DCL Design of Reinforced Concrete Moment Resisting Frames Can be unsafe

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SUMMARY:
A " DCL design" in Eurocode 8 is made with the minimum value q=1,5 of the behavior factor: the seismic action is established following Eurocode 8 but design checks are made to the code for non-seismic action. The structure is thus designed for strength, not for ductility. It is recommended that DCL design be applied only in low seismicity areas, but it is not an obligation. The evaluation of reference buildings presented in the paper shows that DCL design of RC moment resisting frames can be unsafe if the ground accelerations at the foundation $\gamma_I a_{gR} S$ is around or above 0,15g, because a design check of the shear resistance of nodes is not explicitly prescribed in non-seismic codes. The standard "strut and ties" method could apply, but no indications are given on how to apply it in this case.

Keywords: Reinforced Concrete. Node failure. Seismic Design.

1.INTRODUCTION

A " DCL design" in Eurocode 8 corresponds to a design made with the minimum value (q=1,5) of the behaviour factor q: the seismic action is established following Eurocode 8, but no other design criterions or checks of that code apply, all checks being made to the regular code for non-seismic action like. For reinforced concrete in Europe, this means Eurocode 2. The structure is in then designed for strength, not for ductility. Eurocode 8 recommends that DCL design be applied only in low seismicity areas, which means a ground acceleration at the foundation $\gamma_I a_{gR} S$ not greater than 0,5g but it is not an obligation. National annexes to Eurocode 8 may take a more constraining position, but it is not the case in most countries. So in those countries the designers can logically wonder: why not do DCL design in moderate or high seismicity zones? That approach looks more attractive for "good" reasons: less design work and a feeling that DCL design, which means bigger concrete sections and elastic behaviour only, would finally be more resistant than "dissipative" DCM or DCH design and suffer no damage during an earthquake. However, the evaluation of reference buildings presented hereafter shows that DCL design of reinforced concrete moment resisting frames can be unsafe, because one critical design check is by-passed: the shear resistance of beam to column nodes.

2. REFERENCE STRUCTURES AND DESIGN CHECKS.

In the course of a research work at University of Liege, reference moment resisting frames in reinforced concrete have been designed. The objective of the work was essentially to assess the potential of expanded metal panels in retrofitting structures which would be under-designed for the present earthquake zonation or code and to develop a design method for those retrofitting elements; this subject is not presented here; one complete reference is available (Phung Ngoc Dung, 2011).
The design parameters are:

- Different zone seismicity and soil conditions, which define ground accelerations at the foundation of the structure $\gamma_I a_{gR} S$ respectively 0,05g, 0,15g and 0,30g;
- Different Ductility class of the design, which could be low (DCL) or medium (DCM);

- Different numbers of storeys, as shown on Figure 1 and Table 1.

Spans are all 5m, first storey height is 3,5m and all other storey height is 3,0m.

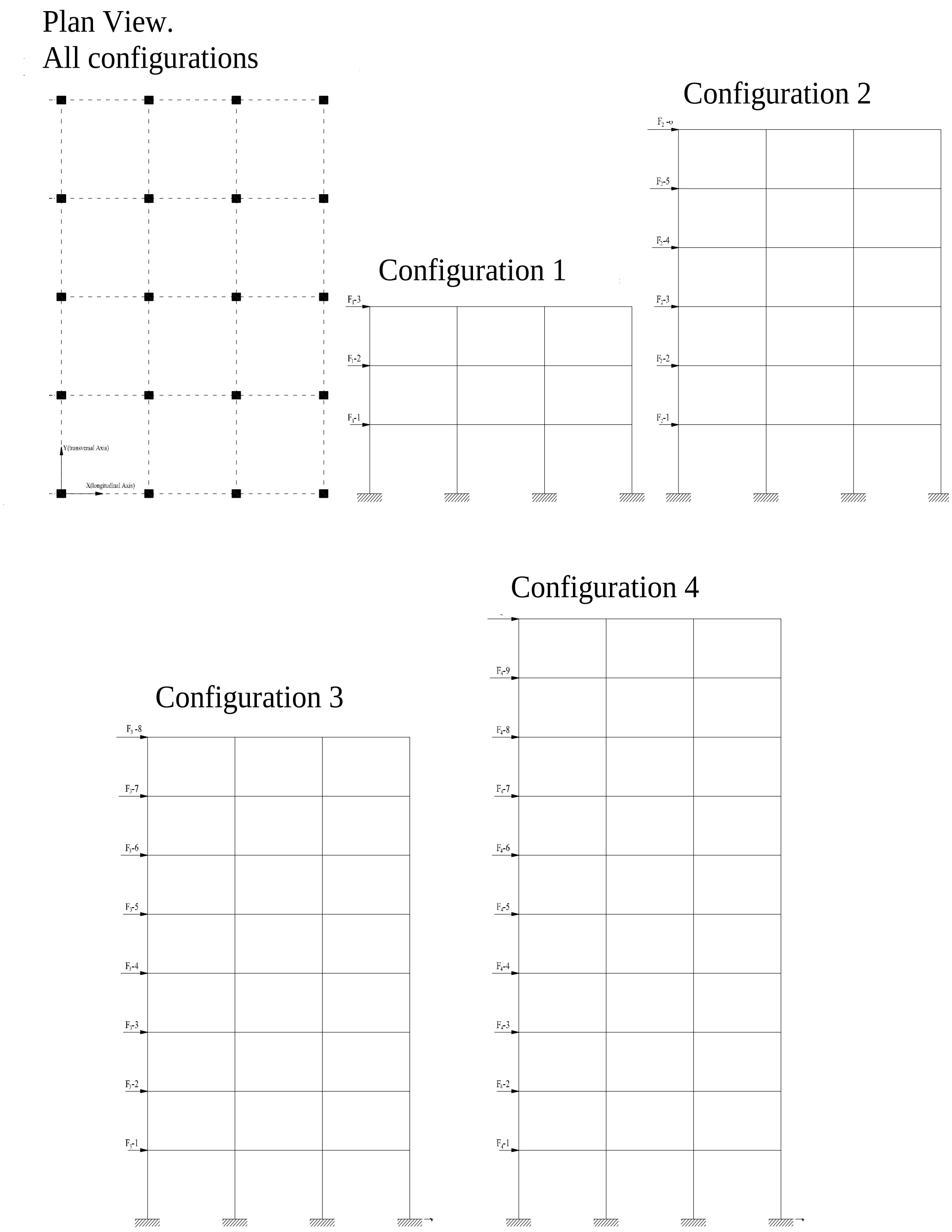

Figure 1. Configurations of the designed buildings.

All sections were designed to Eurocode 2 (DCL design) or to Eurocode 2 and Eurocode 8 (DCM design). Dimensions of beams and columns in the bottom zone of the buildings are given in Table 1. All beams have T sections with a 0,15 m thick slab. Required and placed reinforcement are presented in Table 2. All studied frames elements have been designed with large shear resistance so that under earthquake there is no failure in beams or column caused by shear. The critical regions, though not explicit in DCL design, are governed by flexural deformations.

Table 1. Section of main reinforced concrete elements in the different building configurations.

Name	Number of storeys	Bottom Internal Column Section (m)		Bottom Beam Section (m)
		0,05g	0,15g	
Configuration 1	3	0,30 x 0,30	0,35 x 0,35	0,35 x 0,25
Configuration 2	6	0,35 x 0,35	0,35 x 0,35	0,35 x 0,25
Configuration 3	8	0,60 x 0,60	0,60 x 0,60	0,40 x 0,25
Configuration 4	10	0,60 x 0,60	0,60 x 0,60	0,40 x 0,25

Table 2. Reinforcements of main reinforced concrete elements in the different building configurations.

Config. Design PGA	Story	Reinforcement in beams (mm^2)				In interior columns (mm^2)
		From analysis		Chosen		
		Top	Bot.	Top	Bottom	
1-0.05g	1	1002	460	12Φ8+2Φ16(1005)	3Φ14(462)	8Φ20(2513)
	2	826	445	12Φ8+2Φ16(1005)	3Φ14(462)	8Φ20(2513)
1-0.15g	1	1332	750	12Φ10+3Φ16(1546)	3Φ18(763)	8Φ25(3927)
	2	1332	750	12Φ10+3Φ16(1546)	3Φ18(763)	8Φ25(3927)
2-0.05g	1-5	1060	448	12Φ10+2Φ10(1100)	3Φ14(462)	8Φ18(2034)
2-0.15g	1-5	1474	887	12Φ10+3Φ20(1885)	3Φ20(942)	8Φ25(3927)
30.05g	1-4	881	322	12Φ10+2Φ10(1100)	3Φ14(462)	12Φ20(3770)
3-0.15g	1-4	1600	945	12Φ10+3Φ20(1884)	4Φ20(1257)	12Φ20(3770)
4-0.05g	1-5	1065	351	12Φ10+2Φ12(1169)	3Φ14(462)	12Φ20(3770)
4-0.15g	1-5	1065	953	12Φ10+3Φ20(1884)	4Φ20(1257)	12Φ20(3770)

There is no check in shear prescribed in Eurocode 2 for beam-column intersection zones or "nodes". The same is very likely to take place in EC8 DCL class design of moment resisting frames. The problem is there is a potential Ultimate Limit State in beam-column node which is in that way disregarded, the failure of nodes in shear. Nodes are submitted to high shear in seismic situations because the bending moments in beam ends generate a sum of local shear in the node which is not set forward by the global analysis of a structure.
Figure 2 presents the situation. The resultant shear V in the node created by the design moments M^{+}_{left} and M^{-}_{right} at beam ends is equal to: $V_{Ed} = \dfrac{\left|M^{+}_{left}\right| + \left|M^{-}_{right}\right|}{h_w}$
where h_w is the height of the beam.

Design rules for that shear situation exist in Eurocode 8 at clause 5.4.3.3 for DCM design: horizontal confinement reinforcement should be not less than that specified for critical regions of columns; if beams frame into all four sides of the joint and their width is at least three-quarters of the parallel cross-sectional dimension of the column, the spacing of the horizontal confinement reinforcement in the joint may be increased to twice that of critical regions of columns, but may not exceed 150 mm. Those are rules detailing rules without calculations, but design expressions are provided for DCH design. Two different expressions exists to estimate the confinement which is needed to provide shear resistance to the node; those two expressions provide different results, probably because the subject has not yet been thoroughly studied. In the study of DCL design submitted to earthquake, it was decided to approach the resistance to failure of unconfined nodes by means of the general "struts and ties" approach, because the latter exists in Eurocode 2 and could be refered to in DCL design.
The resistance of the joint is dependent on the efficient compression part, which is a concrete compression strut. Figure. The effective width of the compression strut $b_{ef,n}$ is estimated as:

$$b_{eff} = 0.2 l_{dia} = 0.2\sqrt{h_w^2 + h_c^2}$$

l_{dia} is the diagonal length of the node and h_c is the height of the column.
The compression strut resistance R_{strut} is computed as:

$$R_{strut} = \nu f_{cd} b_{ef,n} b_c = 0.5 f_{cd} 0.2 b_c \sqrt{h_w^2 + h_c^2}$$

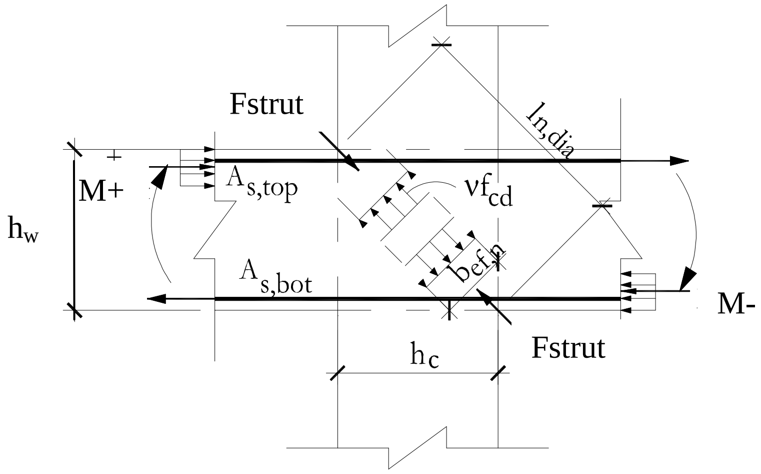

Figure 2. Compression strut force induced by seismic bending moments at interior nodes.

3. EVALUATION OF THE SEISMIC BEHAVIOUR OF THE DESIGNED FRAMES.

Pushover analysis defined in Eurocode 8 Part 1 and 3 have been adopted to make the evaluation of the set of designed structures. It follows the steps defined in the N2 method proposed by Faijar (2000). Complete explanations of the method are given in (Phung Ngoc Dung, 2011). The method defines for each structure and each level of seismic action a performance point which correspond to the minimum displacement which the structure should be able to reach in order to avoid failure. The model takes into account the specific characteristics of each structure, such as steel content and ductility in bending; the action effect at nodes is also computed and compared to the failure criterion mentioned above. The pushover curves and performance points corresponding to DCL design and to peak ground acceleration at the foundation equal to 0,05g and 0,15g are presented at Figure3.

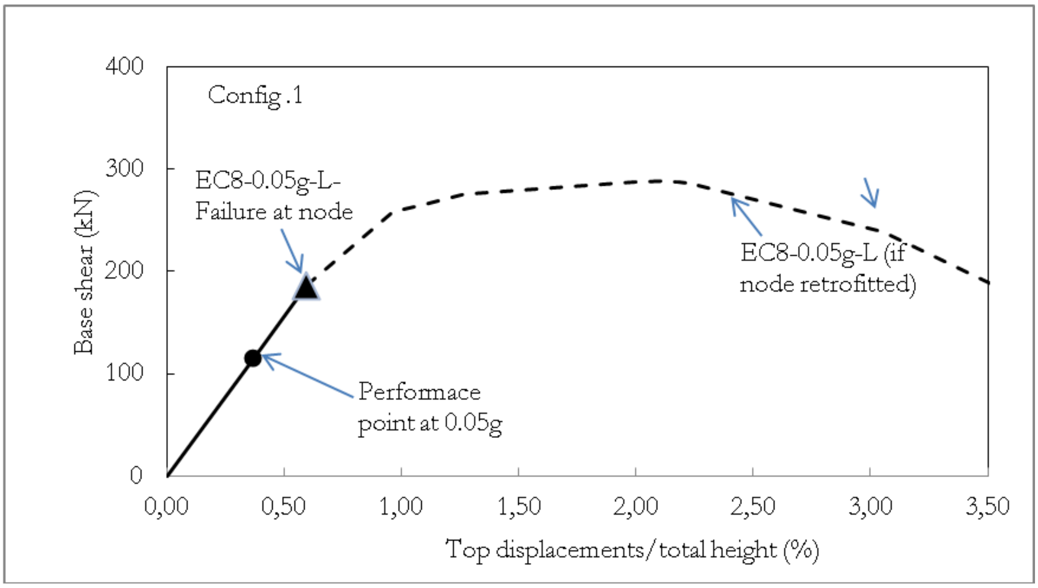

Figure 3. Pushover curve and performance point for configuration 1 DCL design for 0,05g.

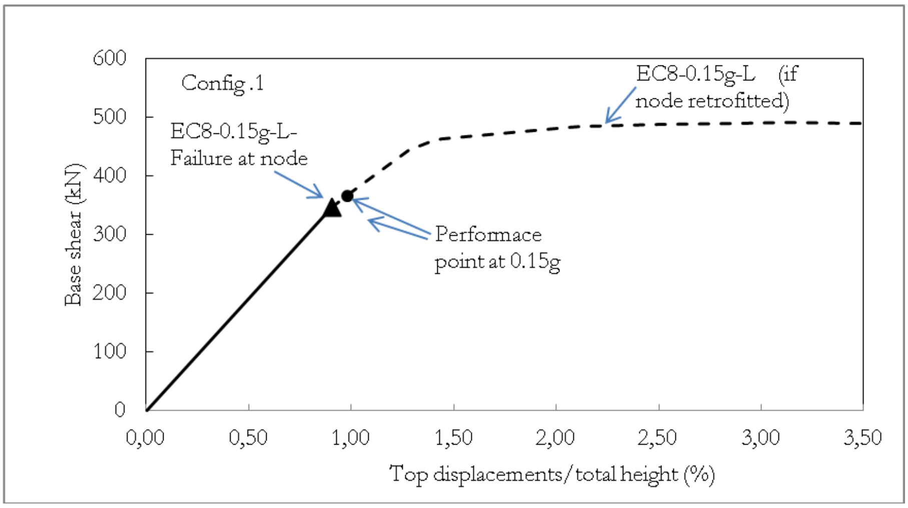

Figure 3 (continued). Pushover curve and performance point for configuration 1 DCL design for 0,15g.

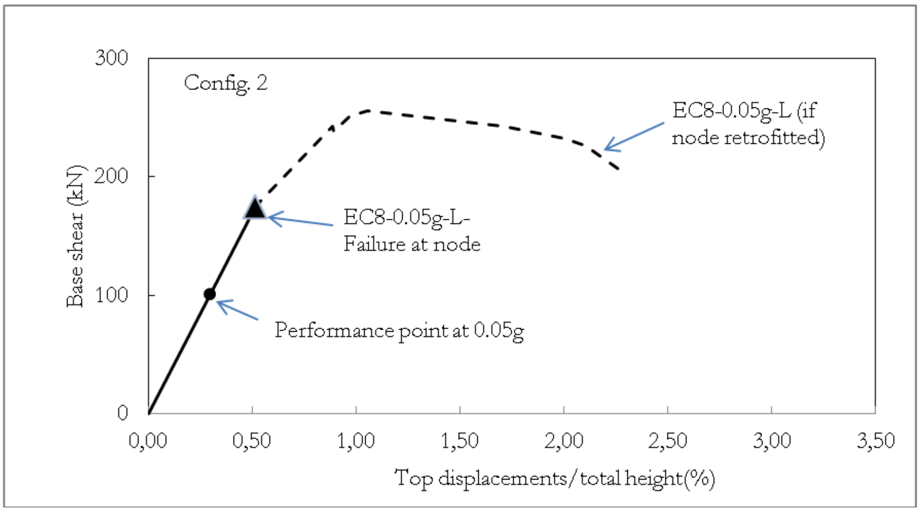

Figure 3 (continued). Pushover curve and performance point for configuration 2 DCL design for 0,05g.

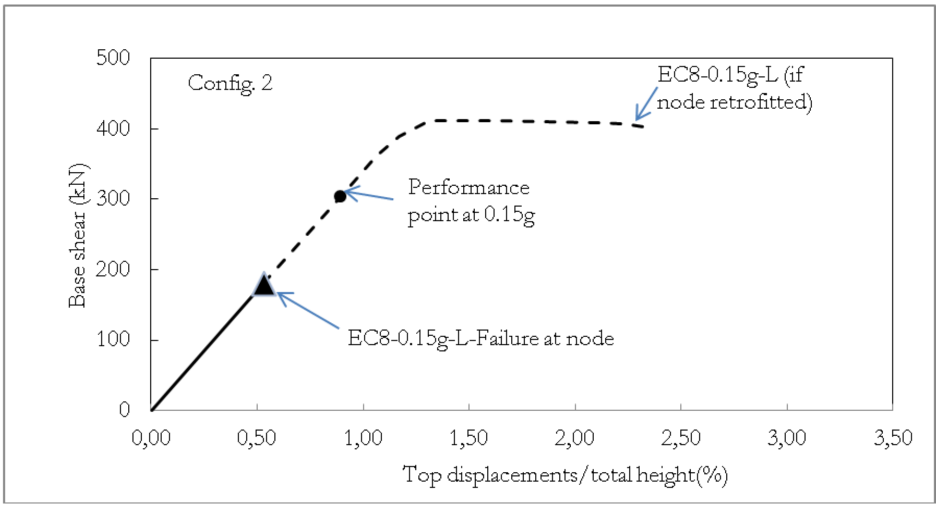

Figure 3 (continued). Pushover curve and performance point for configuration 2 DCL design for 0,15g.

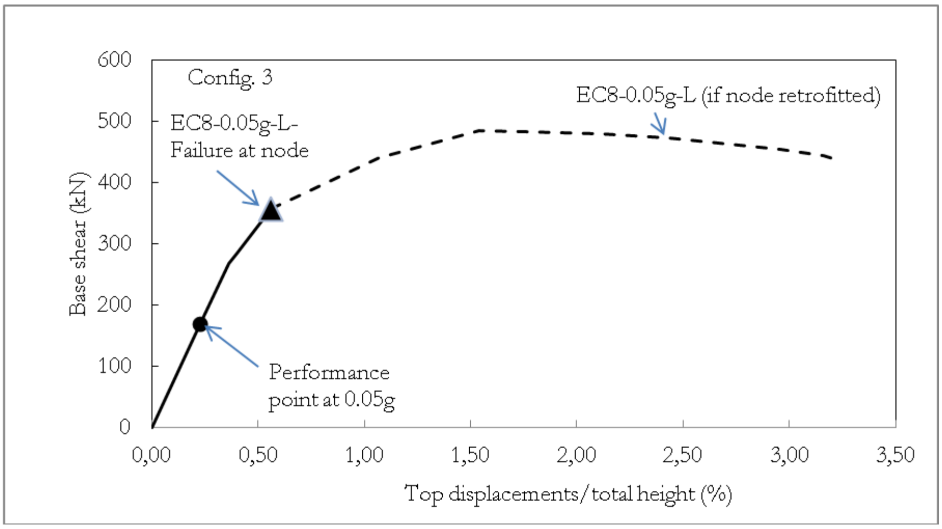

Figure 3 (continued). Pushover curve and performance point for configuration 3 DCL design for 0,05g.

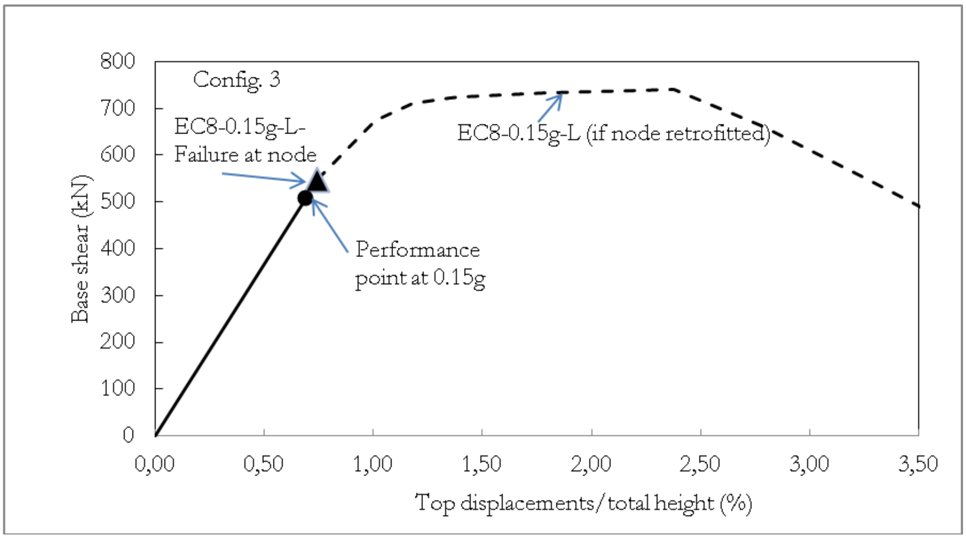

Figure 3 (continued). Pushover curve and performance point for configuration 3 DCL design for 0,15g.

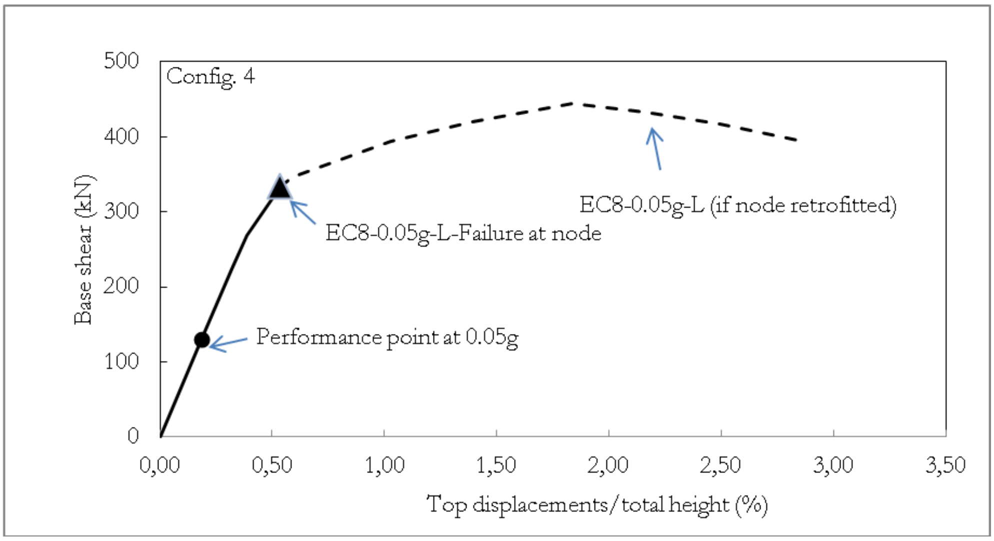

Figure 3 (continued). Pushover curve and performance point for configuration 4 DCL design for 0,05g.

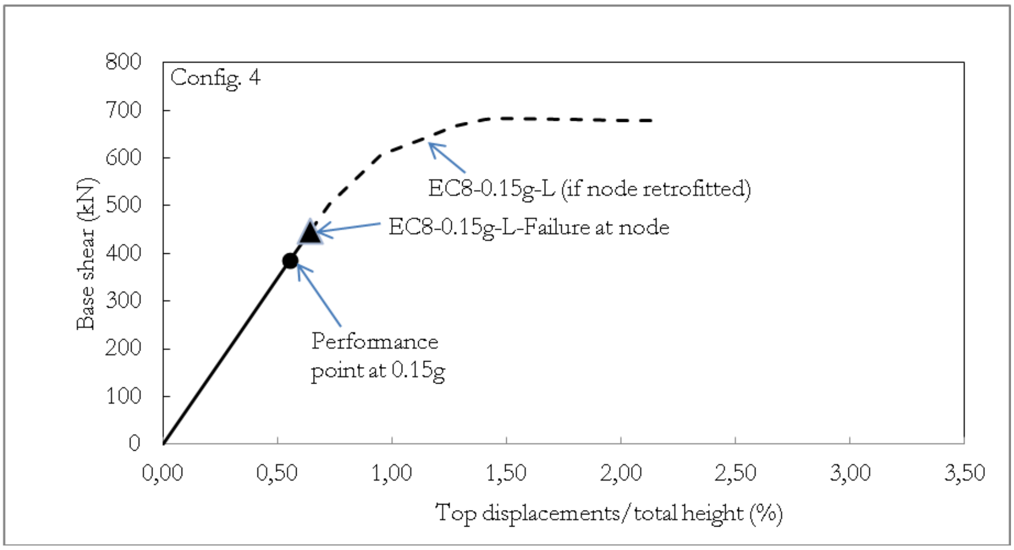

Figure 3 (continued). Pushover curve and performance point for configuration 4 DCL design for 0,15g.

4. OBSERVATIONS AND CONCLUSIONS

Looking at all the graphs presented at Figure 3, it appears that DCL design is satisfactory for design peak ground accelerations equal to 0,05g, as all failures take place for displacements which are more than 1,5 times the displacement at performance point.

But DCL design is questionable for design peak ground accelerations equal to 0,15g.
Indeed:

- Failure at nodes take place for displacements which are smaller than the displacement at performance point for structures Configurations 1 and 2; and failure at nodes will generally induce a global failure of a building.
- Failure at nodes take place for displacements which are just above the displacement at performance point for structures Configurations 3 and 4;

Given the uncertainties in the evaluation of the shear resistance of nodes, the results should not be taken as mathematical certainties, but it can be concluded that DCL design is unsafe for peak ground acceleration at the foundation around and above 0,15g.
This conclusion should be taken into account. Practically, this could be done in two ways.

One way would consist in changing the recommendation on the limitation of DCL design into a prescriptive rule, like for instance an absolute limitation to a maximum design PGA equal to, for instance, 0,1g.

Another way would consist in creating a DCL+ class, with a certain number of requirements.
The requirement on design of nodes could be that the nodes of DCL design of MRF's should be checked using a strut and ties approach like the one proposed in paragraph 2. But it must be mentioned that such an expression still requires calibration work based on experiments.
An alternative would be the application of the DCM detailing rules on confinements of nodes. Other requirements, related to other brittle types of failure mechanisms, would also be necessary in that DCL+ class. They would bear on length of overlap, maximum longitudinal steel content, etc.

The graphs also indicate that retrofitting beam column nodes for shear resistance can modify a lot the behaviour; but it is known that such retrofitting is not easy work.

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