Seismic Retrofit of Steel Frames Based on Friction-Damped Linked Columns with Pier Bases Allowing Uplift

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Abstract: Seismic retrofit of existing steel structures can employ modern systems for passive energy dissipation. Previous research of the authors confirmed the efficiency of a new retrofit technique based on adding friction-damped linked columns to low-ductile moment resisting frames designed to older generation seismic codes. This approach can be classified as a "soft intervention" because it avoids dramatic change in overall stiffness and massive strengthening of existing frame members and joints. To further enhance the practical applicability of the proposed retrofitting technique, an improved detailing is proposed for the link column bases in order to limit the build-up of large tension forces during strong earthquakes. This solution allows base uplift when the slip capacity of the added rotational friction dampers is reached. The numerical study on a representative three-storey frame retrofitted with friction-damped linked columns with pier bases allowing uplift confirmed the efficiency of the concept. The results obtained by nonlinear time history analysis indicate that the seismic demand on the existing moment resisting frame can be significantly reduced through well-selected combination of added stiffness and supplemental damping.

Keywords: seismic upgrade, friction damper, linked column, nonlinear analysis

1. Introduction

The Linked Column Frame is a seismic force resisting system initially developed for applications in newly constructed bridge piers and steel frame buildings. Its detailed description and features are given in Dusicka et al. (2009) and Malakoutian et al. (2012).

For improving the seismic performance of existing seismic-deficient steel moment resisting frames (MRFs) designed to older generation seismic codes a new retrofitting technique was developed by Bonchev et al. (2018). By adding Linked Columns (LCs) to the original steel MRFs a dual lateral force resisting system is created with a complicated interaction between the two sub-systems. A typical LC consists of two closely spaced vertical piers connected throughout their height by short shear links (seismic fuses) typically arranged at each floor level. To avoid the limitations of conventional shear links which are expected to develop large plastic deformations and eventually may fail due to low-cycle fatigue, in the concept proposed by Bonchev et al. (2018) the ductile shear links of the LCs are replaced by with rotational friction dampers (RFD). Friction dampers of this type have previously been used in many projects over the last two decades as described in Mualla and Belev (2015). They can be classified as displacement-dependent anti-seismic devices to CEN EN 15129 (2018) and have stable energy-dissipation capacity. RFDs of various designs and slip capacities are commercially supplied by the Danish company Damptech A/S.

The suggested retrofitting technique based on LCs with RFDs makes use of LCs arranged symmetrically on both sides of the interior columns of the existing steel MRFs. The set of all LCs is designed to perform as a primary lateral force resisting sub-system in the respective direction while the existing MRF-sub-system essentially supports the gravity loads and should supply the lateral stiffness required after the onset of RFDs slipping and for self-centering of the building structure following a strong earthquake.

Our previous numerical studies on this retrofitting technique performed on low-ductile steel MRFs up to nine-storey high proved the efficiency of the embedded concept but also showed that large tensile forces could arise in the LC piers with increasing the building height. This phenomenon may complicate their anchorage to the foundations and render the practical implementation of the concept difficult. The present paper describes a further improvement of the retrofitting technique based on LCs with RFDs which addresses the issue of high-tension forces build-up in the LC piers. To put an upper limit to the tension forces at the pier bases induced by strong seismic ground shaking, improved detailing of the LC-pier base allowing uplift was developed. It also provides supplemental damping via additional RFDs connecting the pier bottom ends to the foundation.

The seismic performance of the original and retrofitted frames was assessed using dynamic nonlinear time history analysis (NLTHA) carried out by SAP2000 software (C&S, Inc. 2019). The results obtained from the numerical analyses are presented and summarized.

2. Overview of the proposed retrofitting technique

The original retrofitting technique proposed in Bonchev et al. (2018) is based on adding a sub-system of LCs with RFDs to the existing steel MRFs. The Linked Columns are arranged symmetrically on both sides of the first interior columns of the existing frame in order to avoid possible conflicts with the façade envelope of the building. The lateral force transfer from the existing frame to the LCs is achieved by short tubular brackets joining the MRF beams and LC piers.

The friction-damped LCs incorporate RFDs at each floor level as shown in Fig. 1. A similar approach for RFD installation within the coupling beams connecting reinforced concrete shear walls was previously proposed by Damptech A/S (Fig. 2).

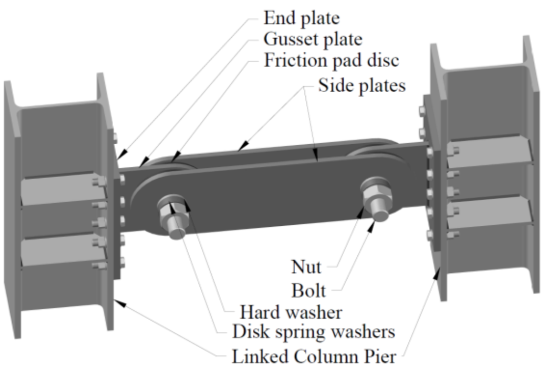

Fig. 1 – Layout of seismic link with RFD including two friction joints

Fig. 2 – Layout of coupling beam with RFD for RC shear walls (courtesy of Damptech A/S)

To limit the tension forces transferred to the foundation, a couple of pin-connected RFDs were added at the bottom end of each pier as shown on Fig. 3. In order to accommodate the uplift of a LC pier when its tensile force reaches the frictional resistance of the inclined dampers, the pier webs are bolted to vertical fin plates with vertically slotted (oval) holes. The compressive forces in the pier bottoms are transferred to the base plate in direct bearing via contact plates welded to the flanges of the cross-section.

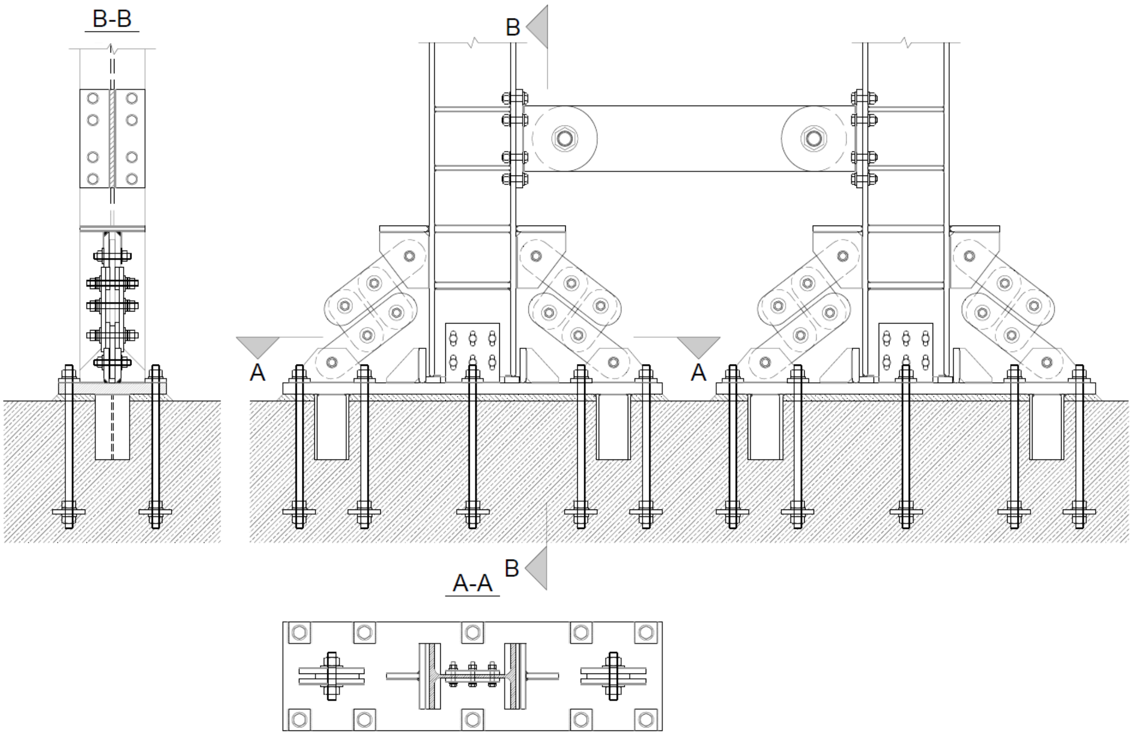

Fig. 3 - Layout of Linked Column base with added friction dampers

Design provisions for structures with friction dampers are not available in CEN EN 1998-1 (2004), but the European standard CEN EN 15129 (2018) for anti-seismic devices or alternatively ASCE 41-17 (2017) can be used as a basis for design of the proposed retrofitting system.

The RFDs of the type shown on Fig.1 are attached to the vertical piers through bolted end-plate connections for easier on-site installation. These connections typically employ high strength bolts working in tension and shear and can be designed in compliance with CEN EN 1993-1-8 (2005) provisions. According to CEN EN 15129 (2018) an over-strength factor γ_{Rd}=1,1 and a reliability factor γ_x=1,2 shall be applied to the actions transmitted by the damper connections to the piers.

3. Description of the investigated structure and response analysis method used

The three-storey four-bay steel MRF shown in Fig. 4 was designed according to Bulgarian code for design of steel structures KTSU (1987a) applicable since 1987. The beam-to-column joints are assumed rigid and the column bases are fully restrained. The member cross-sections are built-up welded I-sections in compliance with the local designers' practice prevailing during the 1970-1990 period.

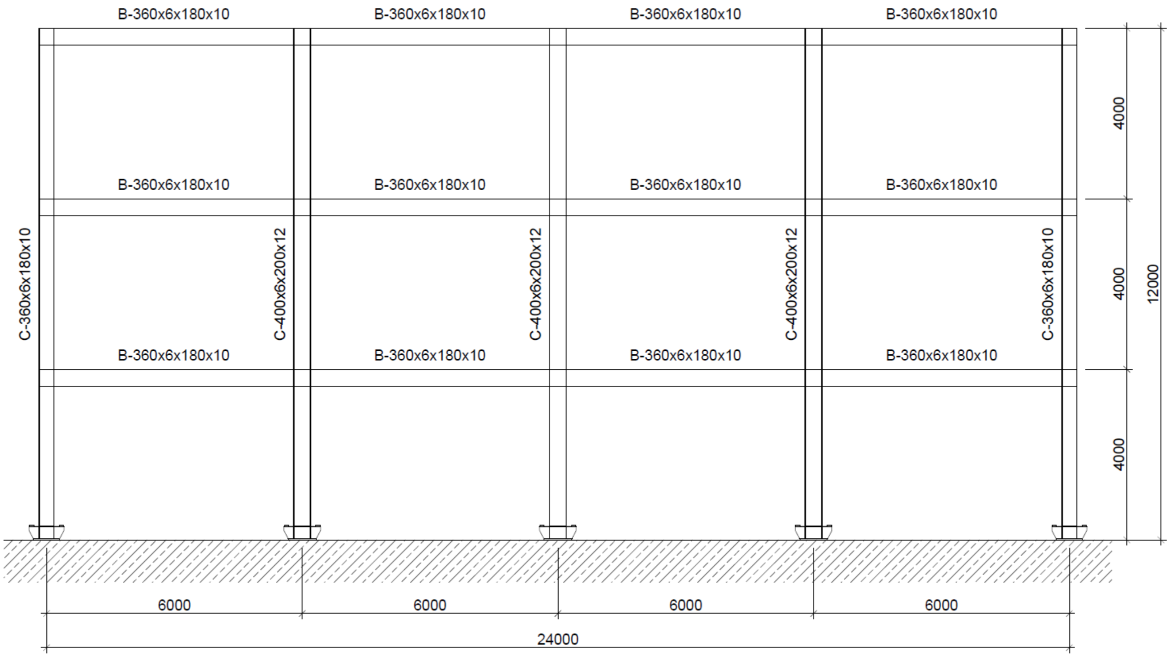

Fig. 4 - Geometry and cross-sections of three-storey representative steel MRF

The design seismic forces are obtained from the elastic response spectrum for Sofia region modified by a response factor *R*=0,20 as prescribed by the Bulgarian code for earthquake-resistant design KTSU (1987b), which corresponds to assuming behaviour factor $q = 5$ to CEN EN 1998-1 (2004). For the sake of material savings slender cross-sections were typically used in the design of steel structures during the 1970-1990 period and such cross-sections are usually classified as Class 3 sections to CEN EN 1993-1-1 (2005a). Class 3 cross-sections are not allowed by CEN EN 1998-1 (2004) for the dissipative zones of steel structures of moment-resisting frames if $q > 2$. In addition, KTSU (1987b) did not impose capacity design requirements ("strong-column-weak-beam" rule for MRFs), which possibly can lead to formation of plastic hinges in the columns and consequently to weak stories. Furthermore, the seismic code of that time had simplified rules for estimating inter-storey drifts and rather tolerant drift limits which could result in excessive structural and non-structural damage.

The seismic upgrade solution includes one horizontal link per storey placed at each floor level with moment slip capacity of 20 kNm per friction joint. Wide-flange hot-rolled I-section HEB400 is chosen for the linked column piers. The inclined pin-connected RFDs at the pier bases have assumed slip capacity of 150 kN.

The seismic performance of the representative steel frame and its retrofitted counterpart was evaluated using NLTHA based on the Direct Time Integration method. A set of seven accelerograms compliant with the elastic response spectrum for ground type C of CEN EN 1998-1 (2004), design ground acceleration a_g = 0,23g and constant damping ratio of 5% were used. Material nonlinearity and P-delta effects were taken into account. Plastic hinges in the beams and columns of the MRF were defined by link elements with bilinear moment-rotation relationships based on ASCE 41-17 (2017). Beam-to-column joint panel zone distortion was accounted for using the popular Krawinkler model and the moment-rotation behaviour of the friction hinges was defined by the Bouc-Wen model available in SAP2000 software. Gravity loads including 100% of the permanent loads and 24% of the characteristic imposed loads were applied prior to the seismic ground motion. The contact of LC piers with the base plate was modelled with a gap element, while the action of the inclined RFDs was represented by link elements with bilinear axial force-displacement relationships.

4. Results obtained from the numerical study

The basic structural and seismic response parameters of the original MRF and its counterpart retrofitted by friction-damped LCs (MRF+LC) are summarized in Table 1.

The retrofitted system has higher initial lateral stiffness than the original one as indicated by their fundamental periods of vibration. This is considered an advantage taking into account that the seismic design of conventional MRFs is commonly governed by inter-storey drift limitations. Fig. 5 displays comparison of peak lateral displacements of the original and retrofitted frames.

Table 1. Structural and response parameters of the investigated original and retrofitted frames

			MRF	*MRF+LC*
Fundamental period of vibration, T		[s]	1,29	0,66
Roof displacement and average drift		[cm]	11,6	5,5
		[%]	0,97	0,42
Base shear	***Total***	[kN]	398	504
	MRF sub-system		398	122
	LCs sub-system		-	382
Input seismic energy		[kJ]	89,3	92,7
		[%]	100	103,9
Hysteretic energy		[kJ]	22,9	33,9
		[%]	25,7	38,0

Due to the increased lateral stiffness the overall base shear in the retrofitted structure increases, but a major part of it is resisted by the Linked Columns. The base shear in the existing frame drops down by nearly 70 % which proves the role of LCs as a primary lateral force resisting sub-system and the benefits of it. The less loaded MRF develops smaller chord rotation of beams and panel zone distortions which respectively reduces the damage potential.

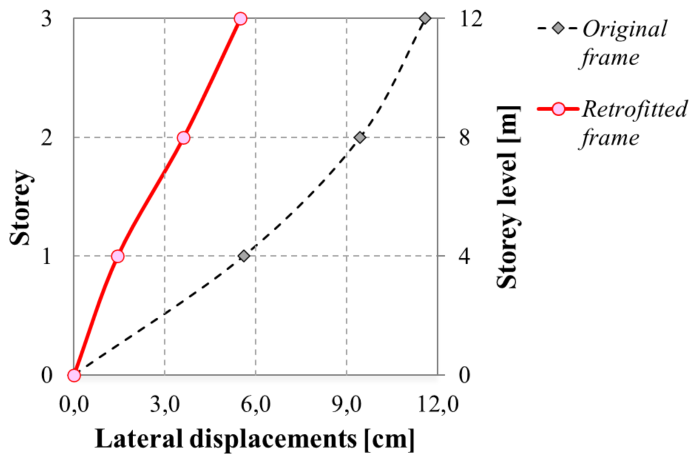

Fig. 5 - Maximum lateral displacement of the original and retrofitted frames

The inter-storey drifts of both examined structures corresponding to the design quake intensity are shown in Fig. 6. The inter-storey drift limits according to CEN EN 1998-1 (2004): 0,005*h* = 2,0 cm; 0,0075*h* = 3,0 cm and 0,010*h* = 4,0 cm where *h* is the storey height are indicated by dashed lines. These limits are applicable to inter-storey drifts induced by a "serviceability" quake with intensity equal to 40% to 50% of the design quake intensity used in the reported numerical study. Obviously the reduction of the inter-story drifts is significant and the proposed retrofit approach easily overcomes the stiffness deficiency of the original (existing) steel MRF.

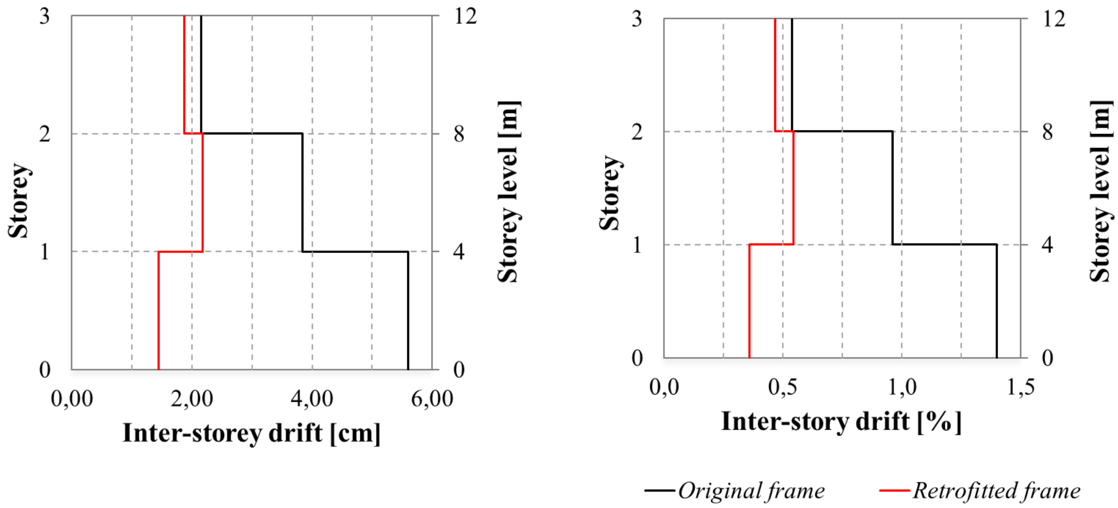

Fig. 6 – Inter-storey drifts of the original and retrofitted frames

Fig. 7 displays and compares the histories of the input seismic and hysteretic energies calculated for both examined structures. These energy histories were obtained by averaging the individual energy histories corresponding the set of seven ground acceleration time histories used as seismic input. The final values of the input seismic energy in the retrofitted and original structures differ only by 4%, while the hysteretic energy dissipated by the retrofitted frame is increased by 12%, as a result of adding dampers along the height of each LC and the supplementary RFDs at the pier bases.

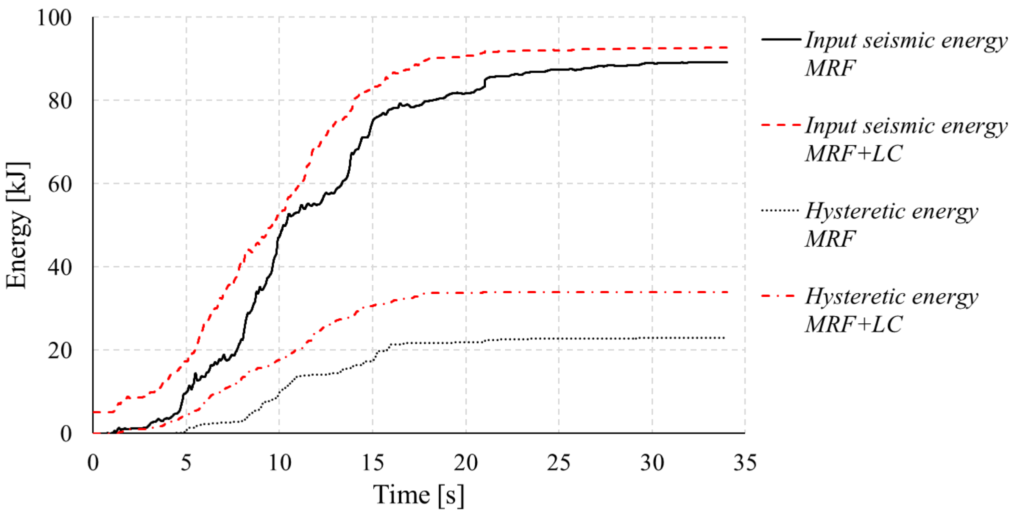

Fig. 7 – Averaged energy histories of the original and retrofitted frames

The maximum tension force in pier bottoms reached 130 kN. Forces of this magnitude can be easily transferred to the foundation by anchor bolts without any special detailing. The slip capacity of the inclined RFDs could be further optimized to reach a cost-efficient foundation design.

4. Conclusions

The paper presents an improved version of a seismic retrofitting technique based on added friction-damped LCs which was proposed by the authors in their previous research. Supplementary friction dampers are placed at the bases of the LC piers with the purpose to accommodate uplift and limit the magnitude of the tension forces which develop in the piers during strong ground shaking. The reported numerical study of a representative seismic-deficient steel MRF via NLTHA confirmed the efficiency of the proposed retrofit technique. The ductile response of the resulting dual structural system can be achieved without damaging the original (existing) steel MRF. The LCs equipped with RFDs can supply appropriate additional stiffness and supplemental damping to the existing frame structure, thus controlling the inter-storey drifts, reducing the base shear demand and protecting it from damage.

The reported study also demonstrated that existing steel MRFs designed to older generation seismic codes can be successfully upgraded by the proposed approach of “soft intervention” without dramatically changing the lateral stiffness and massive local strengthening of existing frame members and joints. The seismic demands on the existing low-ductile steel frames can be efficiently reduced through a well-selected combination of added stiffness and supplemental damping.

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