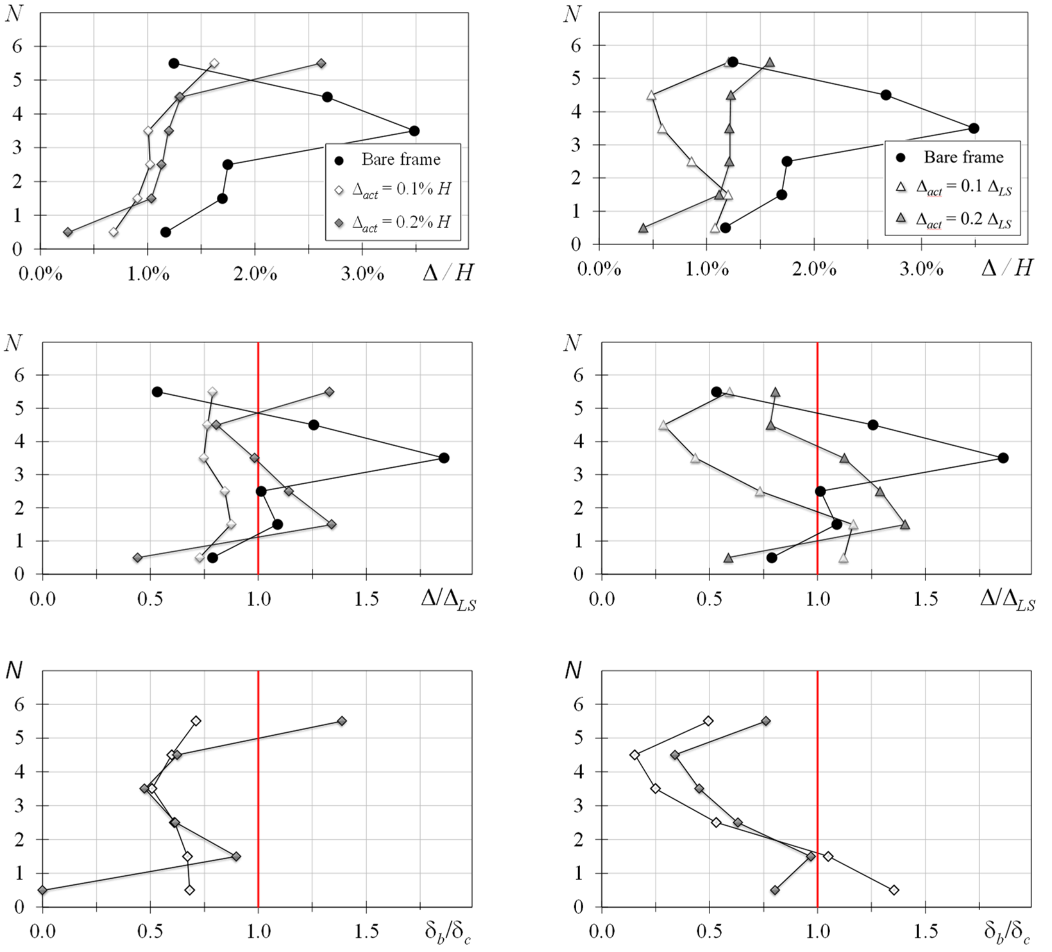
The performance of the bare RC frame and those of the upgraded frames are evaluated considering a seismic excitation level representative of earthquakes with 5% probability of exceedance in 50 years in high seismicity regions in Europe. In order to simulate this seismic excitation, a set of ten artificial ground motions, compatible with the EC8 elastic spectrum for soil type C, characterized by 5% damping ratio and PGA of 0.45 g, is adopted as seismic input. The SIMQKE computer program [SIMQKE, 1976] is used to generate the ground motions. Each ground motion is characterized by a total duration of 30.5 s and is enveloped by a three branch compound function. The earthquake rise time is 4 s; the parameter IPOW of the first branch and the parameter ALFA0 of the third one are assumed equal to 2 and 0.25, respectively To generate a set of artificial accelerograms with dynamic properties similar to those of a previously defined set of natural ground motions, different durations of the strong motion phase were investigated in a previous paper [Amara et al., 2014]. For each set of artificial accelerograms with a given duration of the strong motion phase, the Arias intensity, the significant duration, the input energy and the number of equivalent cycles have been calculated and compared with those relative to the target natural ground motions. Based on these comparisons, the duration of the strong motion phase of the accelerograms was assumed equal to 7.0 s

4.2 Results

The seismic performance of the analysed frames is expressed in terms of distribution along the height of drift demand Δ, drift demand-to-capacity ratio Δ/Δ_{LS} and stroke demand-to-capacity ratio δ_b/δ_c. The maximum drift angle and the maximum stroke demand of each storey are determined for the 10 ground motions. Afterwards, the median value of drift and stroke demands over the values of the 10 ground motions are calculated assuming a lognormal distribution of the data. Figure 5 shows the heightwise distribution of the results of the nonlinear dynamic analyses. The format of the figures is the same adopted in Figure 4. The response of the bare frame (black circular markers) is compared to that of the rehabilitated frames (white and grey markers). Focusing on the bare frame, it is evident that it suffers from a severe damage concentration at the fourth storey, and the drift demand strongly overcomes the capacity (the maximum ratio Δ/Δ_{LS} is significantly larger than 1). In all the examined cases, the insertion of the bracing-friction damper system mitigates the damage concentration, reduces the storey drift and in turn the drift demand-to-capacity ratio. However, the beneficial effect is influenced by the criterion adopted to assign Δ_{act} and by the value assigned to Δ_{act}. This is more evident when the comparison is carried out in terms of drift demand-to-capacity ratio. The comparison of the curves plotted in Figure 5(a) to those of Figure 5(b) shows that Δ/Δ_{LS} is smaller when Δ_{act} is assumed constant along the height of the frame. Furthermore, the comparison of the curves related to the two rehabilitated frames plotted in each figure shows that the ratio Δ/Δ_{LS} of the system designed by Δ_{act} =0.10%H is smaller than that of the system designed by Δ_{act}=0.20%H. Regardless of the design criterion, it is noteworthy that on one side, the insertion of the friction bracings decreased the drift capacity of columns (Figure 4). However, on the other side, friction dampers reduced even more the drift demand and this allowed the attainment of a drift demand-to-capacity ratio more uniform along the height and smaller than that of the bare frame. Out of the analysed systems, the best improvement of seismic performance is achieved when the design is performed by a constant Δ_{act} equal to 0.1%H. This frame exhibits drift demand-to-capacity ratio almost constant along the height and always smaller than 1. Furthermore, this design led to a stroke demand lower than the designed stroke capacity at each storey, and a distribution of the stroke demand-to-capacity ratio uniform along the height of the frame. On the contrary, the ratio Δ/Δ_{LS} of the three frames designed with the other design settings, even if smaller than that of the bare frame, is still larger than 1. Moreover, the ratio δ_b/δ_c is larger than one at some storeys and its distribution at all storeys is less regular.

5 Conclusions

This paper investigates the effectiveness of the seismic upgrading of RC frames by steel braces equipped with friction dampers and proposes a design procedure to size braces and dampers. The design procedure is controlled by two parameters: Δ_d and Δ_{act}. The first parameter is the storey drift that should not be exceeded for the design seismic excitation level and is defined as a fraction of the storey drift capacity Δ_{LS}. The second parameter is the storey drift corresponding to the activation of the friction dampers. The parameter Δ_d is assumed equal to 0.6 Δ_{LS} in all the analysed cases. The value of Δ_{act} is assigned according to two different criteria:



(a) (b)

Figure 5 - *Drift demand, drift demand-to-capacity ratio, stroke demand-to-capacity ratio of the analysed frames: (a) constant Δ_{act}, (b) Δ_{act} equal to a fraction of Δ_{LS}.*

(i) Δ_{act} equal to a constant fraction of the storey height *H*, (ii) Δ_{act} equal to a fraction of the storey drift capacity. For each criterion, two values of Δ_{act} are considered: 0.1% and 0.2% of *H* or 10% and 20% of Δ_{LS}. Hence, the case study frame has been retrofitted with four designs, which differed for the values assigned to the design parameters. The seismic performance of the four rehabilitated frames, determined by nonlinear dynamic analysis, showed that the insertion of steel braces equipped with friction dampers reduced storey drift and storey drift demand-to-capacity ratio with respect to those observed for the bare frame. The best seismic performance is obtained when a constant Δ_{act} equal to 0.1% of the storey height *H* is adopted in design. Indeed, this allowed an almost uniform distribution of the storey drift demand along the height of the building. Furthermore, at each storey of the frame the drift demand was lower than the drift capacity and the stroke demand lower than the stroke capacity.

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PROGETTO DI INTERVENTI DI ADEGUAMENTO SISMICO DI TELAI IN C.A. MEDIANTE DISSIPATORI AD ATTRITO

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SOMMARIO: *Oggi molti edifici con struttura intelaiata in c.a. necessitano di interventi di adeguamento sismico. L'inserimento nel telaio di controventi in acciaio dotati dissipatori ad attrito è una tecnica di adeguamento sismico promettente per questi edifici. Infatti, controventi e dissipatori ad attrito riducono la domanda di spostamento, fornendo rigidezza e dissipazione di energia. Inoltre, i dissipatori controllano le forze trasmesse dai controventi evitando che il sistema di adeguamento sovraccarichi la struttura esistente. In questo articolo si formula una procedura di progetto del sistema di adeguamento. La procedura viene applicata a un caso di studio prendendo in considerazione varie combinazioni dei parametri che controllano il progetto. Quindi, l'analisi della risposta sismica di telaio nudo e telai adeguati fornisce informazioni sull'efficacia della tecnica di adeguamento e sulla corretta impostazione dei parametri di progetto.*

PAROLE CHIAVE: *controventi, acciaio, miglioramento sismico, edifici esistenti, procedura di progetto*

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