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RE-CENTRING DUAL ECCENTRICALLY BRACED FRAMES WITH REMOVABLE LINKS

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Abstract. Conventional seismic design philosophy is based on dissipative response, which implicitly accepts damage of a structure under the design earthquake load, which results in significant economic losses. Repair of a structure is often impeded by its permanent (residual) drifts. The repair costs and downtime of a structure hit by an earthquake can be significantly reduced by adopting removable dissipative members and providing the structure with the re-centring capability. These two concepts were implemented in a dual structure, obtained by combining steel eccentrically braced frames with removable bolted links and moment resisting frames. Firstly, the paper summarizes the results of a large-scale experimental program on a dual eccentrically braced frame with replaceable links performed at the European Laboratory for Structural Assessment (ELSA) at the Joint Research Centre (JRC) in Ispra within the framework of Transnational Access of the SERIES Project. Secondly, a test based calibrated numerical model is applied via pushover and time-history analyses with the purpose to assess the seismic performance of these innovative structures.

Key words: re-centring, bolted links, eccentrically braced frames.

1. INTRODUCTION

Self-centring systems, a relatively new concept that addresses the drawbacks of the conventional yielding systems, have received much attention recently. Among many practical implementations, the following are most relevant to our study: self-centring moment-resisting frames with post-tensioned beam to column joints [1] and column bases [2], self-centring concentrically braced frames [3].

An alternative solution is to provide re-centring capability (as opposed to self-centring), by removable dissipative members and dual (rigid-flexible) structural configuration.

A dual eccentrically braced frame (EBF) with removable dissipative links proposed by authors is shown in Fig. 1 [4]. The link to beam connection is done by a flush end-plate and high-strength friction grip bolts. The main advantage over other dissipative devices is that removable links can be designed using methods already available to structural engineers and can be fabricated and installed using standard procedures.

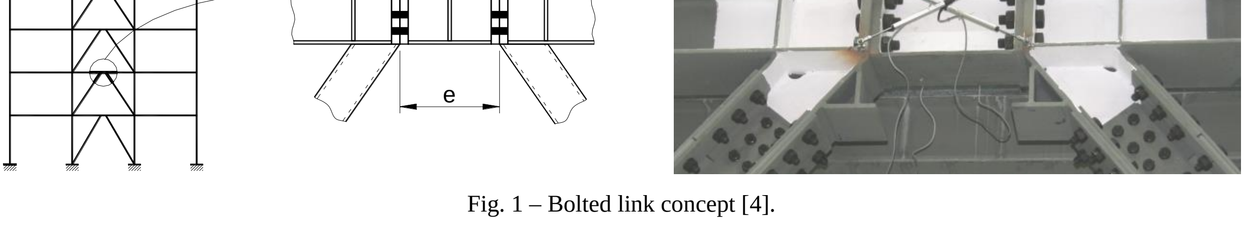


Fig. 1 – Bolted link concept [4].

The re-centring of the system is attained by designing the structure as a combination of eccentrically braced frames (EBFs) and moment-resisting frames (MRFs). The elastic response of the flexible subsystem (MRF) provides the restoring forces, once the links damaged during an earthquake are removed. For this principle to be efficient the flexible subsystem should remain in the elastic range. Standard capacity design principles can be used to meet this objective. However, nonlinear structural analysis is advised. To ensure the elastic response of the flexible subsystem some members could be made in high-strength steel [5]. Additionally, the damaged links should be reasonably easy to remove. If the link deformations are small, links could be simply unbolted and then removed. In case of large residual link deformations, unbolting may prove difficult. In such a case the residual deformations should first be released by flame cutting the link.

2. SUMMARY OF THE EXPERIMENTAL INVESTIGATION

A full-scale experimental investigation on a dual eccentrically braced frame with replaceable links was performed at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) in Ispra, Italy. Its objectives were to: (1) validate the re-centring capability of dual structures with removable dissipative members (links), (2) assess overall seismic performance of dual eccentrically braced frames and (3) obtain information on the interaction between the steel frame and the reinforced concrete slab in the link region. The prototype structure had 3 spans of 6 meters and 5 bays of 6 meters, and 3 storeys with the height of 3.5 meters each. The main lateral load resisting system is composed of eccentrically braced frames. Additionally, there are 4 moment resisting frames in transversal directions and 10 moment resisting frames in longitudinal (test) direction, to assure the restoring forces after an earthquake. Considering that in the transversal direction the lateral force resisting system is located on the perimeter frames only, and in order to reduce the cost of the experimental campaign, the test structure is composed of the two end frames only (Fig. 2a).

The test structure was devised in order to study two alternative solutions of the slab – link interaction. In the south frame the removable link was disconnected from the reinforced concrete slab, by an additional secondary beam placed in parallel with the beam containing the link (Fig. 2b). A conventional solution was adopted in the north frame, where the slab was cast over links, but no shear studs were used in the link region.

(a) (b)

Fig. 2 – The experimental mock-up in front of the reaction wall (a) and plan layout (b) of the test structure.

The prototype structure was designed according to EN1990 [6], EN1991 [7], EN1992 [8], EN1993 [9], EN1994 [10] and EN1998 [11]. Gravity loads of 4.9 kN/m^2 (permanent load) and 3.0 kN/m^2 (live load) were considered. The structure was assumed to be located in an area characterised by 0.19g peak ground acceleration and stiff soil conditions (EC8 spectrum for soil type C). A behaviour factor q = 4 (ductility class M) and inter-storey drift limitation of 0.0075 of the storey height were assumed. The structural steel components were designed using S355 grade steel, with two exceptions. Grade S460 steel was used for the columns, in order to obtain a larger capacity without increasing the stiffness. This approach helps promote elastic response of non-dissipative components. Links were designed using S235 grade steel (which was replaced during fabrication with the equivalent DOMEX 240 YP B).

The record selected for pseudo-dynamic (PsD) tests was 15613_H2 earthquake (recorded at Yarimca (Eri) station during the Izmit aftershock event in Turkey on 13.09.1999). With reference to the performance levels shown in Table 1, Table 2 summarises the sequence of PsD tests and link replacements performed on the test structure.

Table 1

Limit states and corresponding scaling factors for seismic input

Limit state	Mean return period, years	Probability of exceedance	a_g/a_{gr}	a_g, g
Full operation (FO)	–	–	0.062	0.020
Damage Limitation (DL)	95	10% / 10 years	0.59	0.191
Significant Damage (SD)	475	10% / 50 years	1.00	0.324
Near Collapse (NC)	2475	2% / 50 years	1.72	0.557

Table 2

Sequence of PsD and link removal tests

Links	Test	Scope	Links
Set 1	Full operation (FO1)	Assess elastic response of the structure.	Full operation (FO1)
	Damage Limitation (DL)	Observe structural response to a moderate earthquake.	Damage Limitation (DL)
	Replacement of set 1 links (LR1)	Investigate removal of links by unbolting and their replacement.	Replacement of set 1 links (LR1)
Set 2	Full operation (FO2)	Assess elastic response of the structure.	Full operation (FO2)
	Significant Damage (SD)	Observe structural response to a design-level earthquake.	Significant Damage (SD)
	Pushover (PO1)	Induce large permanent drifts.	Pushover (PO1)
	Replacement of set 2 links (LR2)	Investigate removal of links by flame cutting and their replacement.	Replacement of set 2 links (LR2)
Set 3	Full operation (FO3)	Assess elastic response of the structure.	Full operation (FO3)
	Near collapse (NC)	Observe structural response to a severe-level earthquake.	Near collapse (NC)
	Pushover (PO2 and PO3)	Investigate ultimate capacity of the structure.	Pushover (PO2 and PO3)

Since the test campaign and the results are presented in detail in JRC-ELSA Report [12], hereafter, the main conclusions of tests are only summarized. Tested dual eccentrically braced structure exhibited excellent performance when subjected to seismic input with intensities corresponding to 95 and 475 years return period, corresponding to Damage Limitation and Significant Damage limit states, respectively. Small residual deformations were recorded for both seismic intensity levels, which were comfortably within the erection tolerance limits. Such small permanent deformations effectively mean that the structure is self-centring to a degree, which allows for an easy repair of the structure by the replacement of damaged links. Residual drifts are further reduced by removing and replacing the bolted links. Re-centring was better for the frame with links disconnected from the slab. Moreover, damage to the concrete was avoided in this case. Nevertheless, good re-centring was observed even for the frame with the slab cast over the links, while damage to the reinforced concrete slab was insignificant at the Damage Limitation and Significant Damage tests. Provided the residual deformations after an earthquake are small, the links can be removed simply by untightening, as was demonstrated after the Damage Limitation test. If larger residual drifts occur, flame cutting of links is recommended to allow for smooth release of forces, as was done after the Significant Damage series of tests. The experimental investigation validated the re-centring capability of dual eccentrically braced frames with removable links, which was accomplished without major technological difficulties.

3. NUMERICAL MODEL

In order to obtain a global numerical model that can be used in further post-test numerical simulations on current practice dual eccentrically braced frames with removable links, a calibration was performed based on experimental results of pseudo-dynamic tests of the DUAREM testing programme [12].

A time-history analysis was performed using SeismoStruct program [13], applying at each of the three levels of the DUAREM specimen model, the displacements obtained from the experimental tests.

Force-based plastic hinge elements for beams, columns and braces were used. Bolted links were modelled using a force-based inelastic beam element with rotational springs at the ends (see Fig. 3). The rotational springs were modelled using the smooth hysteresis curve (as defined by SeismoStruct) for link elements, having the ratio between ultimate shear force (V_u) and yield shear force (V_y) equal to 1.7 and the ultimate shear deformation $\gamma_u = 0.15$ rad.

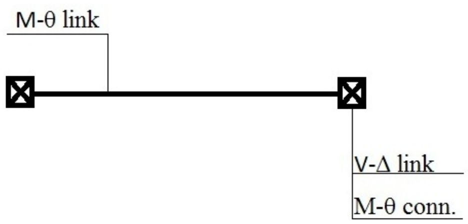


Fig. 3 – Link model.

The results of numerical simulations and experimental testing were compared in terms of first storey link rotation versus shear force and drift versus base shear force (see Fig. 4). A very good match was observed, the new calibrated numerical model being adopted for further numerical simulations on current practice structures.

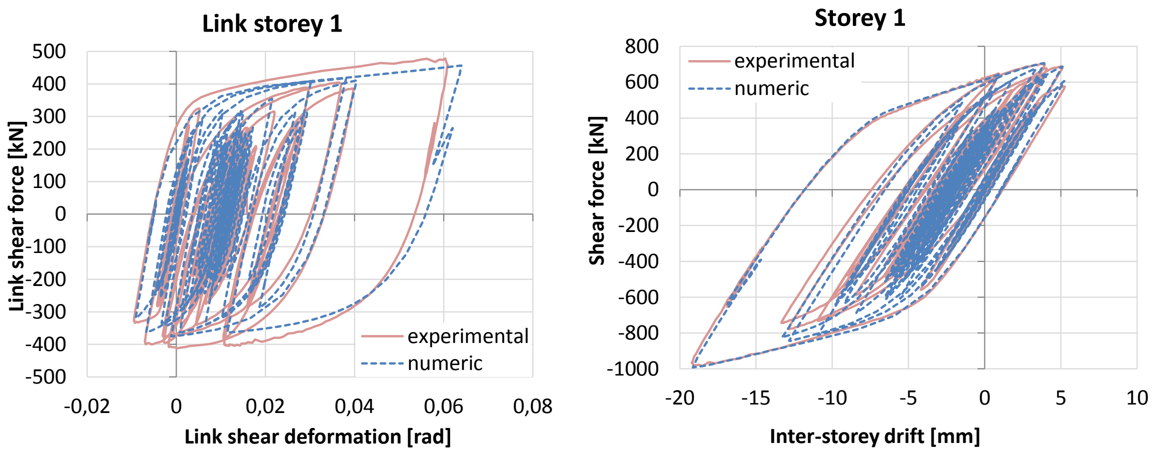


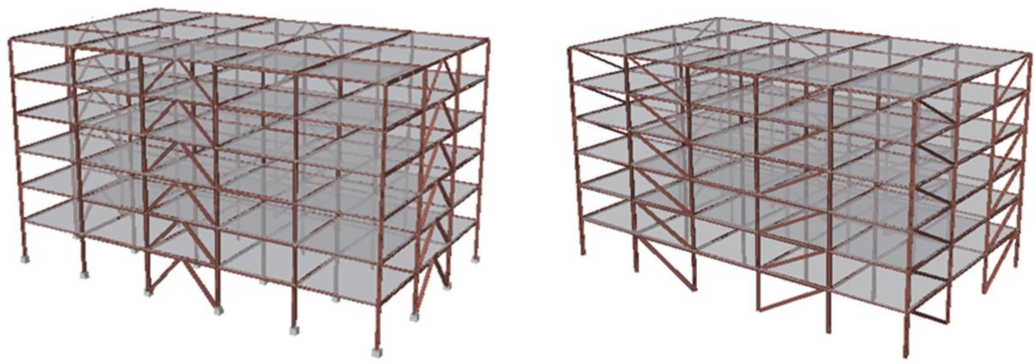
Fig. 4 – First storey – link and global hysteresis.

4. ANALYZED STRUCTURES

In order to validate numerically the design methodology of current practice EBF structures with removable links and the link replacement procedure, two dual structural configurations were designed: the first, like in the case of the experimental specimen [12], having a central EBF (the link being placed at mid-span) and two side MRFs, being further referred to as "configuration A" (Fig. 5a) and the second having a central MRF and two side EBFs (the links being placed marginally, connected to columns on one side) being further referred to as "configuration B" (Fig. 5b). The links from EBFs were conceived as removable (bolted) dissipative elements because they are intended to provide the energy dissipation capacity and to be easily replaceable. The more flexible moment resisting frames provide the necessary re-centring capability to the structure.

Both structures have 3 spans of 7.5 meters and 5 bays of 7.5 meters, and 6 stories of 3.5 meters each (4.0 m at the ground level). The main lateral load resisting system is composed of eccentrically braced frames (2 on each horizontal direction for structure A and 4 on each horizontal direction for structure B).

Additionally, there are 4 moment resisting frames on each horizontal direction in case of configuration A and 2 moment resisting frames on each horizontal direction in case of configuration B, to assure the restoring forces after an earthquake. All the other frames are gravitational loads resisting systems (with pinned composite steel-concrete beams).



(a) (b)

Fig. 5 – Configuration A (a) and configuration B (b) structures.

The capacity design of the structure was carried out according to European codes. A 4.9 kN/m^2 dead load and 3.0 kN/m^2 live load were considered. The building was analysed for stiff soil conditions (EC8 type 1 spectrum for soil type C), characterized by 0.35g peak ground acceleration. A behaviour factor q = 4 (ductility class M) and inter-storey drift limitation of 0.0075 of the storey height are used.

The perimeter columns are fixed at the base and all the central ones (resisting to gravity loads only) are pinned at the base. Short links were used, with the length e = 800 mm and with equivalent stiffness reduced from the theoretical shear stiffness of continuous links (to account for the shear stiffness of the link and rotational deformations in the bolted connection), made from welded H sections. All the structural elements from EBFs are made from mild carbon steel S355, while the ones from MRFs are made from high-strength steel S460.

The element sections presented in Table 3 and Table 4 were obtained.

Table 3

Elements sections for configuration A

Storey	Links	EBF beams	Braces	EBF columns	MRF beams	MRF columns
1	640x260x22x12	HEA650	HEM300	HEM340	IPE450	HEB340
2	590x260x22x10	HEA600	HEM280	HEM340	IPE400	HEB340
3	540x240x22x10	HEA550	HEM260	HEB340	IPE360	HEB300
4	490x230x22x9	HEA500	HEM240	HEB340	IPE360	HEB300
5	440x230x20x8	HEA450	HEM220	HEB300	IPE300	HEB300
6	390x210x16x6	HEA400	HEM180	HEB300	IPE300	HEB300

Table 4

Elements sections for configuration B

Storey	Links	EBF beams	Braces	EBF columns	MRF beams	MRF columns
1	540x260x20x9	HEA550	HEM300	HEM340	IPE600	HEB340
2	490x230x20x8	HEA500	HEM280	HEM340	IPE550	HEB340
3	440x230x20x8	HEA450	HEM260	HEM340	IPE550	HEB300
4	390x230x20x8	HEA400	HEM240	HEM340	IPE500	HEB300
5	350x230x18x8	HEA360	HEM240	HEB340	IPE500	HEB300
6	330x190x15x5	HEA340	HEM200	HEB340	IPE400	HEB300

5. SEISMIC PERFORMANCE ASSESSMENT

For a structure with re-centring capability, the design objective consists in preventing yielding in members other than removable dissipative ones, up to a desired deformation. Ideally the latter should be the ultimate deformation capacity of the removable dissipative member. From a preliminary pushover analysis, it was observed that following code-based capacity design rules was not enough to accomplish the objective stated above for the investigated structures. But, using S690 higher-strength steel in moment resisting frames was shown to be efficient in avoiding yielding in the their members, increasing their strength, but not the stiffness, this being the material used in further analyses.

5.1. Nonlinear static analyses

The calibrated 2D numerical models of the designed structures were subjected to pushover analyses on transversal direction, with a modal (inverted triangular) distribution of lateral forces, in order to assess their structural performance.

The pushover curves presented in Fig. 6 illustrate the seismic performance of the two structures.

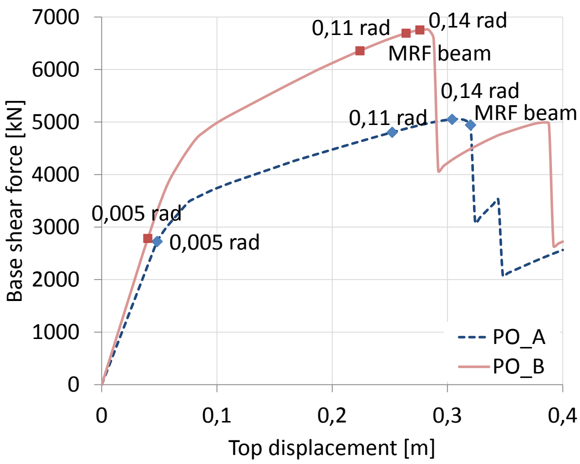


Fig. 6 – Pushover curves.

The objective of having no yielding in the MRFs before the attainment of the ULS deformation in the removable links (0.11 rad plastic rotation according to FEMA356 [14]) of the EBFs is accomplished, representing the basic design requirement for dual frames with removable dissipative members. MRFs provide the re-centring of the structure even until after the links ultimate deformation (0.14 rad plastic rotation according to FEMA356) in case of configuration A.

5.2. Nonlinear dynamic analyses

Seven natural seismic records of European earthquakes were used for seismic performance assessment of the test structure using nonlinear time-history analysis. Individual records were scaled to the target spectrum (EN 1998 type 1, soil type C, a_g = 0.35g) using EN 1998 criteria.

Structural performance was evaluated for the limit states shown in Table 5, where a_{gr} is the reference peak ground acceleration (corresponding to 10% / 50 years earthquake) and a_g represents the peak ground acceleration for a specific earthquake level.

Table 5

Limit states and corresponding scaling factors for seismic input.

Limit state	Return period (years)	Probability of exceedance	a_g/a_{gr}
Damage Limitation (DL / SLS)	95	10% / 10 years	0.59
Significant Damage (SD / ULS)	475	10% / 50 years	1.00
Near Collapse (NC)	2475	2% / 50 years	1.72

At the Damage Limitation (DL) limit state all structural components except links are in the elastic range. Shear deformations in links exceed the FEMA 356 limits of 0.005 radians (0.039 rad for structure A and 0.026 rad for structure B), but this is normal, since the design carried out according to EN 1998 did not impose any limitation on yielding of structural members at DL limit state. Even so, peak inter-storey drifts are within the limits imposed (< 0.75%). In these conditions, the advantage of the proposed system is obvious, since the damaged dissipative members (links) can be replaced easily due to negligible permanent drifts. Even if some structural damage is present, it can be repaired easily by replacing the bolted links.

At the Significant Damage (SD) limit state damage is still constrained to links only, which exhibit plastic deformation demands below 0.11 rad (0.092 rad for structure A and 0.078 rad for structure B). Permanent drifts are only slightly larger than at the DL limit state. Due to low permanent drifts, the structures are easily repairable at this limit state as well.

At the Near Collapse (NC) limit state the structural damage is widespread. Shear deformations in links are well over acceptable values of 0.14 rad (0.282 rad for structure A and 0.266 rad for structure B). However, due to moment resisting frames, the overall performance of the structures can be considered acceptable for this limit state. Plastic deformation demands are present in moment resisting frames (beams and columns) and braces. Even so, permanent inter-storey drifts are not very large. However, repairing of the structures is considered not to be feasible and desirable at these large levels of seismic input.

6. LINK REPLACEMENT INVESTIGATION

As shown in the DUAREM Report [6], the technically easiest way to release the forces in links is by flame cutting the web and flanges of the link [15] if large permanent drifts occur or by unbolting otherwise, on a storey by storey basis [16].

In order to numerically simulate the link removal order, the structures were subjected to a uniform distribution of lateral forces up to the attainment of 0.11 rad plastic rotation in links on the transversal direction, simulating the seismic action. Then the structures were unloaded, simulating the state of the structure after an earthquake and links are removed level by level.

Three possibilities of links removing order within a storey were studied: firstly removing the links on the longitudinal direction and secondly the ones on the transversal direction (Fig. 7a,d), *vice versa* (Fig. 7b,e), and in a circular pattern (Fig. 7c,f).

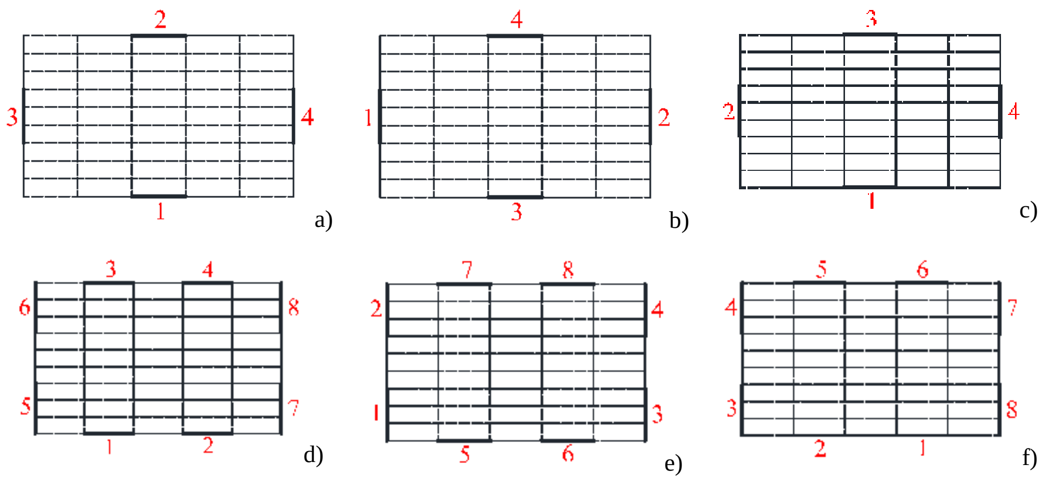


Fig. 7 – Link removal solution for configuration A (a to c) and for configuration B (d to e).

It was observed that for the first version (Fig. 7a,d) the residual shear force drop is about 21% (configuration A) and 16% (configuration B) smaller than for the second version (Fig. 7b,e) and the redistribution of forces between the links of the same storey is also smaller. The situation for the third version (Fig. 7c,f) is in between the other two.

On the first version of link removal order within a storey (Fig. 7a,d), was analysed also the removal of links starting from the most loaded to the least loaded storey (from the lower storey toward the upper one). In this case, was observed a larger interaction between stories, but values of the shear force drop with 43% (configuration A) and 39% (configuration B) smaller than in the case of eliminating links from the upper one toward the lower one and smaller redistribution of forces between the links of the same storey.

7. CONCLUSIONS

A large experimental investigation including full-scale PsD tests in JRC-ELSA laboratory was conducted to validate technical solution and confirm the re-centring procedure.

For a structure with re-centring capability, the design objective consists in preventing yielding in members other than removable dissipative ones, up to a desired deformation. Ideally the latter should be the ultimate deformation capacity of the removable dissipative member. Following conventional code-based capacity design rules is not enough to accomplish this objective for the investigated structures. Sometimes, in order to keep elastic the MRFs into the dual configuration, which is essential for providing the re-centring capability, it might be necessary to use high strength steel for their members.

Numerical simulations were performed in order to investigate solutions for removing links. The optimal link removal order is the one which results in smaller drop of force during link removal. From this perspective, within a given storey links should be removed from the least loaded toward the most loaded. In the height-wise direction, links should be removed from the lower storey toward the upper ones.

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