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OPTIMIZATION OF DISSIPATIVE REPLACEABLE LINK FRAMES BY ELASTIC HIGH STRENGTH STEEL COUPLING BEAMS

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Abstract. *The Dissipative Replaceable Link Frame (DRLF) is an innovative lateral load resisting system, recently developed, investigated, and further optimized in the European research projects FUSEIS, MATCH, INNOSEIS and DISSIPABLE. The system consists of two strong columns, which are rigidly interconnected by several horizontal beams, forming a vertical VIERENDEEL beam. The interconnecting beams are attached by a bolted head plate connection, making them easily detachable. As they are separated from the slab, they do not take part in the gravity load bearing function. The dissipative beam links being the only intended spots of inelastic action and therefore energy dissipation, plus being easily exchangeable because of their bolted connection and missing participation in the gravity load bearing paths, act as exchangeable seismic fuses during a strong earthquake. After such an event, exchanging the beam links is foreseen to bring the structure back to its undamaged pre-earthquake stage quickly. However, Eurocode 8 demands on damage limitation affecting non-structural components – like e.g. partition walls – might represent a tough requirement on global lateral stiffness of the Link Frame systems. For high buildings, displacement verifications often govern the design, similar as known for conventional Moment Resisting Frames (MRF). In order to overcome this restriction, in this paper, a solution to increase lateral stiffness of Dissipative Replaceable Link Frames is investigated: Keeping the connections of the regular gravity frame hinged, strong elastic steel coupling beams are introduced, connecting two adjacent Link Frames. These coupling beams are not intended to dissipate energy, but instead should remain elastic, in order to keep the whole building easily repairable, by restricting seismic damage to the Dissipative Replaceable Links. Moreover, such additional source of lateral stiffness is able to improve re-centering capability to diminish accumulation of residual displacements during an earthquake and further to re-center the building when the seismic fuses are replaced. In this paper, preliminary conclusions on the concept and performance of this innovative system under development in the ongoing project DISSIPABLE are presented.*

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1 INTRODUCTION

The Dissipative Replaceable Link Frame (DRLF) is an innovative lateral load resisting system, which was initially developed and investigated in the European research project "FUSEIS - dissipative devices for seismic-resistant steel frames" [1]. The objective of the FUSEIS project was to overcome the drawback of non-repairability of traditional systems such as Moment Resisting Frames (MRF), Concentrically Braced Frames (CBF) or Eccentrically Braced Frames (EBF). While such conventional systems perform well during strong seismic events due to dissipation of a large amount of energy, however, they are not economically repairable after the design basis earthquake has struck the structure. Thus, usually the only available option is the demolishing of the damaged building and replacing it by a new one. To overcome this issue, within the FUSEIS project several innovative systems have been developed, where the dissipative elements called fuses are the only source of energy dissipation and above all are easily replaceable after an earthquake.

One of the investigated systems was the formerly called FUSEIS-1 system (see [1], [2], [3]), which in this paper is called Dissipative Replaceable Link Frame (DRLF). It was further developed and investigated within the MATCH [4] and INNOSEIS [5] projects, and is further enhanced within the ongoing DISSIPABLE [6] project.

The DRLF system consists of two strong columns, which are rigidly interconnected by several beams, forming a vertical VIERENDEEL beam. In order to be easily detachable, these interconnecting beams – the Dissipative Replaceable Links (DRL) – are attached by a bolted head plate connection. As they are not coupled with the slab, consequently, they are not a part of the gravity load bearing frame. Optimally, inelastic action is foreseen to take place solely in the Dissipative Replaceable Links (DRL). When achieving this, they can act as exchangeable seismic fuses during a strong earthquake. Being the only source of energy dissipation by design, along with the missing participation in the gravity load bearing paths and moreover their detachable connection design, exchanging the Dissipative Replaceable Links is foreseen to bring the structure back to its undamaged pre-earthquake stage quickly, without the need of rebuilding the whole structure.

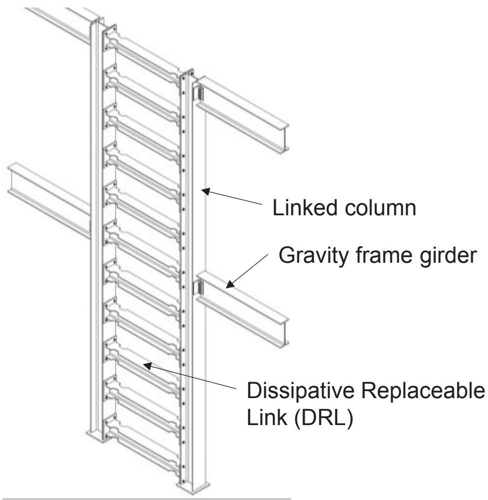
Eurocode 8 [7] demands on damage limitation affecting non-structural components – like e.g. partition walls – represent a tough requirement on global lateral stiffness of such Dissipative Replaceable Link Frame systems. For high buildings, displacement verifications often govern the design. In this case, usually partial-strength moment connections may be introduced to the gravity load bearing frames, in order to increase global stiffness. However, this may result in damage of the gravity frame connections, which are usually not feasibly repairable, finally pre-venting full exploitation of the potentially excellent energy dissipation capacity of such a system.

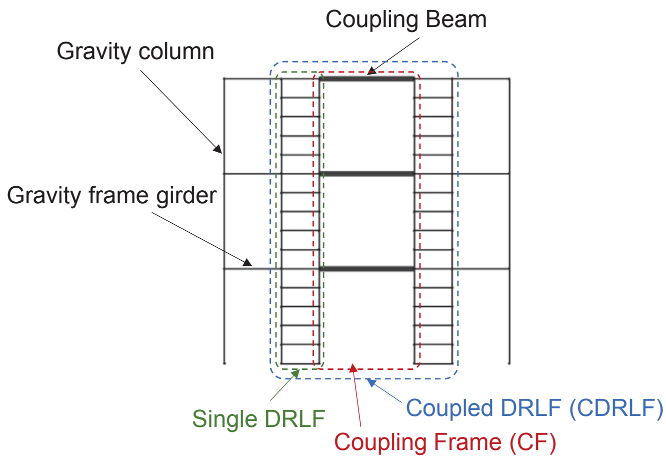
In this paper, an alternative solution of increasing lateral stiffness of Dissipative Replaceable Link Frame systems is investigated: Keeping the connections of the gravity frame hinged, strong elastic steel coupling beams are introduced, connecting two Dissipative Replaceable Link Frames. These coupling beams are not intended to dissipate energy, but instead should remain elastic, in order to keep the whole building easily repairable, by restricting seismic damage to the Dissipative Replaceable Link fuses only.

In the following, after a general introduction to the innovative concept of the Coupled Dissipative Replaceable Link Frame (CDRLF) system, two case studies of an 8-storey office building are presented. The first one, acting as 'reference system', is the already known Dissipative Replaceable Link Frame. Because the building is quite high, in total three bays with such frames were required, to comply with seismic requirements on stiffness. The second system, labelled as 'optimized system', introduced the concept of coupling beams. Consequently, only two bays of DRLFs were needed. In a first step, the performance of both systems was assessed and compared via pushover analyses. Target displacements were derived based on the N2 method. Further on, for verification of the conclusions derived by the non-linear static analyses, dynamic time history analyses with seven recorded accelerograms have been conducted. The gain in performance achieved by the introduction of the coupling beams is discussed in terms of pushover curves, plastic mechanisms, inter-story drifts, residual displacements and repairability. It is demonstrated, how the optimized system is well able to comply with the foreseen performance targets.

2 DISSIPATIVE REPLACABLE LINK FRAMES

A schematic sketch of the Dissipative Replaceable Link Frame (DRLF) is shown in Figure 1, left panel. For high buildings, this type of system may have the issue of inherent flexibility, which is also a well-known issue for classical Moment Resisting Frames (MRF). Moreover, residual displacements may hinder the repairability performance target. In order to increase stiffness and re-centering capability, two of such frames might be connected by Coupling Beams, as shown in Figure 1, right panel. This combination results in the Coupled Dissipative Replaceable Link Frame (CDRLF). The lateral strength and stiffness of such a coupled frame is basically due to the frame action of the two single DRLFs as well as the Coupling Frame, which consists of the coupling beams and the strong columns, where the coupling beams are attached to. Thus, the internal strong columns contribute to both frames. The in total three frame actions act in series resulting in the global frame behavior (see Figure 2).





Dissipative Replaceable Link Frame (DRLF) Coupled Dissipative Replaceable Link Frame (CDRLF)

Figure 1: Single Dissipative Replaceable Link Frame (DRLF) (left) and two DRLFs coupled by elastic HSS Coupling Beams (CB) forming the Coupled Dissipative Link Frame (CDRLF) (right).

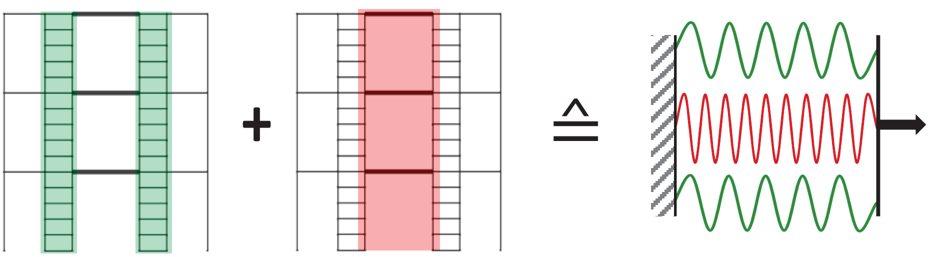


Figure 2: Two single Dissipative Replaceable Link Frames (DRLFs) and the Coupling Frame (CF) acting in series as resulting total lateral load resisting system.

The concept of coupling beams is a well-known one for linking two reinforced concrete shear walls. However, in this context, usually it are the coupling beams, which are foreseen to dissipate energy. See e.g. [8] for an innovative system of concrete shear walls connected via replaceable steel coupling beams. Contrary, in the scope of the research presented herein, the coupling beams need to remain elastic for the design basis earthquake (usually 10% in 50 years / 475 years return period). Therefore, the investigated innovative system could be best compared to dual frame systems, e.g. a combination of an EBF with a MRF, where the MRF is deemed to add stiffness and residual strength to the system, see e.g. [9].

Design equations for the DRLF have been developed in the FUSEIS [1] project and disseminated throughout the INNOSEIS [5] project. However, the coupling frame connecting two DRLF is a recently developed enhancement during the ongoing DISSIPABLE [6] project. Therefore, design equations and guidelines for the coupling frame are missing yet. For the time being, design verification by non-linear static (pushover) and/or response history analysis (RHA) is needed. Development of design guidelines is foreseen during the DISSIPABLE [6] project.

The coupling beams are suspected to high seismic strength demands. Moreover, once the beam links are plastified, the contribution of the Coupling Frame on the overall stiffness increases, further enlarging demands on coupling beams. Consequently, usually relatively high strength steels will be needed to be applied for these elements.

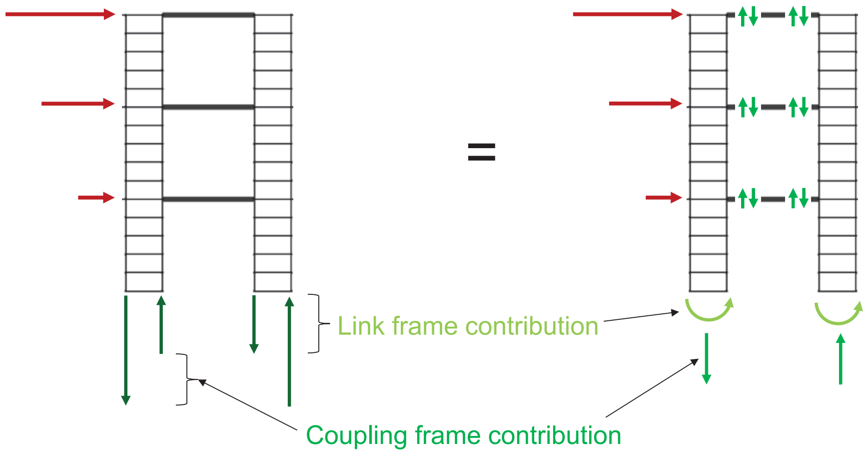


Figure 3: Load transfer and resistance contribution of the single DRLF and the CF.

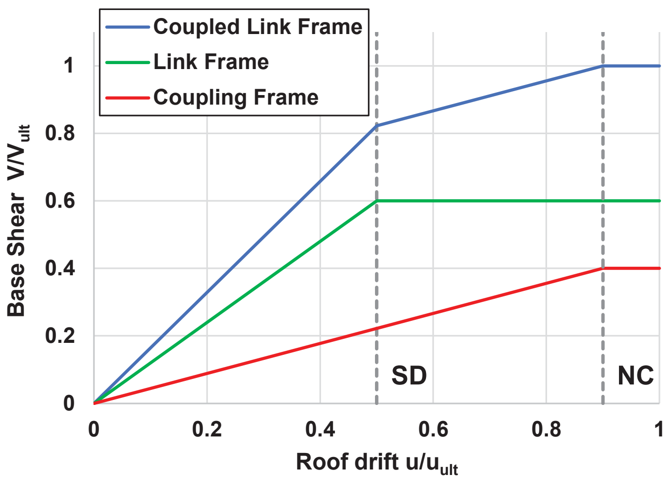


Figure 4: General behavior of the CDRLF and contribution of its single components.

The general design philosophy behind the CDRLF is summarized in Table 1. For frequent earthquakes, no damage shall occur to the overall lateral load resisting system. For the design basis earthquake, only the Dissipative Replaceable Links shall be damaged, while coupling beams are foreseen to remain elastic. Thus, the whole building results to be repairable, preventing its demolition. Only for very rare earthquakes, also the coupling beams are allowed to suffer damage, in order to prevent collapse of the building while sacrificing its reparability.

Possible design scenarios	**10% in 10 yrs 95 yrs**	**10% in 50 yrs 475 yrs**	**2% in 50 yrs 2475 yrs**
Performance target	No damage	Reparability	No collapse
Damage to dissipative links	No	Yes	Yes
Damage to coupling beams	No	No	Allowed
Reparability	---	Yes	No, if coupling beams damaged

Table 1: Design philosophy for the Coupled Dissipative Replaceable Link Frame (CDRLF).

3 CASE STUDY

To illustrate the concept of the CDRLF, in the following a typical 8-story office frame building has been designed in two different ways. In the first one, no coupling beams have been introduced, which rendered the system to be a DRLF. In the latter approach, the coupling frame effect was introduced, resulting in a CDRLF system.

The main dimensions of the initial situation used as starting point are shown in Figure 5. Firstly, the two designs have been conducted to satisfy seismic design requirements according to Eurocode 8 [7] regarding strength as well as stiffness. Table 2 shows an overview of the loads considered in design. Furthermore, the characteristics of the considered seismic action are summarized in Table 3.

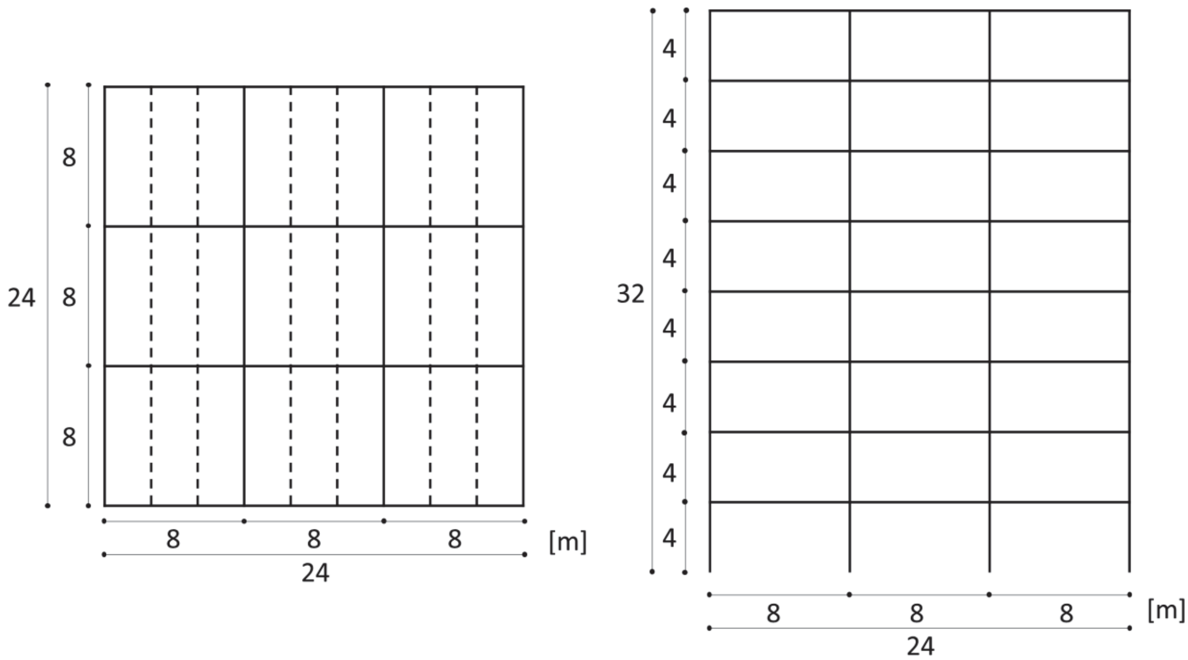


Figure 5: General floor plan and elevation of 8-story case study.

Design loads	Description	Load value
Dead loads	Composite slab and steel sheeting	2.75 kN/m²
Superimposed loads	Services, ceiling and raised floor	
	- intermediate floors	0.70 kN/ m²
	- top floor	1.00 kN/m²
	Perimeter walls	4.00 kN/m²
Live loads	Offices (Class B)	3.00 kN/m²
	Movable partitions	0.80 kN/m²

Table 2: Loading conditions used for design.

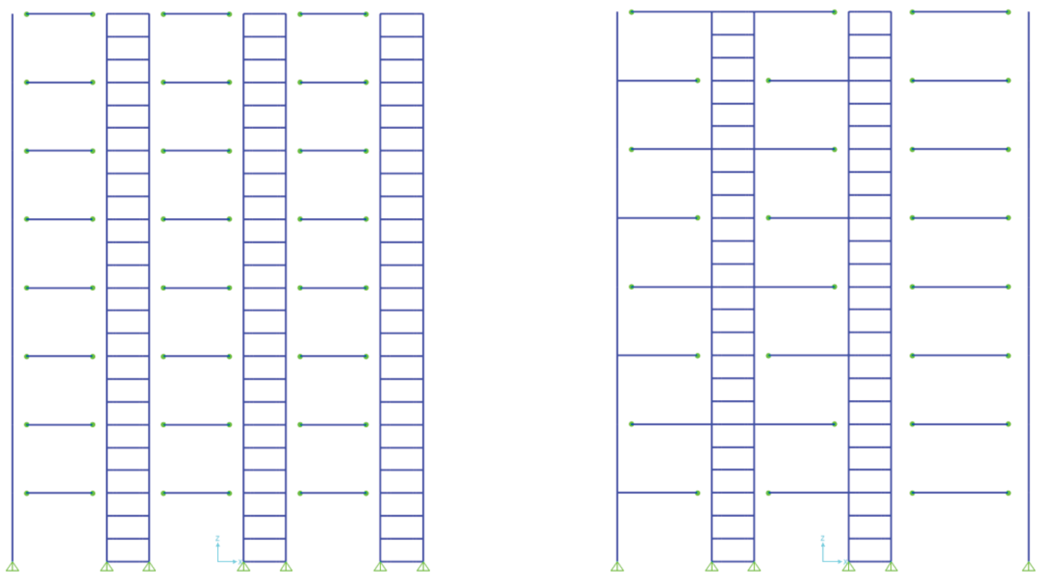
Seismic design action	Description
Peak ground acceleration	$\alpha_{gR} = 0.3g$
Importance factor (Class II)	$\gamma_1 = 1.0$
Ground type	B ($S = 1.2$, $T_B = 0.15$ s, $T_C = 0.5$ s, $T_D = 2.0$ s)
Type of response spectrum	Type 1
Lower bound factor β	0.2
Behavior factor q	2.0 for the reference structural system 2.8 for the optimized structural system

Table 3: Seismic design action according to Eurocode 8.

Finally, the designs resulted in the following case study frames shown in Figure 6:

- Reference structural system: 2D frame with three Dissipative Replaceable Link Frames and pinned coupling beams (see Figure 6a), thus no coupling frame action;
- Optimized structural system: 2D frame with two Dissipative Replaceable Link Frames and two bays with coupling beams, for which alternating pinned/fixed connections were used (see Figure 6b); thus, the whole system is a Coupled Dissipative Replaceable Link Frame.

For a better understanding of the structural response, especially the influence and contribution of dissipative links and coupling beams, these two structural configurations were studied in detail by means of pushover and response history analyses.



(a) Reference system – 3 × DRLFs
(all coupling beam ends are pinned)

(b) Optimized system – 2 × DRLFs + 2 × CFs
(coupling beam ends alternating pinned and fixed)

Figure 6: Case study frames: (a) reference system; (b) optimized system.

The general properties for the case study frames are summarized in Table 4. The design and analyses were performed with RSTAB 8 [10] and SAP2000 [11]. Based on the presence of the reinforced concrete slabs, a rigid diaphragm constraint was considered for the nodes of each floor. The masses were calculated automatically from the applied gravitational loading on the structure. The importance of the second order effect was evaluated through the value of the inter-story drift sensitivity coefficient θ [12]. The coefficient θ was computed for each level, and the maximum value was found between $0.1< \theta \leq 0.2$. Consequently, second order effects were considered in an approximate way by multiplying the design values of the demands with the following factor: $\alpha=1/(1-\theta)$.

The cross-sections used for the lateral force resisting system of the reference and optimized structural systems are presented in Table 5. The design of the lateral force resisting system was governed by the deformation limitation (corresponding to the serviceability limit state – SLS, and/or to the ultimate limit state – ULS). In case of the optimized system, the coupling beams were modeled with alternating pinned/fixed connections with the aim to increase their flexibility and to avoid the formation of plastic hinges at serviceability limit state (SLS), ultimate limit state (ULS) and as much as possible at near collapse (NC) seismic intensity level. Beside the pinned/fixed connection solution, the coupling beams corresponding to floors 1÷4 were realized with S460 steel grade (Table 5). Reduced beam sections (RBS) were considered for the dissipative links for both structural systems. The P-Delta effect of the seismic masses that was not attributed to the frame was considered by employing a leaning column. The maximum usable behavior factor q was 2.0 for the reference system and 2.8 for the optimized system. Larger behavior factors were not exploitable (i.e. not needed), because stiffness requirements did not allow for a further decrease of cross sections.

Structural system	Reference	Optimized
Stories	8 @ 4 m	8 @ 4 m
Bays	3 @ (5.5 + 2.5 m)	2 @ (5.5 + 2.5 m) & 1 @ 8 m
Column base connection	pinned	pinned
Coupling beam connections	pinned at both ends	fixed at one end, pinned at the other end

Table 4: General properties of the case studies.

Reference system (T_1=*2.75 sec.*)			Optimized system (T_1=*2.56 sec.*)		
Floor	Links (S235)	Columns (S355)	Links (S235)	Coupling beams (S460 & S355)	Columns (S355)
8	HEA-200	HEB-450	HEA-200	HEB-400 (S355)	HEB-450
7	HEA-220	HEB-450	HEA-200	HEB-400 (S355)	HEB-450
6	HEA-220	HEB-450	HEA-200	HEB-400 (S355)	HEB-450
5	HEA-240	HEB-450	HEA-220	HEB-400 (S355)	HEB-450
4	HEA-240	HEB-450	HEA-240	HEB-400 (S460)	HEB-450
3	HEA-260	HEB-450	HEA-260	HEB-400 (S460)	HEB-450
2	HEA-260	HEB-450	HEA-280	HEB-400 (S460)	HEB-450
1	HEA-280	HEB-450	HEA-280	HEB-400 (S460)	HEB-450

Table 5: Cross-sections of elements for the reference and optimized system.

4 SEISMIC PERFORMANCE EVALUATION

The seismic performance was evaluated through: (i) nonlinear static analyses (pushover) using the N2 method [13]; and (ii) nonlinear dynamic analyses (time-history). Both types of analyses were performed using SAP2000 [11] and considering the 2D structural configurations illustrated in Figure 6. The N2 method [13] was used for determining the target displacements and the state of the two structural systems corresponding to the three seismic intensity levels. The response under a set of seven recorded accelerograms was evaluated through time-history analyses.

4.1 Nonlinear static analyses

The N2 method, developed by Fajfar [13] and comprised in Annex B of EN 1998-1 [7], can be used to verify the seismic performance of buildings designed by current methods (i.e. response spectrum analyses). The method combines the nonlinear static analysis of a multiple degrees of freedom system (MDOF) with an analysis based on the response spectrum of a single degree of freedom system (SDOF). The lateral story forces were assumed to be proportional to the 1st mode of vibration. Furthermore, the structural performance was evaluated for the limit states shown in Table 6, where "a_{gr}" is the reference peak ground acceleration of $0.3g$, and "a_g" represents the peak ground acceleration for a specific earthquake level.

Limit state	Return period, [years]	Probability of exceedance	a_g/a_{gr}	a_g/g
Damage Limitation (DL / SLS)	95	10% / 10 years	0.5	0.15
Significant Damage (SD / ULS)	475	10% / 50 years	1.0	0.30
Near Collapse (NC)	2475	2% / 50 years	1.7	0.51

Table 6: Limit states and corresponding scaling factors for the seismic input.

For beams subjected to flexure, based on FEMA 356 [14] and Annex B of EN 1998-3 [15], the following acceptance criteria were considered corresponding to the rotation at DL, SD and NC: $1{\cdot}\theta_y$, $6{\cdot}\theta_y$ and $8{\cdot}\theta_y$, where θ_y is the yield rotation. The damage state conditions of the plastic hinges associated to three limit states are similar in both codes, and it can be assumed that Immediate Occupancy (IO) corresponds to Damage Limitation (DL), Life Safety (LS) stands for Significant Damage (SD), and Collapse Prevention (CP) relates to Near Collapse (NC).

The plastic hinge definitions for the Dissipative Replaceable Links (see Table 7) were based on the parameters reported in [2].

Point	$M/M_{pl,RBS}$	$\theta/\theta_{pl,,RBS}$	Acceptance criteria	
A	0	0	Limit state	($\theta/\theta_{pl,RBS}$)
B	1	0	IO	15
C	1.27	40	LS	20
D	0.6	40	CP	35
E	0.6	45		

Table 7: Non-linear hinge parameters considered for the Dissipative Replaceable Links.

From the nonlinear static analyses, a pushover curve (capacity curve) was obtained for each structure. Further, using the N2 method [13], the target displacements (D_t) were obtained for the two structural systems corresponding to each of the three seismic intensity levels (i.e. SLS, ULS, NC). The obtained target displacement values are summarized in Table 8. As can be observed, very similar values were obtained for the two structures.

	Limit state	a_g/a_{gr}	Reference structure	Optimized structure
	DL / SLS	0.5	0.157	0.157
D_t [m]	SD / ULS	1.0	0.314	0.315
	NC	1.7	0.533	0.535

Table 8: Target displacements of reference and optimized structures.

The outcomes of the nonlinear static analyses are shown in terms of: (i) capacity curve (base shear force vs. top displacement); (ii) the state of the lateral load resisting frames corresponding to the three seismic intensity levels (SLS, ULS, NC), i.e. at the computed target displacements. It is to be noted that the target displacements are marked on each capacity curve. The outcomes are presented for the following configurations: (i) Reference structure (see Figure 7); (ii) Optimized structure (see Figure 8).

Based on the state of each structural configuration (plotted for the three seismic intensity levels – SLS, ULS and NC), the following observations can be made:

- at SLS – both structural configurations were characterized by an elastic response;
- at ULS – the reference structure was characterized by an elastic response, while plastic hinges developed in the dissipative links at the first five stories of the optimized structure;
- at NC – both structural systems evidenced plastic hinge development in the dissipative links, but the rotations did not reach the life safety or collapse prevention levels; in case of the optimized structure, the coupling beams did not develop plastic hinges.

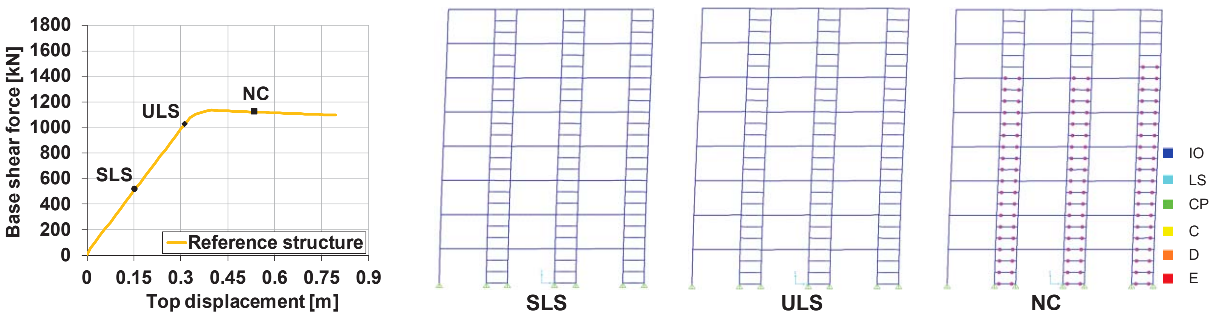


Figure 7: Reference structure: capacity curve and the state of the frame at the three seismic intensity levels – SLS, ULS, NC (i.e. a_g/a_{gr}=0.5; 1.0; 1.7).

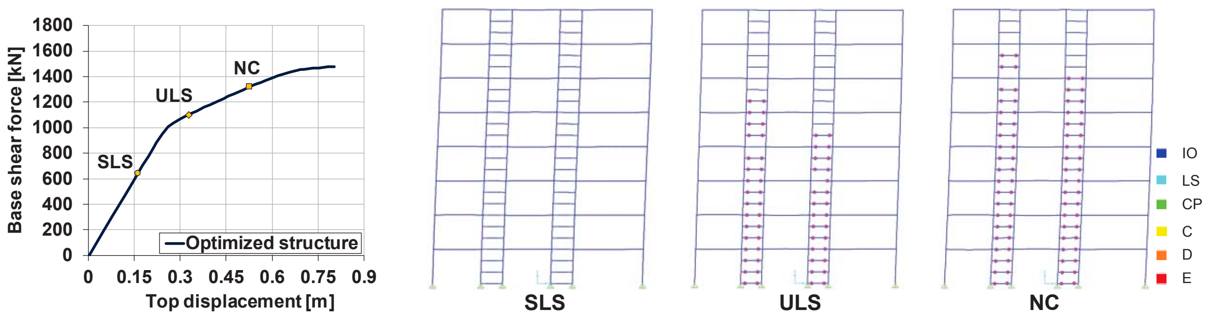
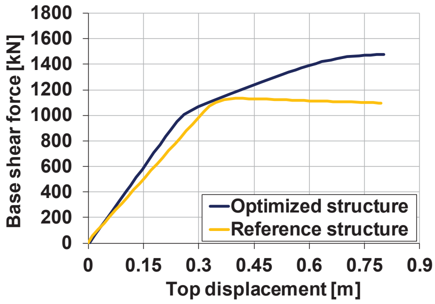


Figure 8: Optimized structure: capacity curve and the state of the frame at the three seismic intensity levels – SLS, ULS, NC (i.e. a_g/a_{gr}=0.5; 1.0; 1.7).

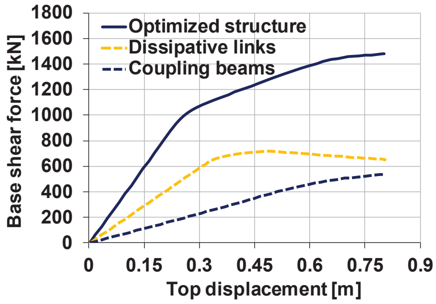
A comparison between the pushover curves of the two structural systems, i.e. reference and optimized, is shown in Figure 9a. As can be observed, the initial stiffness and the post-yield response of the two structural systems evidence some differences:

- The reference structure (for which the coupling beams were pinned at both ends) showed a relative lower initial stiffness. In the post yield region, the pushover curve was characterized by a horizontal tendency (i.e. large deformations with little decrease of force).
- In case of the optimized structure (for which the coupling beams were modeled with alternating pinned/fixed connections), the pushover curve evidenced a linear ascending tendency in the post yield region (i.e. capacity increases with the deformation).

The improved response of the optimized structures, i.e. bilinear shape with a linear ascending tendency in the post yield region, can be explained by examining the shape of the pushover curves of the two subsystems (i.e. coupling beam frames, dissipative link frames). Subsequently, the contribution of each of the two subsystems is illustrated in Figure 9b. Compared to the link-frame subsystems, the coupling beam frames are characterized by a linear-elastic response, with a lower stiffness and capacity. The linear-elastic response of coupling beam frames is therefore responsible for the linear ascending tendency in the post yield region of the optimized structural system and confirms the potential for improved performance and increased re-centering capacity.



(a)



(b)

Figure 9: (a) Comparison between the pushover curves of the reference and optimized structural systems; (b) response/contribution of the dissipative link frames and coupling beam frames.

4.2 Nonlinear dynamic analyses

The two systems were further analyzed by means of dynamic time history analyses. A set of seven accelerograms matching the target spectrum has been chosen from the PEER NGA online ground motion database [16]. The scaling factors have been set such, that requirements according to the current draft of the revision of Eurocode 8 are met. These are as follows:

- Scaling factors not larger than *2.0*, nor smaller than *0.5*.
- The period range of interest has been set to $0.2 \cdot T_{1,min}$ to $1.5 \cdot T_{1,max}$, whereby $T_{1,min}$ and $T_{1,max}$ are the minimum and maximum fundamental periods of the considered frames.
- Within this period range, the ratio of mean to target spectrum should be within the range of *0.75* to *1.30*. Moreover, the average of this ratio over the considered period range should be larger than *0.95*. Also, for each individual spectrum the ratio should nowhere be below *0.5* within that range.
- Not more than two records of the same earthquake should be used.

The final suite of seven accelerograms is listed within Table 9. Each record can be correlated to the PEER data base with the Record Sequence Number (RSN). Figure 10 shows the target, mean and individual spectra along with a vertical line, indicating where the period range of interest begins. The first mode periods T_1 for the reference and optimized structure correspond to 2.75 s and 2.56 s, respectively. The 50%, 75% and 130% scaled target spectrum used for ground motion selection is also indicated in the figure.

#	RSN	Scale factor	R [km]	M	Earthquake name	Year	Station
1	6	1.11	6.1	6.95	Imperial Valley-02	1940	El Centro Array #9
2	15	1.95	38.7	7.36	Kern County	1952	Taft Lincoln School
3	20	1.32	26.9	6.5	Northern Calif-03	1954	Ferndale City Hall
4	30	1.65	9.6	6.19	Parkfield	1966	Cholame-Shandon #5
5	68	1.70	22.8	6.61	San Fernando	1971	Hollywood Stor FF
6	95	1.38	3.8	6.24	Managua Nicaragua-01	1972	Managua ESSO
7	96	1.85	4.7	5.2	Managua Nicaragua-02	1972	Managua ESSO

Table 9: Recorded acceleration time histories used for dynamic analyses.

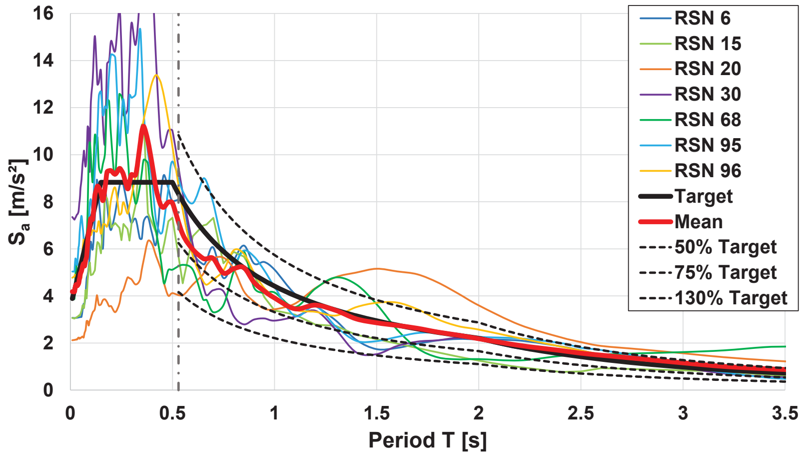
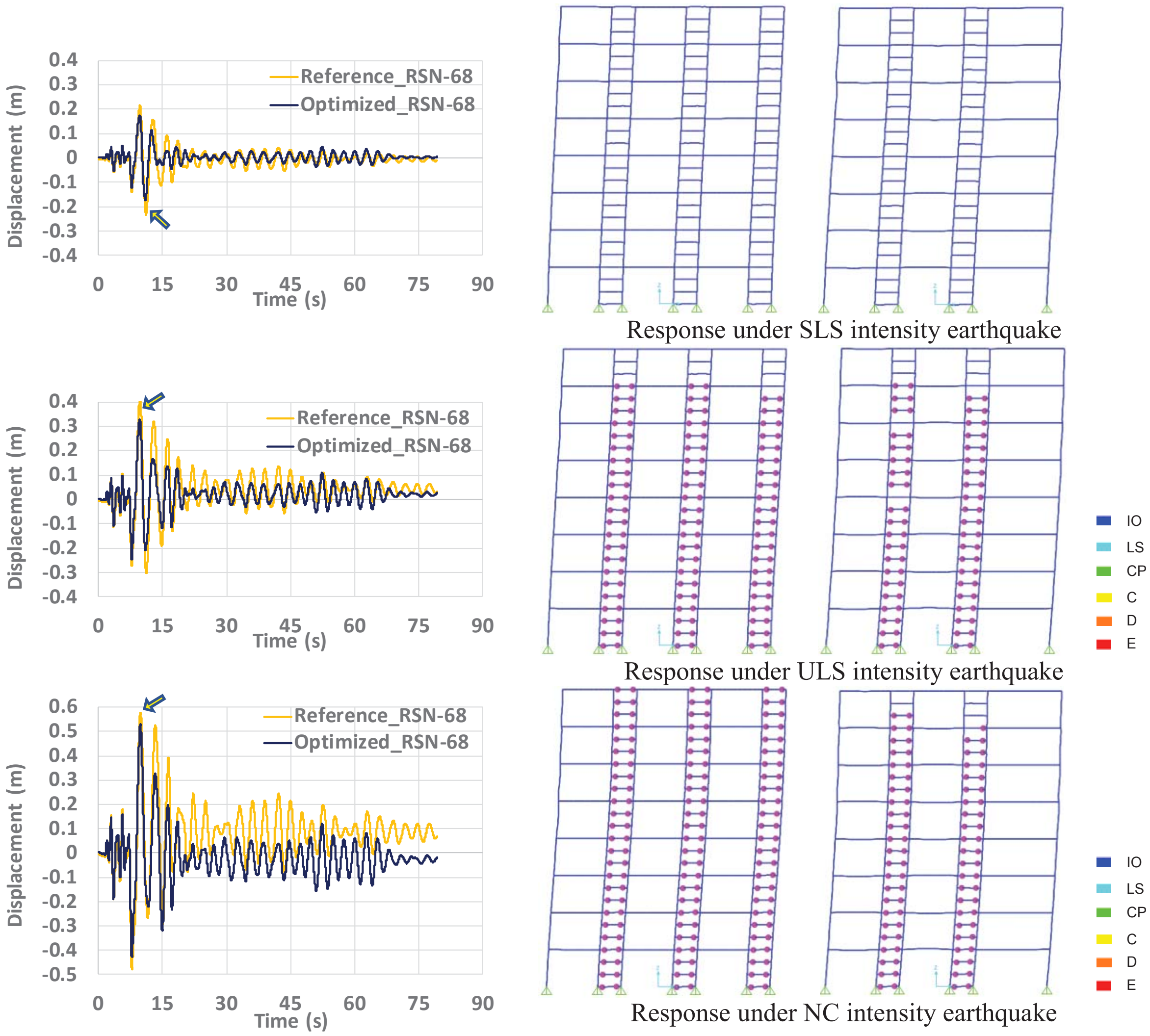


Figure 10: Elastic response spectra of mean and individual accelerograms compared to the target spectrum.

The outcomes of the time-history analyses are presented for both reference and optimized structural configuration in terms of: (i) comparison of the displacement at top floor; (ii) state of frame corresponding to the maximum displacement (i.e. scaled deformed shape with marked plastic hinges). Consecutively, the response under one of the seven accelerograms (i.e. #5 – *RSN 68*) at the three intensity levels (SLS, ULS and NC) is shown in Figure 11.

As can be observed, the time-history analyses evidenced the following:

- *at SLS (serviceability limit state – a_g/a_{gr}=0.5)*: ▪ plastic hinges did not develop in any of the two structural systems (reference or optimized); ▪ the comparison between displacements at top floor showed similar deformations – with slightly lower values corresponding to the optimized structure;
- *at ULS (ultimate limit state – a_g/a_{gr}=1.0)*: ▪ plastic hinges developed in the dissipative links (at seven out of eight stories) for both structural configurations; ▪ the comparison between displacements at top floor showed some differences, i.e. lower deformations corresponding to the optimized structure; ▪ the rotations within plastic hinges did not reach the life safety or collapse prevention levels;
- *at NC (near collapse – a_g/a_{gr}=1.7)*: ▪ a global plastic mechanism was evidenced with plastic hinges in the dissipative links for both structural systems; ▪ the comparison between displacements at top floor showed significant differences between the reference and optimized structure for five out of seven accelerograms which lead to residual deformations (i.e. *#1,3,5,6,7*); ▪ for both structural systems, the replaceable dissipative links did not evidence rotations corresponding to life safety or collapse prevention levels; ▪ the optimized structure evidenced plastic hinges in the coupling beams only for one out of seven accelerograms (i.e. *#3 – RSN 20*).



(a) Comparison of the response (b) Reference structure (c) Optimized structure

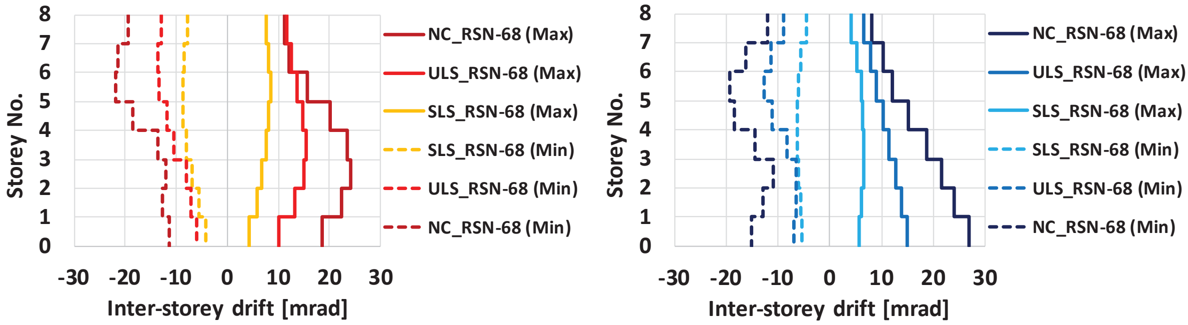
Figure 11: Outcomes of time-history analyses: (a) comparison of the response (top displacement vs. time); (b) & (c) state of reference and optimized structures.

The nonlinear dynamic analyses also allowed to evaluate the inter-story drifts corresponding to each: (i) structural configuration (reference / optimized); (ii) accelerogram (*#1 to #7*); (iii) seismic intensity level (SLS, ULS, NC).

Exemplarily, Figure 12 illustrates the inter-story drifts generated by one of the accelerograms (*#5 – RSN68*) within the two systems. In general, largest inter-story drifts occur at the mid-floors for the reference system, whereby the optimized system is characterized by larger drifts at the bottom and small ones at the top. Besides the lower floors, inter-story drifts tend to be less large for the optimized system compared to the reference structure.

The average of the maximum (+/-) inter-story drift values for the reference and optimized structures, corresponding to each seismic intensity level (SLS, ULS, NC), was computed from the outcomes of the set of seven selected accelerograms listed in Table 9. The average values summarized in Table 10 confirm the observations mentioned above regarding the minimum inter-story drifts also for other accelerograms. The maximum inter-story drifts occurred at 6^{th}÷7^{th} floor for the reference structure, respectively at 5^{th}÷6^{th} floor for the optimized structure.

In Figure 13 the average inter-story drifts between reference and optimized structure can be compared. Besides the first two floors, the optimized structure showed in general lower drifts. Moreover, the inter-story drift profile is more uniform throughout the height.

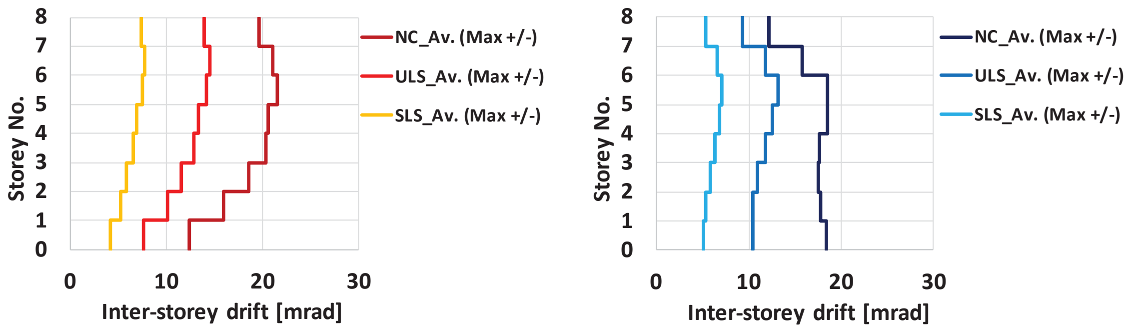


(a) (b)

Figure 12: Inter-story drifts generated by *#5 – RSN68* within the corresponding structural systems: (a) reference structure; (b) optimized structure.

Story	Reference structure			Optimized structure			Ratio optimized/reference		
	SLS	ULS	NC	SLS	ULS	NC	SLS	ULS	NC
8	7.4	13.9	19.6	5.3	9.3	12.2	0.72	0.67	0.62
7	**7.7**	**14.5**	21.0	6.5	11.8	15.8	0.84	0.81	0.75
6	7.5	14.1	**21.5**	**7.1**	**13.2**	18.5	0.95	0.94	0.86
5	6.9	13.4	20.5	6.8	12.6	**18.6**	0.99	0.94	0.91
4	6.5	12.9	20.3	6.4	11.8	17.6	0.98	0.91	0.87
3	5.8	11.5	18.6	5.8	10.9	17.5	1.00	0.95	0.94
2	5.2	10.1	15.9	5.3	10.5	17.7	1.02	1.04	1.11
1	4.1	7.6	12.3	5.1	10.4	18.4	1.24	1.37	1.50

Table 10: Average of the maximum inter-story drifts [mrad] corresponding to the three seismic intensity levels (SLS, ULS, NC) and the two structural configurations (reference / optimized).



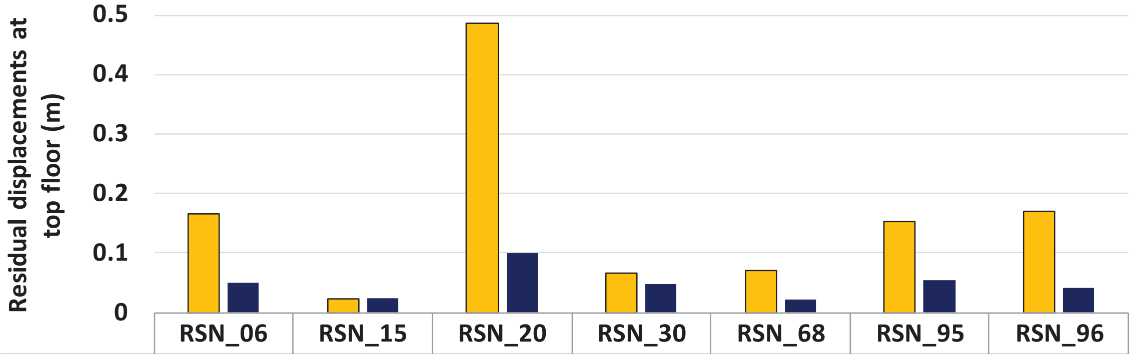
(a) (b)

Figure 13: Average of the maximum (+/-) inter-story drifts corresponding to the three seismic intensity levels (SLS, ULS, NC) and the two structural configurations: (a) reference; (b) optimized.

The response of the two structural systems was evaluated by comparing the residual deformations. Figure 14 shows the residual deformation at the top floor corresponding to each of the seven accelerograms at NC intensity level for both structural configurations. As can be observed, the largest residual deformations were evidenced for *#3 – RSN 20*. Furthermore, for five out of seven accelerograms (*RSN 6 / 20 / 68 / 95 / 96*) the residual deformations of the optimized structure were approximately by two thirds lower compared to the reference structure.

Regarding the seismic performance and resilience of a structural system, some important aspects are represented by: (i) the possibility of structural repair; (ii) a low amount of residual deformations; (iii) the re-centering capability of the structure. Table 11: Reference vs. optimized structural configuration: fulfilment of resilience/seismic performance criteria shows an overview with regard to the fulfillment of the resilience and seismic performance criteria by the two structural systems (reference vs. optimized). Through the use of replaceable dissipative links, potentially both structural systems have the possibility to be repaired after a seismic event. In contrast to the reference structure (with relatively high residual deformations, and no re-centering capability), the optimized structural system fulfils all three criteria:

- *Structural repair* – through the use of replaceable dissipative devices;
- *Low residual deformations* – through the use of coupling beams of high strength steel and with alternating pinned/fixed connections (for an elastic response);
- *Re-centering capability* – by incorporating the two bays with coupling beams (of high strength steel and pinned/fixed connections) characterized by an elastic response, which will contribute to the structural re-centering during the replacement of the dissipative links.



	RSN_06	RSN_15	RSN_20	RSN_30	RSN_68	RSN_95	RSN_96
Reference	0.166	0.024	0.485	0.066	0.070	0.152	0.171
Optimized	0.050	0.024	0.099	0.048	0.021	0.055	0.041

Figure 14: Reference vs. optimized structural system: residual displacements at the top floor corresponding to each of the seven accelerograms (NC seismic intensity level).

Performance criterion	Reference structure	Optimized structure
Structural repair	***Yes***: both structural systems are generally reparable through the use of Dissipative Replaceable Links (DRL)	
Low residual deformations	***No***: the reference structure is characterized by relatively high residual deformations	***Yes***: the optimized structural system is characterized by a significant reduction of residual deformations, resulting from the use of the two bays with pinned-fixed coupling beams
Re-centering capability	***No***: the reference structure does not possess any worth mentioning re-centering capability	***Yes***: the optimized structure contains two bays with pinned-fixed coupling beams that responded elastically, and therefore, would contribute to the re-centering of the structure during the replacement of the dissipative links

Table 11: Reference vs. optimized structural configuration: fulfilment of resilience/seismic performance criteria.

5 CONCLUSIONS

The preliminary investigations on the newly developed CDRLF system within this paper show its general feasibility and potential of enhancing the already known DRLF system regarding stiffness increase and improved re-centering capability for mid to high rise buildings. Thus, it renders an alternative to application of partial-strength joints to the gravity load bearing frame of a system with DRLFs, as conventionally done for introducing additional stiffness. Main conclusions derived from the investigated case study are as follows:

- The design philosophy was shown to work effectively for the considered case study. Damage to coupling beams was not triggered up to NC and beyond, while almost all of dissipative links showed plastic response.
- The optimized system, two DRLFs with 'one end fixed – the other end hinged' coupling beams, showed a significant improvement over the reference system with three DRLFs.
- The improvement consists of:
  - the decrease in needed amount of DRLF bays (two compared to three),
  - an improved global force displacement relationship (introduced hardening slope after damage to dissipative links),
  - lower inter-story drifts,
  - noteworthy re-centering capability and, thus,
  - much lower residual displacements.
- This optimization results in a system that fully complies with the envisaged performance targets, which is repairability by restricting damage to dissipative replaceable links only, limiting of residual displacements and providing re-centering capability by the coupling beams.
- To achieve the performance targets – widespread damage to dissipative links before damage to coupling beams is triggered – a balanced stiffness distribution between the two DRLFs and the coupling frames is necessary. The coupling frame needs to be stiff enough, to increase noteworthy overall stiffness, but on the same time flexible enough, to prevent attraction of too much force, once the DRLFs soften du to energy dissipation.
- The balance of the stiffness was achieved by introducing a hinge at one end of each coupling beam.
- Sophisticated static nonlinear (pushover) are needed to assure that the foreseen performance targets are met. Simple application of current seismic design equations for designing the CDRLF system does not necessarily guarantee a well performing structure. Therefore, simplified design equations especially for the coupling beams are needed to ease application of the CDRLF system in practice.
- The strength design of the coupling beams might be challenging, especially when both coupling beam ends are designed to be fixed. Thus, often coupling beams in high strength steel will be required, as the increase of section size would jeopardize the need of a balanced stiffness distribution between DRLFs and CFs.

Finally, it can be stated that the CDRLF system is currently under investigation within the ongoing DISSIPABLE project. The study presented in this research shows promising results. A testing campaign consisting of hybrid tests on a 2D frame as well as shaking table tests on a 3D structure is underway. Eventually, the project consortium will deliver design guidelines along with a worked example.

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