Building structural behavior under low-medium seismic action

Thesis Report

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Under Natural Hazards and Catastrophic Events

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Executive Summary

The present thesis is dedicated to understand seismic performance of structural building designed according to the current version of Eurocode 8 part 1. Thence, new design guidelines can be formulated for low-medium seismic region which may be more effective than current methodologies stipulated in EC8, either DCL or DCM/DCH.

The finding has that the current version of code imposes heavy demand on building subject to low-medium seismic action. The demand is in form of brace overstrength homogeneity, slenderness limit rules and conservative period prediction that aim to ensure high ductility performance in heavy seismic region. This leads to inefficient design, tonnage wastage and impractical solution. The parametric study is then carried out to identify how the structure response changes with relaxed rules. It is observed that higher overstrength limit, relaxed homogeneity rule is more practical in many situation and introduce acceptable reduction in ductility and safety factor; while relaxing slenderness limit can reduce total tonnage but increase second mode contribution to top displacement, which is not taken into account in the design. In addition, the simulation also proves the insignificant contribution of column flexural stiffness to overall building lateral stiffness, and that there is another way to design columns following the corresponding brace overstrength of individual level but not verified in this study due to analysis type limit.

Alternative methods are proposed, through the parametric study, which combine the relaxation of such strict rules with reduced ductility factor that deemed adequate for low-medium seismic requirement. This study finds that cutting q factor by half but allowing higher brace overtrength limit or applying the homogeneity rule only to a portion of top floors are viable solution to obtain equally safe structure, which means the same safety factor. The cost of such new alternatives is extra tonnage due to using lower load reduction factor q.

1 Introduction

1.1 Report Objective

This report aims to discuss the set of case studies on concentrically braced steel building (CBF) structural behaviour in low-medium seismic zone. It focuses on the relation between design requirement and seismic performance in term of reliability, ductility and economic design; from which the conclusion about how effective the EC8 part 1 design guideline is will be drawn.

To be specific, there are 2 set of case studies, general set and parametric set, presented to serve the above-mentioned purposes. The first set present the structural performance (ductility, overstrength, tonnage) of frames strictly designed according to EC8 part 1, with variation across the spectrum of seismic input and structure height (number of stories). According to those observations from the general set, the parametric set investigates the effect of several typical parameters to improve the design effectiveness.

The study is part of the larger European commission research that targets to develop specific design methodologies for steel and steel-concrete structure in low-medium seismic region and help address both safety and economical concerns. The work in this report contributes to work package 3 (Behaviour of concentrically braced frames under moderate seismic action) task 3.2 (Development of a set of case-studies strictly designed to Eurocode 8).

1.2 Background

Regarding the strict application of current EN1998 for the low-medium seismic region, the following observations are mentioned in form B2- Research Proposal Description.

- The use of DCL principle is only recommended for low seismicity regions. According to EC8, low seismicity regions are those in which the parameter ag·S is smaller than or equal to 0.1g, where ag is the Peak Ground acceleration (PGA) and S is a coefficient depending on the soil conditions. As a matter of example, for normal soil conditions, important parts of Belgium (Eastern part), of France (North) and even of Germany cannot be considered as "low seismicity" since ag·S is likely to range from 0.1 to 0.2g and these regions must be considered as moderately seismic. The restriction on the use of DCL principle is generally justified by the fact that the uncertainty on the seismic action should be compensated by a capacity of the structure to deform beyond its yield limit without collapsing, which is only guarantee from DCM principle and above.
- Regarding the manufacturing and according to EN1990-2 Tab. B2 classification, structures designed with DCL are category SC1 and structures designed with DCM are category SC2. This usually leads to a higher executing class and therefore to higher cost for manufacturing, quality control, etc.
- The design values for ag were subject of discussions in national committees with an industrial pressure (e.g. from the masonry industry) pushing for assigning as many regions as possible to low seismicity. Further development of seismic zonations based on harmonized methodologies is however very likely to increase PGA values beyond 0.1g.
- The use of DCM principle requires the application of all design rules prescribed to provide ductility to the structure, whatever the value of the behaviour factor q considered in the design. For example, structures designed with $q = 2$ or $q = 4$ must fulfil the same requirements in terms of local ductility and structural homogeneity while the overall ductility demand is clearly different. It must also be noted that these requirements in terms of local ductility and homogeneity are often more decisive for the design than the resistance to earthquake itself.

- In general, for regions of moderate seismicity, the use of a q factor equal to 2 to 3 is sufficient to reduce the seismic action to a reasonable level (i.e. comparable to design wind action, which is the other main horizontal action challenging the structure).

As a conclusion, 2 attitudes are possible for moderate seismicity regions:

- Design according to DCL, contrary to what is recommended by Eurocode (although allowed), which means design for unreduced seismic actions, without any guarantee and control of ductility and thus with a very limited reliability level.
- Design according to DCM, with the possibility of using high q factor to reduce the action, even if these high q-values are often not necessary, and with the guarantee of a global ductile behaviour. In counterpart, significant local ductility and structural homogeneity requirements have to be fulfilled. Furthermore the over-assessed behaviour factor needs to be corrected by the global overstrength factor Ω which leads to even more complex design.

It is thus desired to carve the intermediate way of design with reduced by controlled amount of ductility, which ensure the safety against uncertain seismic load, but with less stringent requirements of local ductility and homogeneity that aim for lower behavior factors. The requirements should also be tuned according to actual seismicity level of the area, which make the design both reliable and economic.

To achieve this, further research on the following topics are expected.

- Bolted connection in CBF
- Seismic behavior of CBF, and in particular relation between design requirements and seismic performance
- Seismic behavior of steel and composite MRF with class 3 and 4 cross-sections

This report will elaborate the work on the second point by the set of design cases strictly following Eurocode.

1.3 Reference Documents

The following documents are referred to in this report:

EN 1990-2002 Eurocode 0 Basis of structural design

EN 1991-1-1:2002 Eurocode 1: Actions on structures — Part 1-1: General actions — Densities, self-weight, imposed loads for buildings

EN 1993-1-1:2005 Eurocode 3: Design of steel structures —Part 1-1: General rules and rules for buildings

EN 1998-1:2005 Eurocode 8: Design of structures for earthquake resistance — Part 1: General rules, seismic actions and rules for buildings

FEMA 356 Pre-standard and Commentary for Seismic Rehabilitation of Building

B1 Research Proposal Administrative Overview

B2 Research Proposal Description

B4 Technical Annex Update

Elghazouli AY, 2010, Assessment of European seismic design procedures for steel framed structures, Bulletin of Earthquake Engineering, Vol:8, ISSN:1570-761X, Pages:65-89

Malaga-Chuquitaype C, Elghazouli AY, 2011, Consideration of seismic demand in the design of braced frames, Steel Construction, Vol:4, 1867-0520, Pages:65-72

Uriz, P., Mahin, S. A.: Towards earthquake-resistant design of concentrically braced steel frame structures, PEER Report 2008/08, Pacific Earthquake Engineering Research Centre, University of California at Berkeley, USA

2 Literature Review

This sector dedicates to discuss state-of-art knowledge on the relationship between Eurocode 8 design guideline and CBF performance under seismic action.

2.1 Frame overstrength and slenderness

The EC 8 part 1 rule follows the tension-based design of braces, in which the compression brace contribution to seismic resistance in DCM is ignored. This will lead to unavoidable overstrength due to contribution of compression member buckling load. That overstrength, the ratio between the design base shear and the yield base shear, can be calculated as below.

$$\frac{V_y}{V_d} = \frac{N_{pl} + N_b}{N_{pl}}$$

It is established that, for tension-based design of braces, the overstrength decreases as λ increases and becomes relatively insignificant for comparatively large slenderness values (Elghazouli, 2011). To elaborate, the higher the non-slenderness ratio, the lower the brace buckling load, and consequently the overstrength factor. It also means when the brace is stocky (no buckling occurs) the overstrength factor approach the value of 2.

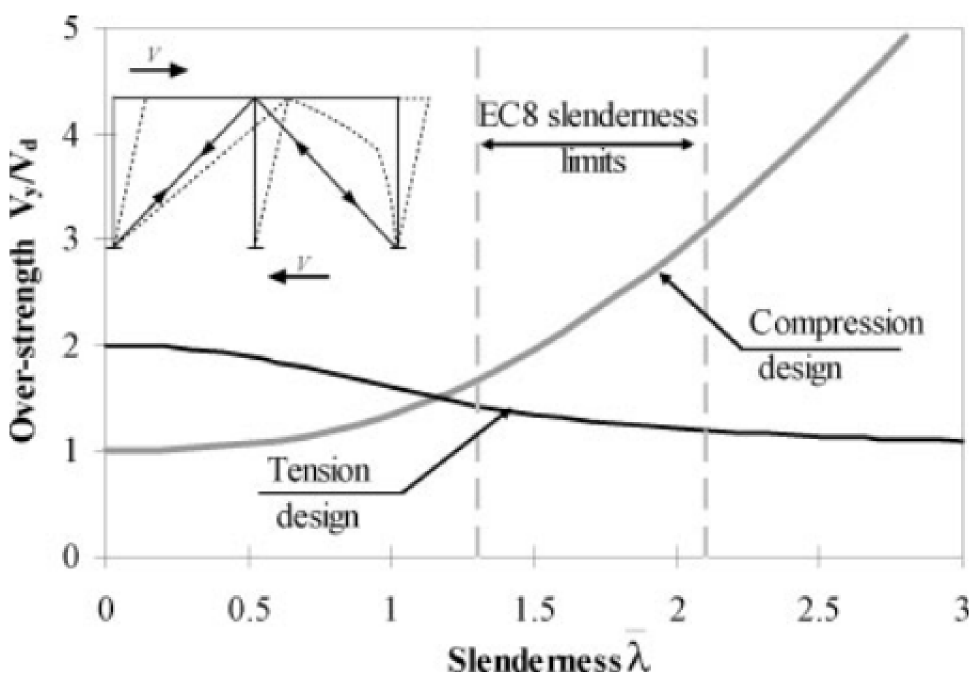


Figure 1:Variation of frame overstrength with brace slenderness (Elghazouli, 2011)

This facts also implies further complication when the frame has more than 1 stories. Each floors has different sections of brace members with different slenderness; and therefore, according the the relation between slenderness and overstrength established above, each storey may have different overstrength factor. This can lead to non-homogenuous effect and soft-storey mechanism, which will be explained further in detail in section 2.3

In addition, the graph also suggests that more slender brace brings the structure overstrength, hence their performance, closer to the design prediction. This fact provides a possibility to further relax upper limit slenderness limit currently set at 2, provided that the associated problems, such as out-of-plan excessive vibration that induces undue load on the frame on the perpendicular direction, can be resolved by other means.

2.2 Ductility Demand

In the research paper “Consideration of seismic demand in the design of braced frames” of Ch. Málaga-Chuquitaype, A. Y. Elghazouli, an interesting study about relation between ductility demand and frame strength is done. The model of one storey, two-bay CBF consists of pin-jointed rigid members and is also pinned at base. The braces are modelled using fibre-based buckling elements following the approach suggested by Urizand Mahin (Mahin, 2008).

One hundred records in total derived from 27 earthquakes in PEER-NGA database (http://peer.berkeley.edu/nga) are used to do response-history analysis on the frame.

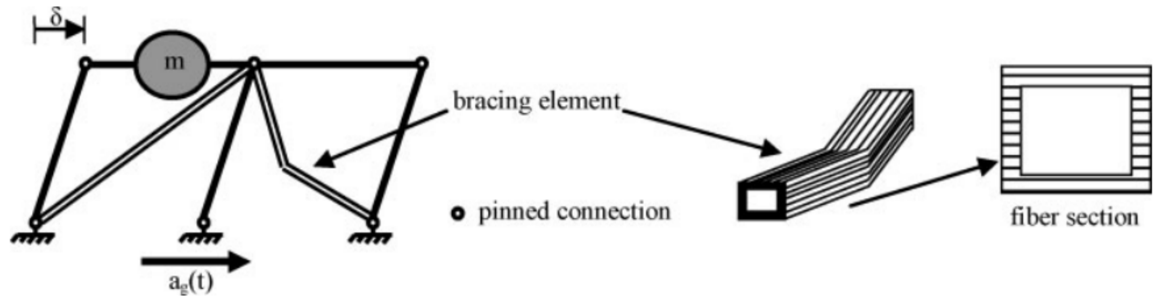


Figure 2: Model of an SDOF CB subjected to ground motion

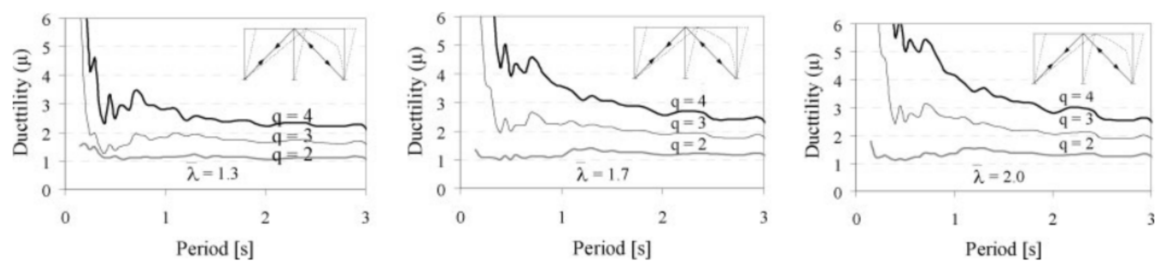


Figure 3: Ductility demands on braced frames in stiff to moderately stiff soils

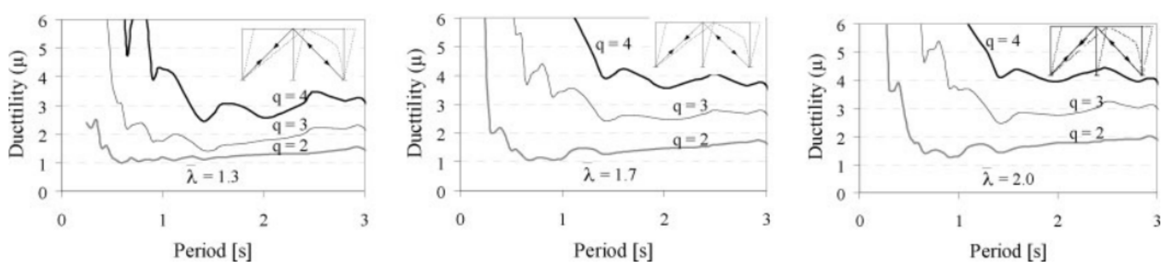


Figure 4: Ductility demands on braced frames in soft soils

The research paper draws conclusion on the effect of soil type on variability of the frame ductility demand, which is evident that the soft soil (type D) creates more fluctuation in the grapth than the hard soils. It also points out that the EC8 prediction of top storey displacement (model elastic displacement times behavior factors) is reliable for hard soil if T>0.5s and λ <1.7 or with T>1 and λ around 2. The prediction is less accurate if the soil is soft.

For the purpose of this research , there are 2 observations made from the case study:

- For high multi-storey building, which means the period is higher and the first storey brace is stockier (slenderness about 1.5 and below), the ductility demand is not affected by structure period. The curve becomes horizontal after a certain period threshold.

- Ductility demand and the structure overstrength, represented by q factor, are complementary to each other and inversely related. It means that the higher structure overstrength will require lower ductility demand and vice versa. Later on in this research, there is an attempt to clump the 2 factors of ductility capacity and overstrength into a single value known of safety factor.

2.3 Brace overstrength and column stiffness

As it is mentioned earlier, it's not only the brace overstrength but also their balance amongst floors in multi-storey building that play vital role in CBF ductility performance. If a certain floor brace is excessively weak as compared to others and yields first, all the ductility demand is unduly concentrated on that floor and creates soft-storey mechanism. From the capacity point of view, this effect prevents others braces to reach their ductile zones and reduce the total ductility that the frame can achieve if all braces yield in a more uniform pattern. EC8 part 1 stipulates the limit of 25% of brace overstrength to prevent this "soft storey" phenomenon.

The departure from such rule, however, can be allowed if the column stiffness is increased and continuity is ensured; which is rationalized by the following relation between column-to-brace stiffness ratio and drift concentration factor.

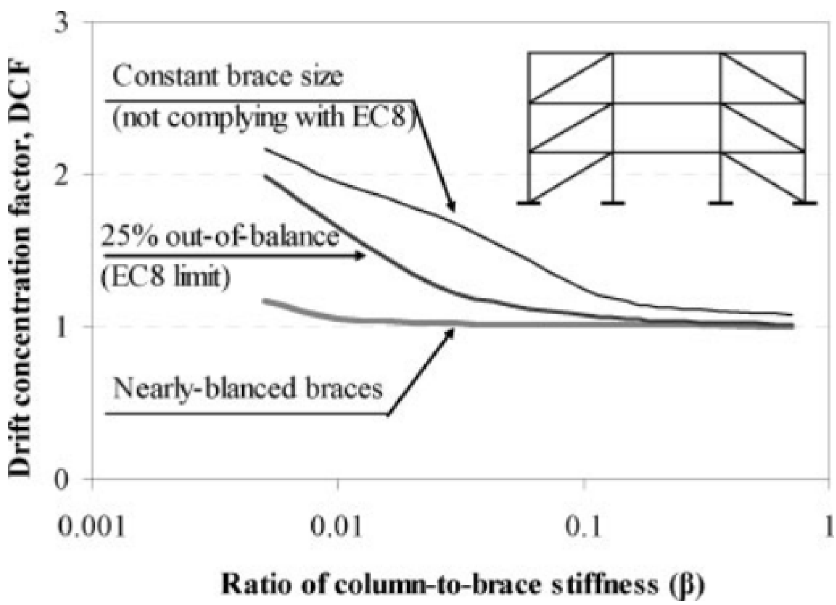


Figure 5: How column stiffness influences inelastic demand (Elghazouli, 2011)

The horizontal axis parameter is relative bending stiffness of column to the lateral tension stiffness at the lowest storey and is calculated by:

$$\beta = \frac{\sum \frac{I_c}{L_c^3}}{\sum \frac{A_d}{L_d} \cos\varphi} \qquad (4)$$

where

A_d and L_d	cross-sectional area and length of diagonal braces respectively
I_c and L_c	second moment of area and height of columns respectively
φ	angle between brace and horizontal projection

The DCF is defined as the ratio between the maximum inter-storey drift at any storey and the roof drift of the structure. The value of 1 indicates a uniform displacement pattern, while the higher values are more prone to "soft storey" mechanism.

The graph shows that higher column stiffness can help neutralize the non-uniform inter-storey drift, and the DCF approach 1. the It means that column design can be increased to compensate for brace overstrength irregularity along the height. In EC8 part 1, the column design, as well as other non-dissipative members, abides to the below formula, which will be re-examined in this research to allow a relaxation of 25% limit of brace overstrength.

$$N_{\mathrm{pl,Rd}}(M_{\mathrm{Ed}}) \geq N_{\mathrm{Ed,G}} + 1{,}1\gamma_{\mathrm{ov}}\ \Omega . N_{\mathrm{Ed,E}}$$

3 Research Methodology

3.1 Manual structural design

This section devotes to providing clear description about how all the cases are designed. Basic input assumptions, such as material, soil and earthquake types, structural layout, step by step design method for frame, are to be specified.

3.1.1 Material

All the structure elements are made of steel class S355 with bi-linear law for non-linear analyses.

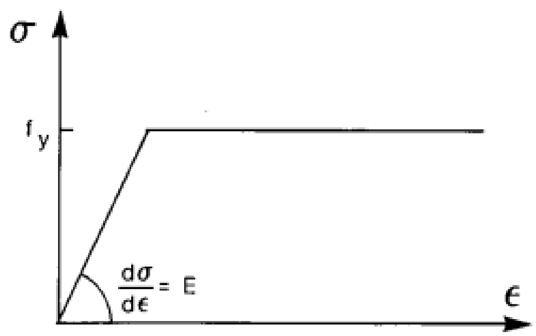


Figure 6: Bi-linear stress-strain relationship

Designation	Tensile Strength [N/mm2]	Min. Yield Strength [N/mm2] for Thickness [mm]	
		Up to and including 40	Over 40 up to and including 80
S355	490/470	355	335

Table 1: Material Property

3.1.2 Soil and earthquake type

In this study, soil type B and earthquake type 1 are chosen universally for all the cases.

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 2: Soil Type

3.1.3 Structural Layout and Analysis Type

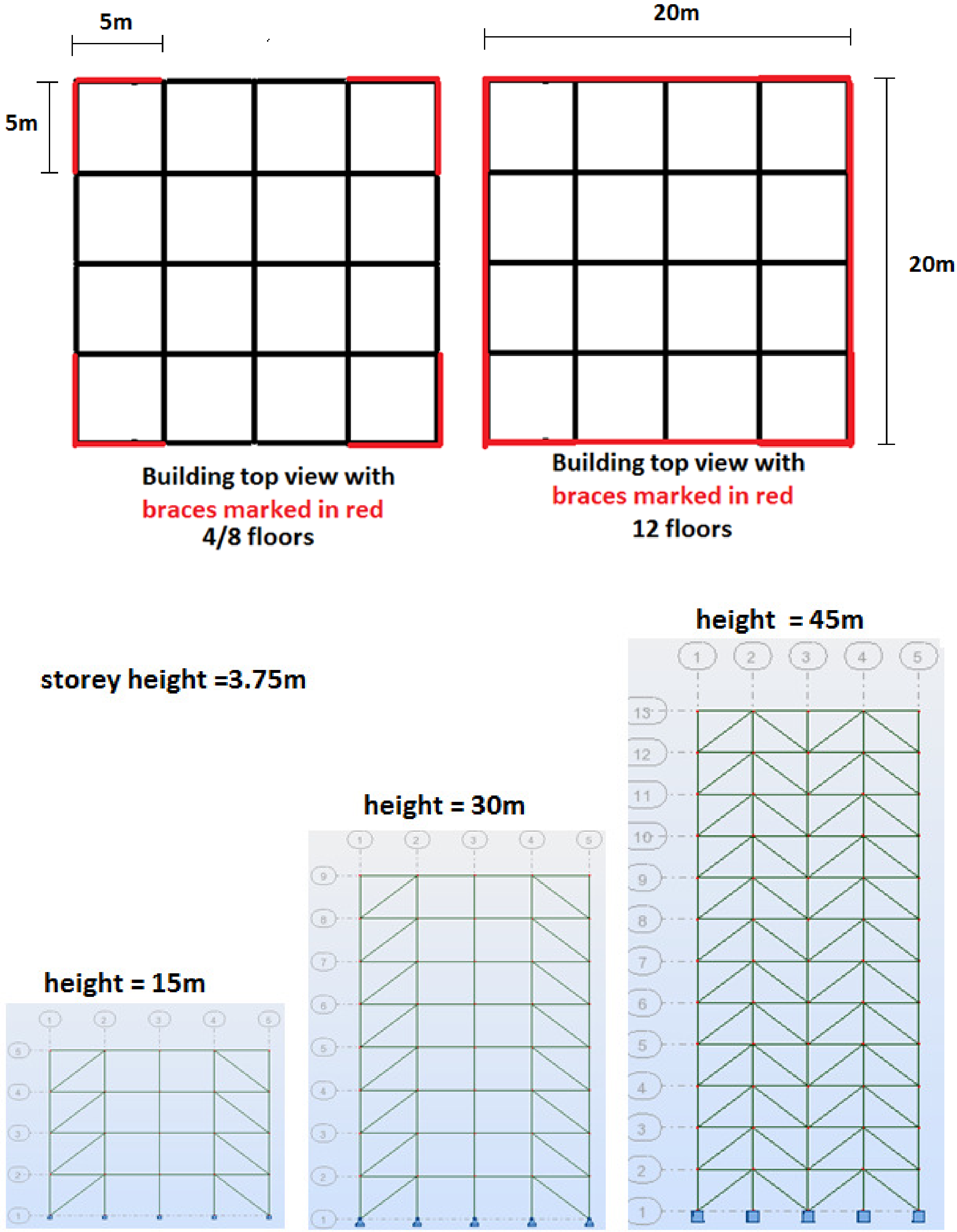


Figure 7: Structural layout for 4/8/12 story structure

All the structural connections are assumed pin, except for the column to column connections. When the building is subject to seismic action, all the circumferential frames with brace are effective in withstanding the load. In the worst case, when the load strikes in the direction parallel to either pair of frames; only 2 parallel frames are effective, each take half the load.

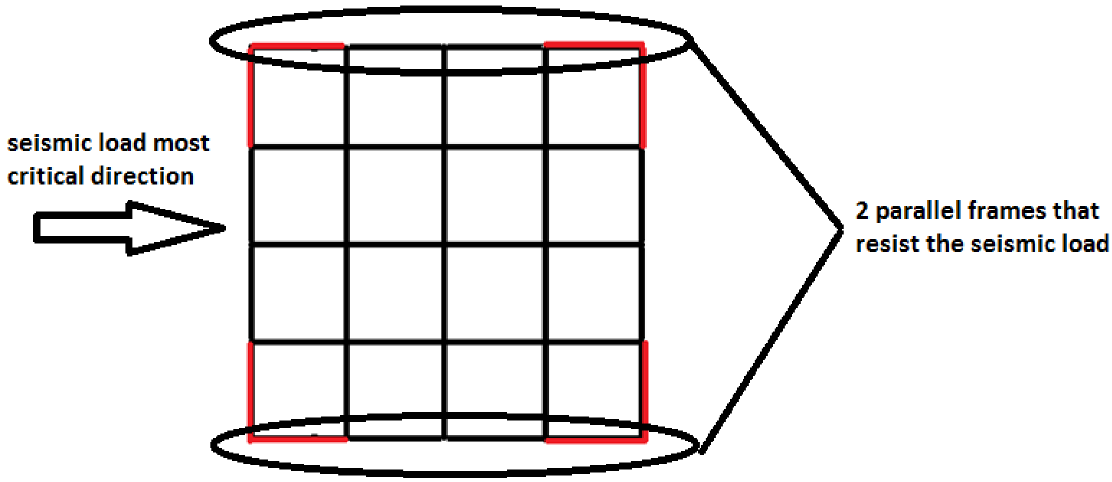


Figure 8: Assumed direction of seismic action on frames

3.1.4 Structural mass

The following permanent and transient loads are used in our structure

$G_k = 7.5kN/m^2$

$Q_k = 4kN/m^2$

storey	Floor area	G_k	Q_k	$\sum G_{k,i}"+"\sum \psi_{E,i} \cdot Q_{k,i}$ Mass Per frame (Ton)	$\sum G_{k,i}"+"\sum \psi_{2,i} \cdot Q_{k,i}$ UDL on beam (kN/m)
Roof	20x20=400	3000	1600	172.5	21.75
Storey	20x20=400	3000	1600	172.5	21.75

Table 3: storey mass (ψ2,i=0.3, ϕ=0.8)

3.1.5 Seismic load determination

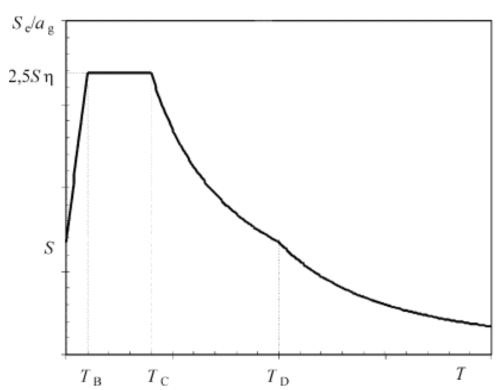


Figure 9: Elastic response Spectrum

The building period is estimated by the following formula

$$T = C_t * H^{0.75}$$

Based on the formula provided by EN1998-1.1, we have

$0 < T < T_B$ $$S_d(T) = a_g \cdot S \cdot \left(1 + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - 1\right)\right)$$

$T_B < T < T_C$ $$S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$

$T_C < T < T_D$ $$S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_C}{T}\right) \\ \geq \beta \cdot a_g \end{cases}$$

$T > T_D$ $$S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_C \cdot T_D}{T^2}\right) \\ \geq \beta \cdot a_g \end{cases}$$

where

a_g, S, T_C and T_D are as defined in **3.2.2.2**;

$S_d(T)$ is the design spectrum;

q is the behaviour factor;

β is the lower bound factor for the horizontal design spectrum.

NOTE The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for β is 0,2.

3.1.6 Load distribution

The triangular shape load distribution is assumed on the structure with the based shear forced calculated by the formula:

$$F_b = S_d(T_1) \cdot m \cdot \lambda$$

and the load on each floor is

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\Sigma z_j \cdot m_j} \tag{4.11}$$

where

z_i, z_j are the heights of the masses m_i m_j above the level of application of the seismic action (foundation or top of a rigid basement).

3.1.7 Member design

3.1.7.1 Design concept

The EC8 part 1 lay out 2 possible methodologies by which the building is designed, DCL (low dissipative structure behavior) and DCM/DCH(medium-high dissipative structure behavior). Each of them is accompanied by a set of rules to achieve the safety and ductility requirement. In this study, the CBF structure is designed based on DCL and DCM.

Design concept	Structural ductility class	Range of the reference values of the behaviour factor q
Concept a) Low dissipative structural behaviour	DCL (Low)	≤ 1,5 - 2
Concept b) Dissipative structural behaviour	DCM (Medium)	≤ 4 also limited by the values of Table 6.2
	DCH (High)	only limited by the values of Table 6.2

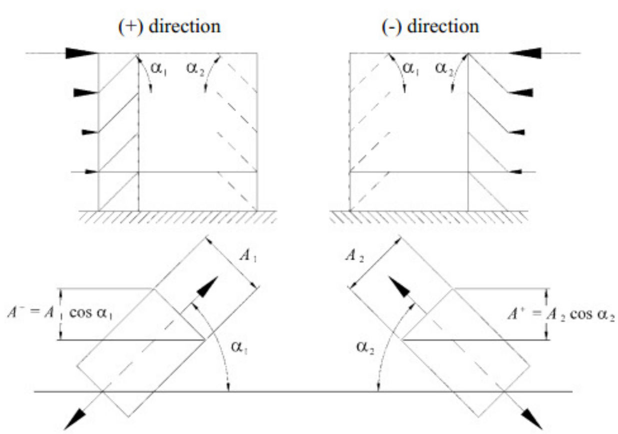
Table 4: Structural ductility class and upper limit reference for behaviour factors

DCL: The design is carried out with elastic global analysis without considering significant non-linear material behavior. A minimal dissipative factor of 1.5 is assumed for this design concept. All members are designed according to EC3 1-1 with no additional requirements. However, in this study, to make it simple and consistent with DCM design, the compression braces will be ignored in the design.

DCM: High level of non-linear material behavior is assumed with q=4, and the structure is stipulated to abide with following rules:

- The frame should exhibit similar deflection characteristics at each floor under load reversal, which is deemed to satisfied if the same cross section is used on each floor.

$$\frac{|A^{+} - A^{-}|}{A^{+} + A^{-}} \leq 0,05$$



- To prevent excessive vibration of the brace, which will imposed lateral load on other direction, the brace non-dimensional slenderness should be kept below 2.

- To make sure that tension member should yield before columns and beams, the following formula should be applied:

$$N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1{,}1\gamma_{ov}\ \Omega . N_{Ed,E}$$

$N_{pl,Rd}(M_{Ed})$ is the design buckling resistance of the beam or the column in accordance with EN 1993, taking into account the interaction of the buckling resistance with the bending moment M_{Ed}, defined as its design value in the seismic design situation;

$N_{Ed,G}$ is the axial force in the beam or in the column due to the non-seismic actions included in the combination of actions for the seismic design situation;

$N_{Ed,E}$ is the axial force in the beam or in the column due to the design seismic action;

γ_{ov} is the overstrength factor (see **6.1.3(2)** and **6.2(3)**)

Ω is the minimum value of $\Omega_i = N_{pl,Rd,i}/N_{Ed,i}$ over all the diagonals of the braced frame system; where

$N_{pl,Rd,i}$ is the design resistance of diagonal i;

$N_{Ed,i}$ is the design value of the axial force in the same diagonal i in the seismic design situation.

- Homogeneous dissipative behaviour of diagonals is assured through checking that maximum and minimum over-strength factors W doesn't differ more than 25%.
- The brace members in compression are ignored, leaving only the tension member active in resisting lateral load.
- The gravity load is resisted only by the beams and the columns, without the contribution from the braces.
- The second order effect can be ignored (which is common for CBF) provided the following condition is made

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10 \qquad (4.28)$$

where

θ is the interstorey drift sensitivity coefficient;

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation;

d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration and calculated in accordance with **4.3.4**;

V_{tot} is the total seismic storey shear; and

h is the interstorey height.

3.1.7.1 Design Limit State

There are also 2 limit states that we need to take into consideration, ultimate limit state (ULS) and damage limit state (DLS). ULS is deemed to be satisfied if the all the member designs conform to the all above-mentioned rule, depending on design concept. DLS, on the other hand, require further verification of inter-storey drift.

a) for buildings having non-structural elements of brittle materials attached to the structure:

$$d_r \nu \leq 0{,}005\, h\,; \tag{4.31}$$

b) for buildings having ductile non-structural elements:

$$d_r \nu \leq 0{,}0075\, h\,; \tag{4.32}$$

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r \nu \leq 0{,}010\, h \tag{4.33}$$

where

d_r is the design interstorey drift as defined in **4.4.2.2(2)**;

h is the storey height;

ν is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.

For this study, the intermediate requirement of case b (buildings having ductile non-structural elements) is chosen for all the designs.

3.1.8 Simplified structural model and step-by step design of structure

3.1.8.1 Simplified structural model

To simplify the structural model, taking into account the fact that braces are inactive in resisting gravity load and compressed brace members are ignored in the seismic load, the structural layout will be transformed into the following.

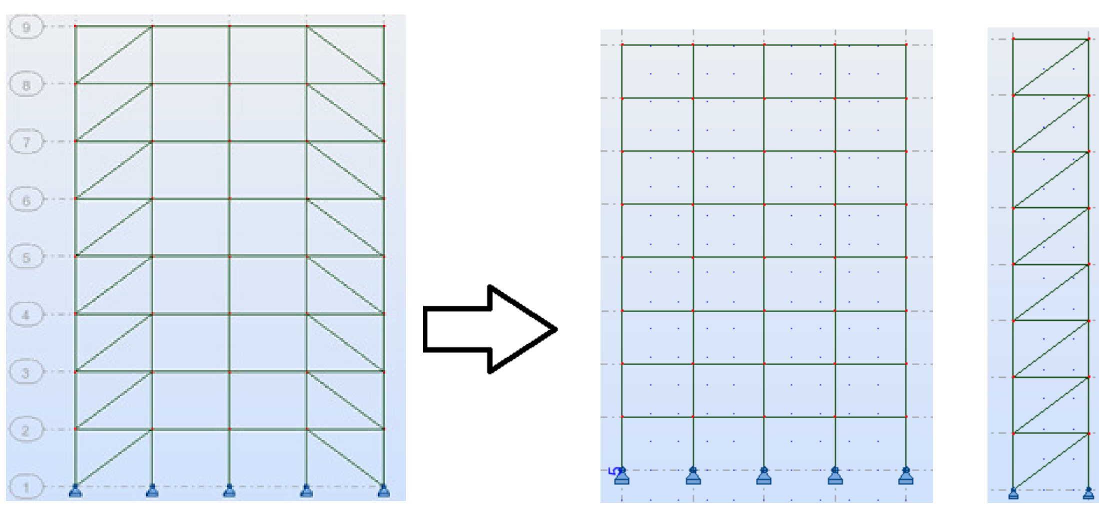


Figure 10: Simplified Structural Layout for gravity load (middle) and lateral load (right)

As shown, the transformed model only allows the gravity load to transfer down the ground through columns and beams, and the pushover load only induce tension in the brace set, and consequent axial loads on relevant columns and beams, hence nullifying the role of compression brace in the pushover curve. The curtailment simply eliminates irrelevant parts that are assumed not to participate in structural resistance.

3.1.8.1 Structural design step by step

Step 1: internal load determination for ULS

After the gravity and seismic load is determined on each floors (which is dependent on design methodology DCL/DCM) member internal loads can be determined by simple structure analysis. As previously mentioned, members internal loads are determined by 2 parallel models, one for seismic action and one for gravity load. Below are some figures illustrating the members loads calculated in ROBOT.

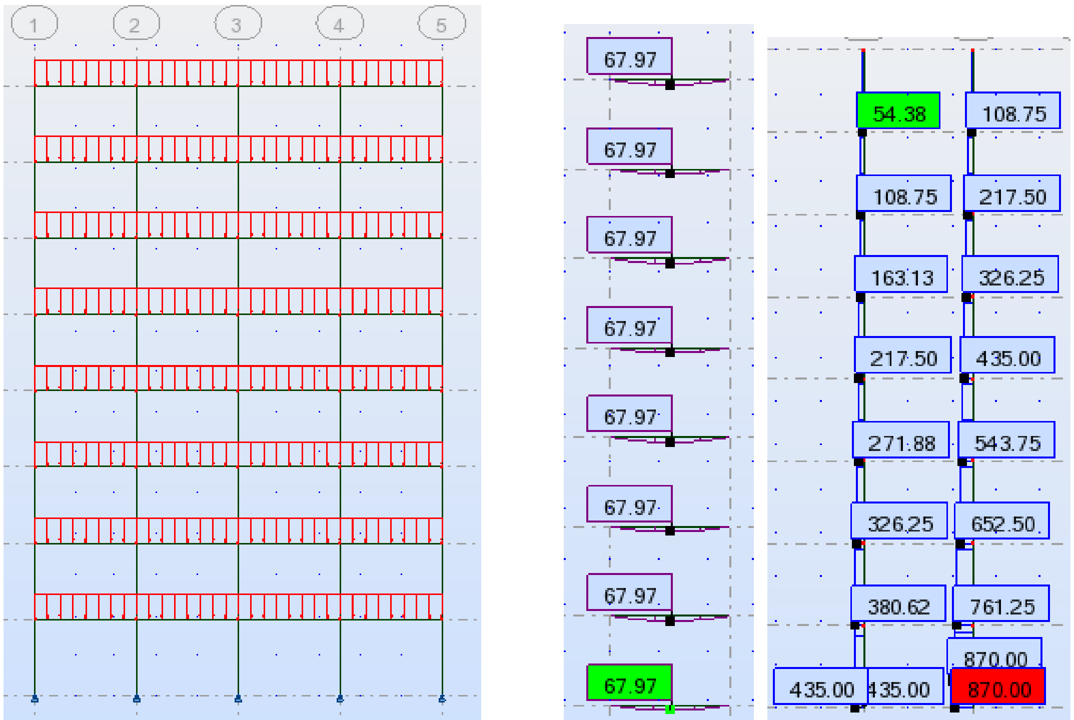


Figure 11: Frame internal load under gravity load

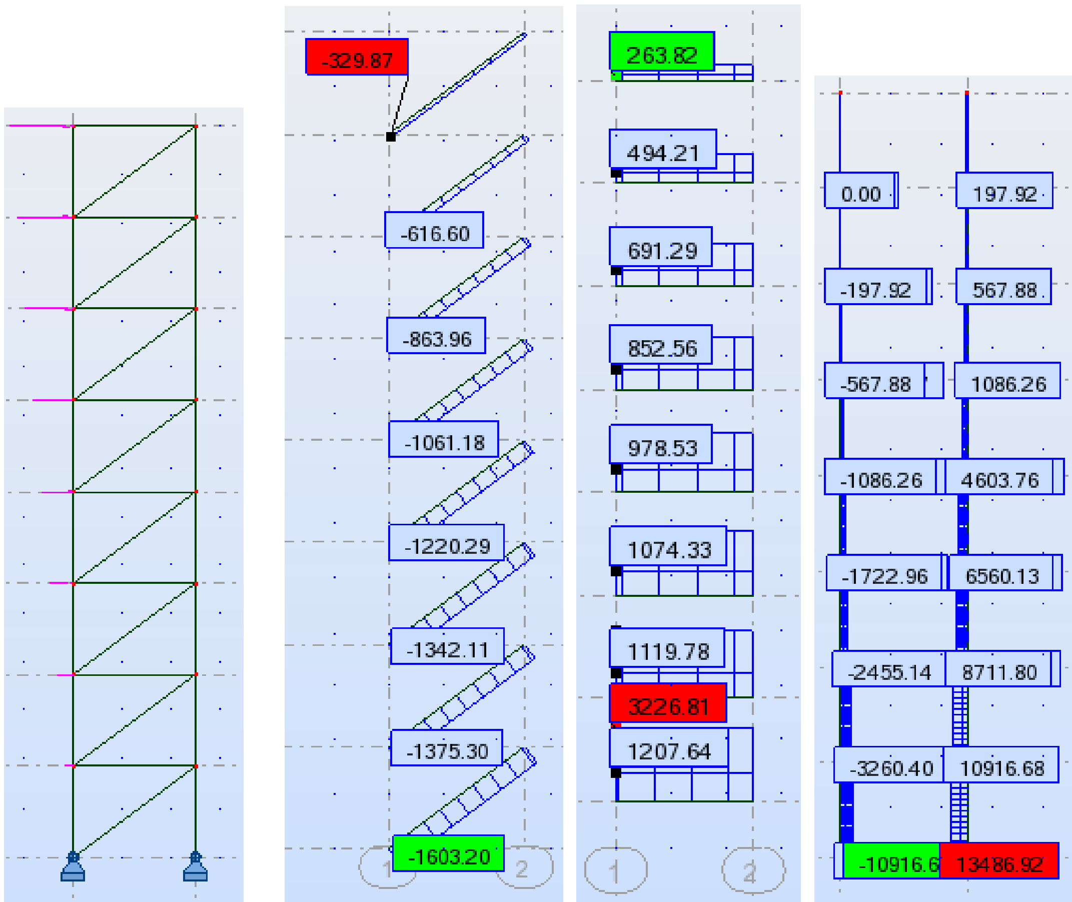


Figure 12: Frame internal load under the seismic action

Step 2: Brace design

It is advisable to start the design with dissipative components, as they determine the overstrength factor for non-dissipative members. In this case, braces are on the top list.

Two rules are to be respected in brace design: homogeneity rules, slenderness limit rules.

Bracing load (kN)	section	λ <2	over strength (max/min)<1.25
330	102x4	1.91	1.41
618.75	108x7	1.85	1.35
866.25	127x8	1.57	1.34
1072.5	133x10	1.51	1.40
1237.5	140x10	1.45	1.36
1361.25	152x11	1.33	1.33
1443.75	168x10	1.14	1.33
1485	194x11	1.05	1.38
		ok	max/min=1.06

Table 5: Typical example of brace design conforming homogeneity and slenderness rules (DCM-ULS-0.15ag-8 floors)

Step 3: beam design

Not only the beams need to take the combined load generated by the gravity and seismic load action, but it is also to abide with the capacity formula provided by EC8 part 1.

$$N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1{,}1\gamma_{ov}\ \Omega.N_{Ed,E}$$

beam moment	Beam shear	Compression =comp*1.1*1.25*min overstrength	section	check
70	55.2	680.5	HEB160	0.91
70	55.2	1276.0	HEB180	0.96
70	55.2	1786.3	HEB200	0.95
70	55.2	2211.7	heb240	0.92
70	55.2	2551.9	HEB240	1.03
70	55.2	2807.1	HEB260	0.93
70	55.2	2977.2	HEB260	0.98
70	55.2	3062.3	HEB260	1.00

Table 6: Typical example of beam design respecting capacity rule (DCM-ULS-0.15ag-8 floors)

Step 4: Column design

Columns follow the same set of rules as beam.

seismic comp	gravity comp	combined load with capacity load	moment	section	check
198	111	493.7876	0.703711	HEB140	0.90
569.25	223	1323.514	10.05301	HEB240	0.85
1089	336	2441.332	27.64577	HEB260	0.99
1732.5	448	3797.392	95.50358	HEB300	0.95
2475	561	5345.845	193.5204	HEB400	1.02
3291.75	673	7036.844	283.9975	HEB550	0.98
4158	785	8823.54	276.4577	HEM550	0.93
5049	899	10660.08	0	HEM700	1.02

Table 7: Typical example of column design respecting capacity rule (DCM-ULS-0.15ag-8 floors)

Step 5: Proceeding to DLS

The only difference between ULS and DLS is limit of inter-story drift, which is restricted by the formula:

$$d_r v \leq 0{,}0075\,h$$

Emperically, the top floor inter-story drift hold the highest value, therefore, control the DLS requirement

To make the work simple, all the DLS structural member capacity are multiples of ULS counterparts and the ratio between ULS maximum inter-story drift and DLS allowable value. In other words, DLS is simply a multiplication of ULS in term of capacity, and conversely, ULS is a multiplication of DLS in term of displacement.

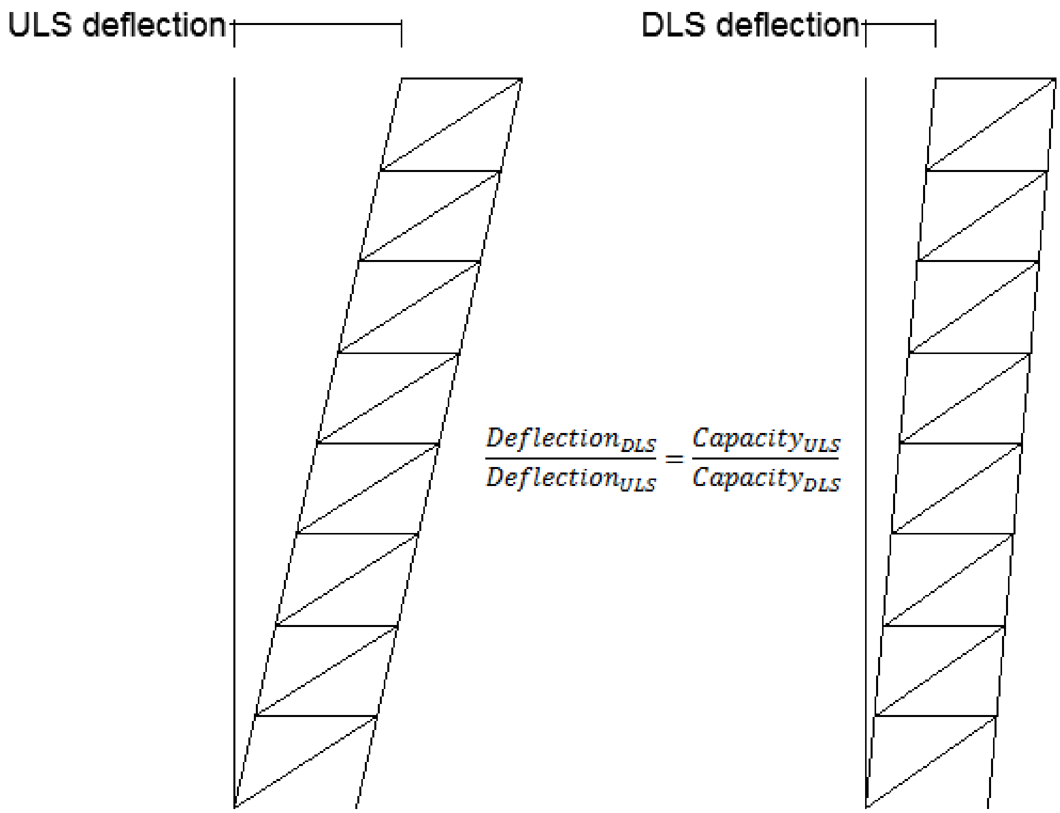


Figure 13: deflection comparison and design philosophy for DLS structure

For example, the top floor drift in ULS in case of DCM-ULS-0.15a_g-8 floors is 32mm, when the allowable value is 14mm. The ratio is roughly about 2.4, which is also the overstrength we aim at.

Bracing load (kN)	section	λ <2	over strength (max/min)<1.25
330	108x8	1.88	2.70
618.75	127x14	1.68	2.80
866.25	133x16	1.59	2.41
1072.5	152x18	1.39	2.51
1237.5	194x16	1.05	2.57
1361.25	324x10	0.60	2.57
1443.75	324x10	0.60	2.43
1485	324x12.5	0.60	2.92
		ok	max/min=1.21

Table 8: typical braces design (DCM-DLS-0.15ag-8 floors) with overstrength being multiple of ULS case

Column and beam sections in DLS cases are chosen accordingly as shown in ULS, only now with higher overstrength factor.

3.2 Finelg Model for Pushover Analysis

After all of the frame cross sections are determined by manual structural analysis according to EC8 and EC3, the frame ductility, overstrength factor and overall reliability are quantified by employing FinelG model to achieve pushover curve of every individual frame.

There are a few notes to make about modelling in FinelG.

3.2.1 Material definition

As steel is the main material in the frame, there are 2 materials defined in the input file: steel, and connection. For steel, two values input are necessary for bi-linear material rule as in figure: stiffness (Young Modulus E=210000MPa), yield strength (f_y=355MPa).

However, due to convergence problem in some cases, strain hardening must be added to smoothen the curve increment. Usually, the strain hardening angle is smaller at 1/100 that of Young Modulus E (E_t<2100MPa).

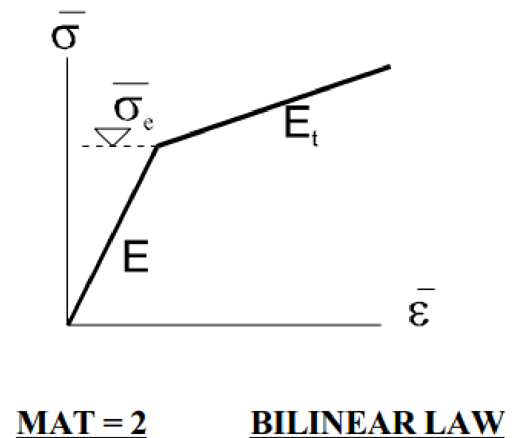


Figure 14: Bilinear material laws

3.2.2 Element Type

Beam and columns are modelled by classical beam elements in plan frame with 3 nodes. For both beam and column, the length of 1 element is 1.25m. Therefore, the column height of 3.75m consists of 3 elements and beam length of 5m, 4 elements. This type of cross section is numbered ISEC=34. On element ends, support condition can be adjusted as either pinned or fixed support. In this model, they are pinned for intersection of beams and columns, column to base support; and fixed otherwise.

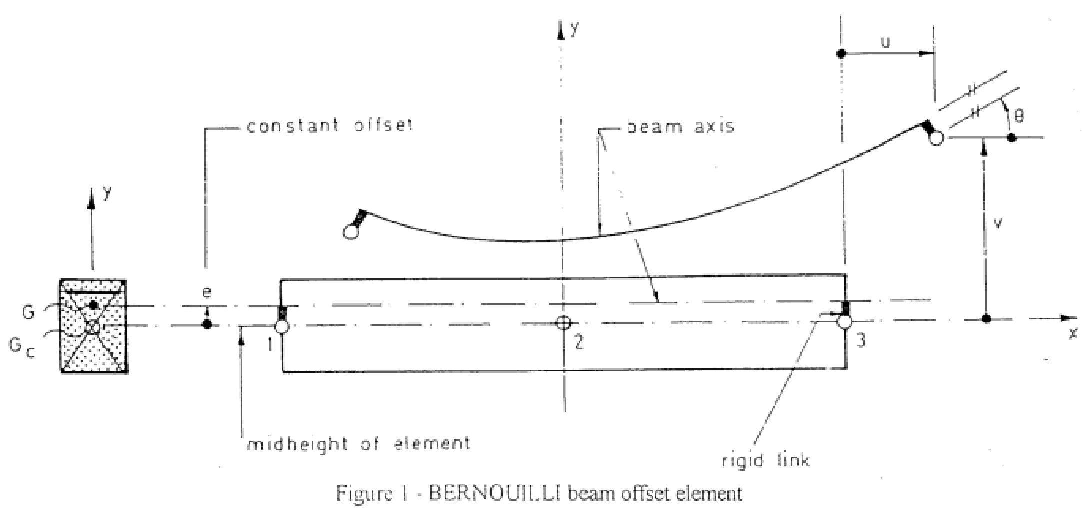


Figure 15: FinelG Beam Element

The brace members, on the other hand, are modeled by plane truss element with 2 nodes, which are capable of resisting only axial loads. For this type of element, ISEC=65. On element end, each node has 3 d.o.f in spaces.

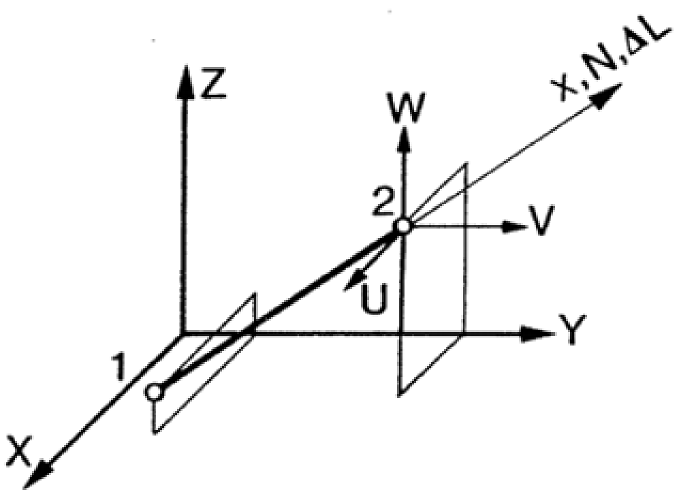


Figure 16: FinelG spacial truss element

3.2.3 Cross section definition

There are different format for each element cross section input. Beam elements requires dimensional values of flanges and web length and thickness with radius at intersection, and is input as following.

IGEO	ISEC	H	B	T_{web}	T	R	

Truss element for braces is only represented by cross sectional area.

IGEO	65	A

Typical geometry input for the 4-floor frame is presented below.

```
GEOM
   1  34        0.28       0.28      0.0105      0.018
   2  34        0.24       0.24        0.01      0.017
   3  34        0.18       0.18      0.0085      0.014
   4  34       0.140       0.14       0.007      0.012
   5  34        0.18       0.18      0.0085      0.014
   6  34        0.18       0.18      0.0085      0.014
   7  34        0.16       0.16       0.008      0.013
   8  34        0.16       0.16       0.008      0.013
   9  65   0.0027992
  10  65   0.0025133
  11  65   0.0018941
  12  65   0.0011129
GEOM_END
```

3.2.4 Node definition

Each node is simply defined by specifying its coordinates x and y. Following the element required length and the number of nodes in one element, it is determined that the node distance, as plotting along frame geometry, is 0.625m.

Element definition

After the overall frame geometry is formed with nodes, the next step is to define each element to its set of nodes, element type, sectional geometry and end support condition with the following format

1			16													64	67				70			73		76				80
NELM	TYPE	IMEC	IGEO	NODES (max. 12)													i	j	m	IDEM	S	I3	NUIT	I5		INES	GLST		INTA	I9	IKT

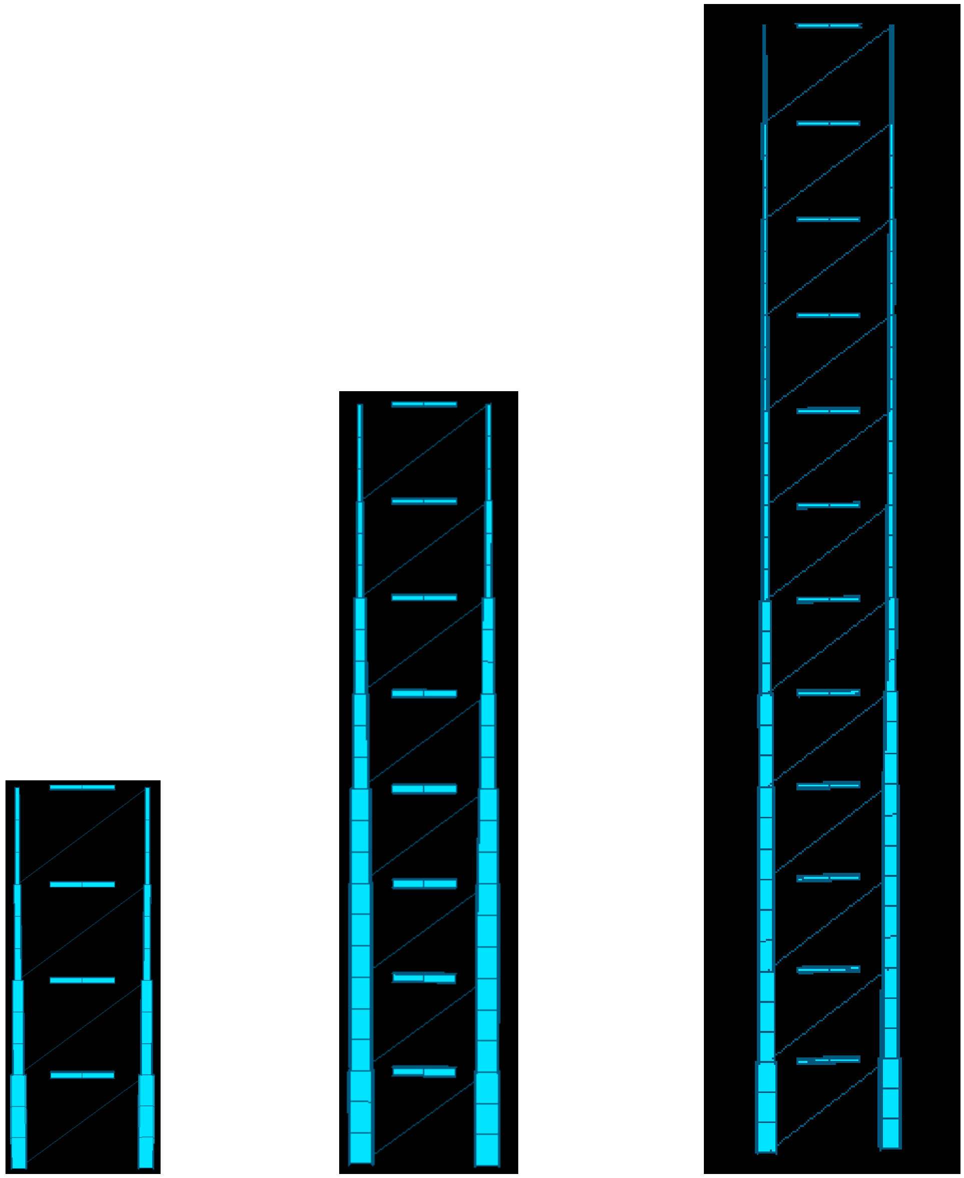


Figure 17: FinelG model for different numbers of story (4/8/12)

It is noted that in the case of 12 floors, because the frame is better braced (refer to figure 3) the load and mass for each simplified model as in figure 4 is reduced by half as compared to its counterpart in 4 or 8 floor. This, however, has little effect on the structural performance in term of ductility and reliability characteristics.

3.2.5 Output

The model output provides structural displacements at each load increment, which is extracted and utilized to draw the pushover curve, the curve plotting base shear force against the top displacement.

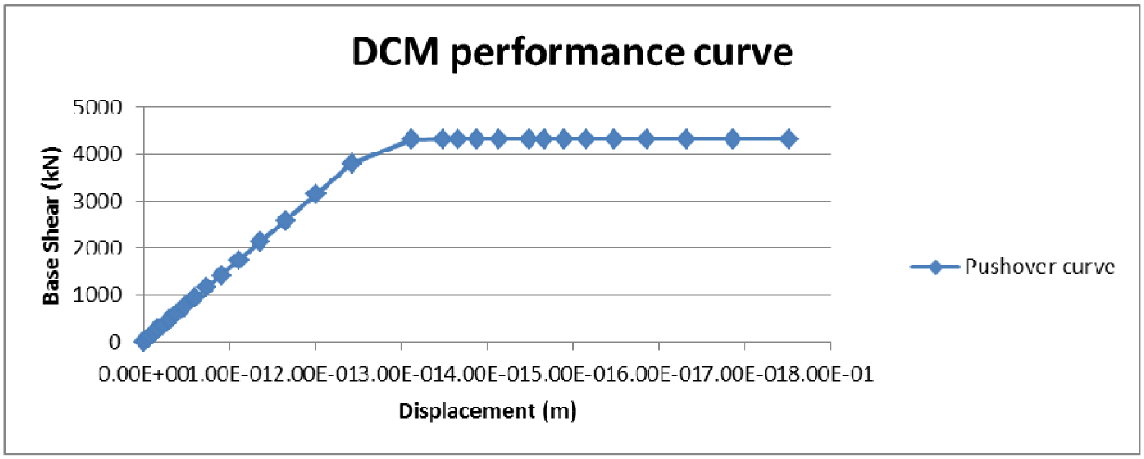


Figure 18: Example of pushover curve obtained from FinelG

This curve is then further analysed employing Annex B of EC8 part 1, transforming the structure to single degree of freedom (SDOF) system by a set of formula.

$$m^* = \sum m_i \Phi_i = \sum \overline{F}_i$$

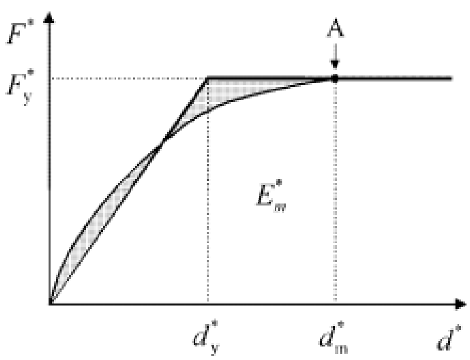
$$\Gamma = \frac{m^*}{\sum m_i \Phi_i^2} = \frac{\sum \overline{F}_i}{\sum \left(\frac{\overline{F}_i^2}{m_i} \right)}$$

$$F^* = \frac{F_b}{\Gamma}$$

$$d^* = \frac{d_n}{\Gamma}$$

m is the mass of story i

Fi is normalized displacement with Fn=1 with n is roof level



$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*} \right)$$

Figure 19: Pushover curve idealization

3.2.6 Plastic mechanism

The plastic mechanism is reached as soon as the first column/beam or brace in tension reach the Life Sasfety (LS) acceptance criteria in FEM 356*.

The target displacement of the structure will then will evaluated based on the formula:

$$T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}}$$

$$d_{et}^{*} = S_e(T^*)\left[\frac{T^*}{2\pi}\right]^2$$

With $S_e(T^*)$ follow the set:

$$0 \le T \le T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right]$$

$$T_B \le T \le T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C}{T}\right]$$

$$T_D \le T \le 4\text{s} : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right]$$

where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I . a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping, see **(3)** of this subclause.

*FEMA 356 suggests the following failure criteria for beam and brace. Those elements are deemed to fail when reaching the Life Safety (LS) limit.

All steel elements deformation curve are assumed to follow the same following shape.

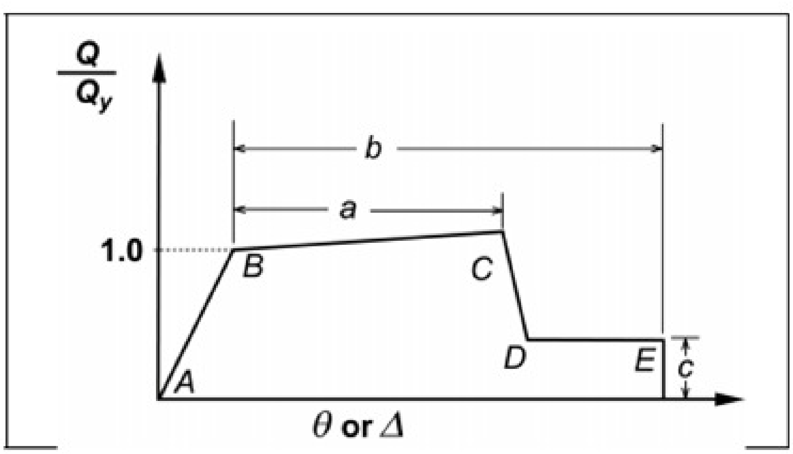


Figure 20: generalized force-deformation relation for steel elements or components

For Column, the LS limit is assumed to be 6Ɵy where Ɵy (member rotation at yield) is calculated as following

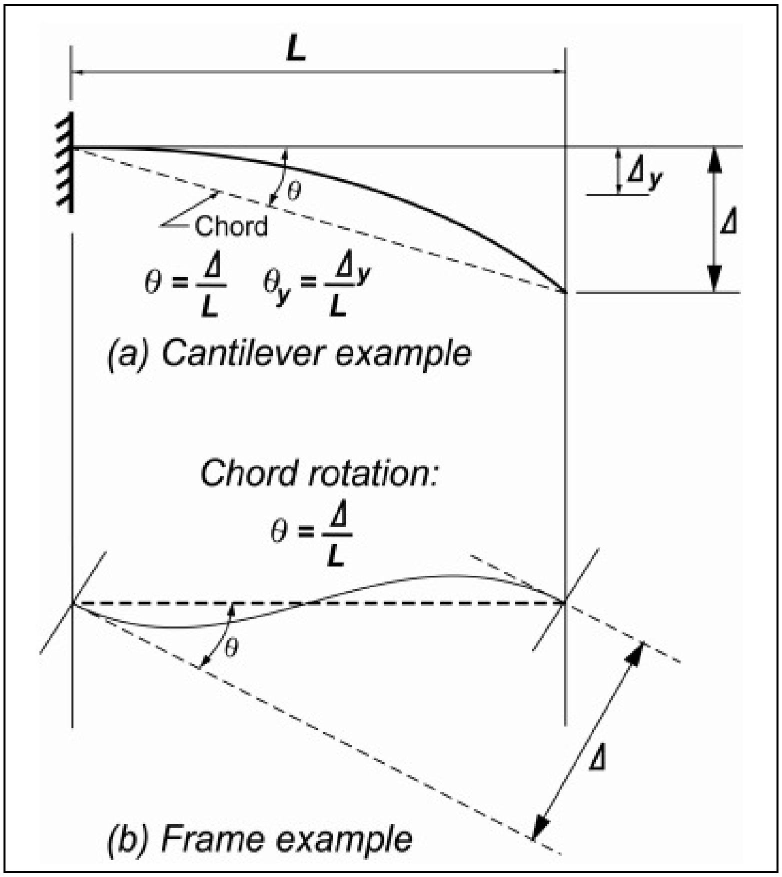


Figure 21: Definition of Chord Rotation

$$\text{Columns:} \quad \theta_y = \frac{ZF_{ye} l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right)$$

d_c = Column depth

E = Modulus of elasticity

F_{ye} = Expected yield strength of the material

I = Moment of inertia

l_c = Column length

M_{CE} = Expected flexural strength

P = Axial force in the member at the target displacement for nonlinear static analyses, or at the instant of computation for nonlinear dynamic analyses. For linear analyses, P shall be taken as Q_{UF}, calculated in accordance with Section 3.4.2.1.2

P_{ye} = Expected axial yield force of the member = $A_g F_{ye}$

θ = Chord rotation

θ_y = Yield rotation

Z = Plastic section modulus

For brace, the LS limit is assumed to be $7D_T$ where D_T is the axial deformation at the expected yield load.

$$\Delta_T = 7\frac{f_y}{E}L_c \; where \; f_y = 355MPa, E = 210000MPa \; and \; L_c = 6.25m$$

3.3 Study Parameter

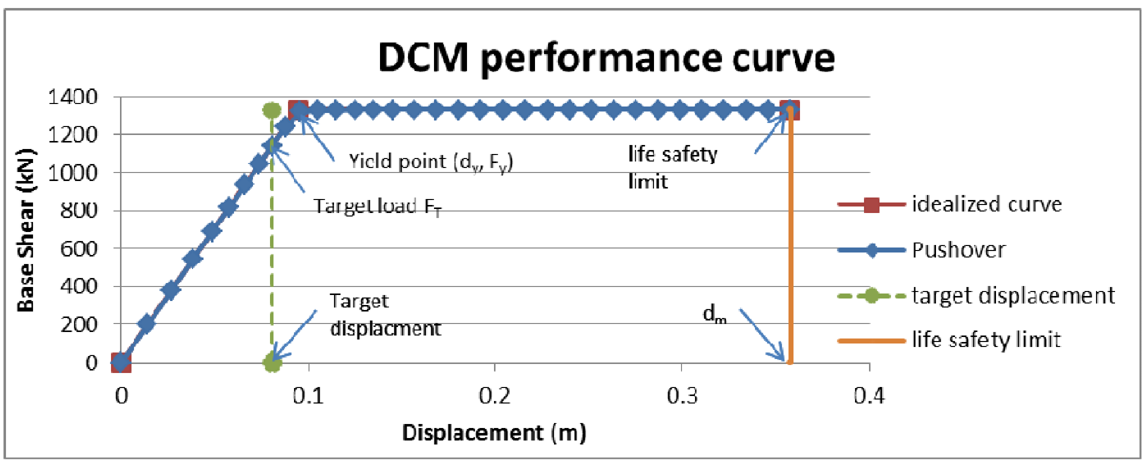


Figure 22: Pushover curve performance parameters

After the pushover curve is achieved, the parameters of behavior factor q, overstrengh factor, acceleration safety factor, will be derived.

Behavior factor (q) is defined based on the ratio of dm/dy. This factor actually indicates the capacity of the structure to deform beyond the yield limit.

Overstrength factor (OS) is the ratio of yield displacement dy and target displacement. If the value is higher than 1, it means the real imposed load in the structure is smaller than the design load, and the frame is still within the elastic range. The value lower than 1, on the other hand, means that the structure are on the plastic plateau.

Safety factor is the ratio of dm/dt, or can be computed as the product of q and OS. It gives general idea on how much more the acceleration the frame can withstand as compared to the imposed acceleration from seismic action.

These parameters are important to get insights into how the design process and guidelines affect the overall structural performance, based on which, further study on how to attune and streamline the design methodologies can be done.

3.4 General case study set

This set is made of structural frame designed according to several varying parameters. They include input ground acceleration (0.1g, 0.15g, 0.2g, 0.25g), design methodologies (DCL/DCM), design limit state (ULS/DLS), and number of storeys (4/8/12 floors). In total, the study consists of 48 cases.

It is noted that the ground accelerations are chosen across the spectrum of all possible cases in small-medium seismic region taking into account soil factor and importance factor

Range of acceleration value	0.1-0.12 a_g
Range of soil factor value	1-1.8
Range of importance factor	1-1.5
Total range	0.1-0.324 a_g

As we already choose soil B with factor 1.2, the range of our study should be 0.08-0.27a_g, which effectively equal 0.1-0.25 a_g.

These 48 cases are done to observe the variation trend of all the above mentioned parameters across the spectrum of height, seismic action, structural stiffness in general. Also, the problems that may arise along those changes would be the slenderness limit, overstrength homogeneity and can bring irregularities to the data. They are further scrutinized with further parametric study, in which individual design rules and guidelines are modified to check their effect on the structural performance. Thence, the effectiveness of those rules are understood and suggestion regarding relaxation

3.5 Parametric case study set

Based on the conclusion drawn from general case study set, the parametric study aims to understand the relationship between the structural behaviors and design guidelines. Certain rules and parameters will be adjusted and modified on a few related frames, so that their impact on ductility and reliability, as well as the sensitivity of that impact, can be observed.

3.6 Frequency and Dynamic analysis

In some of the parametric trials, the dynamic analysis is necessary to obtain the real effect of earthquake actions on structure. This analysis is carried out with FinelG and accelerogram generator Gosca.

Firstly, the accelerograms of different seismic load spectrums (from 0.1a_g to 0.25a_g) are churned out with predetermined set of attributes such as earthquake type (1) , soil type (B), duration (30s), critical damping (5%), transient function (Hanning's window) with time lag before and after hard phase (5 second), and standard (EC8). This is done with Gosca, a local software mainly for seismic signal generation according to input attributes. The example of such is shown below.

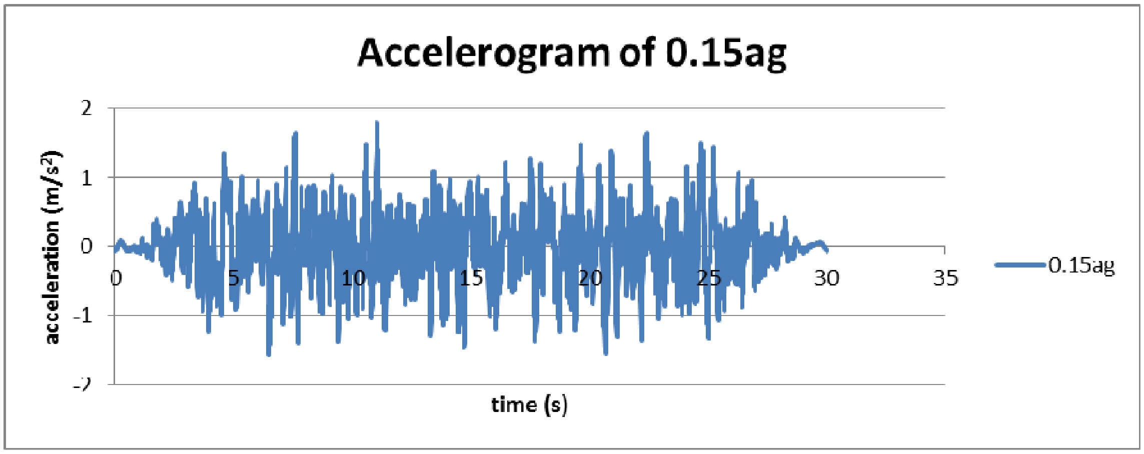


Figure 23: Example of accelerogram generated by Gosca

Secondly, frequency analysis is to obtain the structure periods, at least of the first two mode shapes, whose modal masses combined are deemed dominant in the frame dynamic behavior. Assuming that the damping matrix is a linear combination of mass and stiffness matrices, the values of periods are important to compute contribution parameters, α and β, from both matrices.

$$[C] = \alpha[M] + \beta[K]$$

With modal coordinate transformation, the equation can be rewritten

$$[\Phi]^T[C][\Phi] = [c] = \alpha[1] + \beta[\omega^2]$$

With $[c] = 2[\zeta\omega]$

$$c_i = 2\,\zeta_i\,\omega_i = \alpha + \beta\omega_i^2$$

$$\zeta_i = \alpha / (2\omega_i) + \beta\omega_i/2$$

If the damping ratios for the 1st and 2nd modes are ζ_1 and ζ_2, then the Rayleigh coefficients α and β are calculated from the solution of the two algebraic equations:

$$\frac{1}{2}\begin{bmatrix} 1/\omega_1 & \omega_1 \\ 1/\omega_2 & \omega_2 \end{bmatrix}\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \begin{Bmatrix} \zeta_1 \\ \zeta_2 \end{Bmatrix}$$

If both modes have the same damping ratio ($\zeta_1 = \zeta_2 = \zeta$), then the values of α and β are given by:

$$\alpha = \zeta\frac{2\omega_1\omega_2}{\omega_1 + \omega_2} \qquad \beta = \zeta\frac{2}{\omega_1 + \omega_2}$$

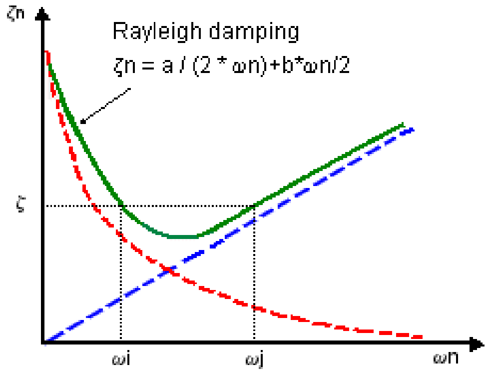


Figure 24: Rayleigh damping Matrix

Both the accelerogram and damping parameters α and β are input into dynamic analysis with FinelG, which simulate frame displacements under real earthquake action. As the whole frame, the braces of which take only tension, is curtailed into simplified frame with only one set of brace; those braces are modeled to take both tension and compression in dynamic situation. This assumption, therefore, only applies to this dynamic analysis and not to pushover analysis.

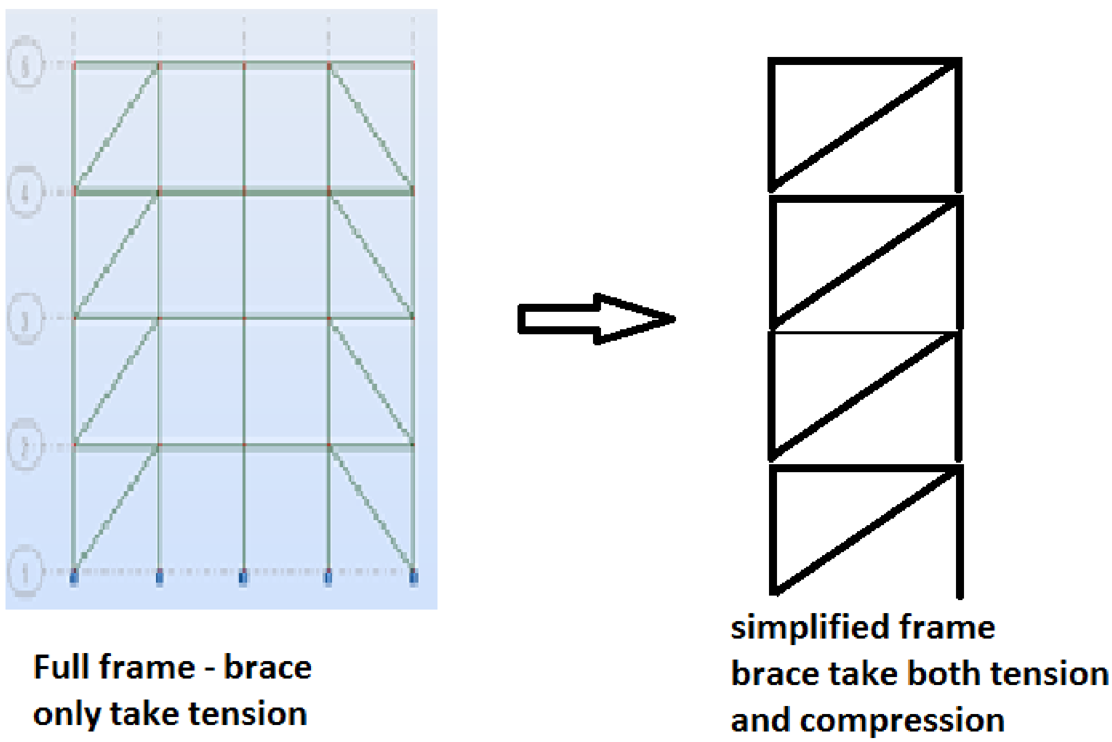


Figure 25: Transformation from full frame to to simplified frame for dynamic analysis

Obviously, the frame geometry and elements is modelled exactly in the same way as in pushover analysis. In addition, some changes are, of course, made on the type of analysis, seismic mass on each floor, and other relevant input values.

The model output is top floor displacement throughout the earthquake period, from which the max displacement is obtained and compared with the predicted value calculated from Annex B of EC8.

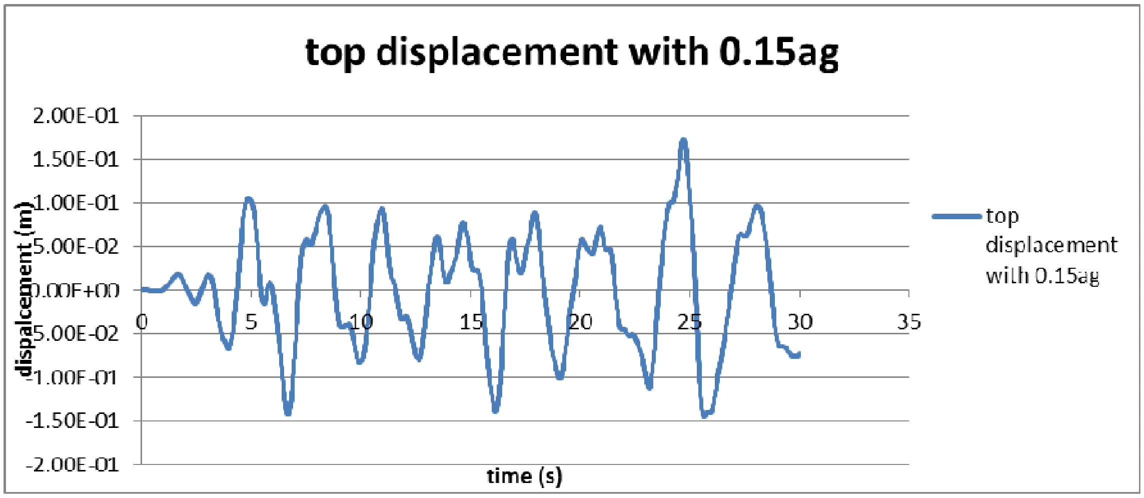


Figure 26: 12-story building top displacement when subject to 0.15ag

4 Case study result and interpretation

4.1 General case study set

4		0.1				0.15				0.2				0.25			
		q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor
dcm	uls	3.92	1.27	5.01	0.8375	3.92	0.85	3.34	0.8375	4.13	0.79	3.05	1.0025	3.4	0.68	2.32	1.1
	dls	3.92	1.27	5.01	0.8375	3.79	1.18	4.43	1.115	3.99	1.02	4.17	1.4975	3.7	0.96	3.57	1.825
dcl	uls	2.3	1.8	4.34	0.97	3.4	1.59	5.5	1.26	1.34	1.41	1.89	1.625	2.69	1.28	3.44	1.92
	dls																
8		0.1				0.15				0.2				0.25			
		q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor
dcm	uls	2.75	2.3	6.29	1.69175	2.75	1.5	4.2	1.69175	2.71	1.2	3.2	1.69175	2.43	1.05	2.48	1.74125
	dls	3.24	2.4	8.21	2.934625	2.66	2.1	5.8	3.07763	2.67	2	5.1	4.01788	2.6	1.8	4.75	4.61688
dcl	uls	2.78	2.8	7.98	1.796125	1.3	2.55	3.5	2.08213	1.38	2.07	2.86	3.20263	1.6	2.22	3.18	3.61988
	dls	1.38	3.3	8.6	2.476	1.1	3.26	3.41	2.55638	1.39	2.66	3.71	4.745	2.36	2.45	5.79	5.7475
12		0.1				0.15				0.2				0.25			
		q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor
dcm	uls	1.94	4.57	8.85	2.78	1.94	3.05	5.9	2.78	1.94	2.28	4.43	2.78	1.94	1.83	3.54	2.78
	dls	1.94	4.57	8.85	2.78	2.32	3.24	7.53	3.72	2.24	3.52	7.8848	5.35	1.96	3	5.88	6.22
dcl	uls	1.35	5.58	7.55	2.25	1.35	3.72	5.04	2.25	1.77	2.83	5.02	2.47	1.32	2.35	3.12	2.89
	dls	1.55	5.89	9.14	2.48	1.38	4.24	5.8512	3.46	1.98	3.86	7.6428	4.43	1.12	3.48	3.8976	5.71

Table 9: General case study data

q: ductility factor =dm/dy

OS: over strength factor =dy/dtarget

am/ag: ratio between acceleration at dm and imposed acceleration

Ton: the structure tonnage per floor (for consistency, the weight/per floor in 12 floor cases are doubled to give a comparable value with 4/8 floor)

4.2 Result Interpretation

4.2.1 General Observation

There are several observations that can be made for the cases of DCM.

- The structure stiffness varies across the cases. Obviously, when the frame is converted to SDOF system, the shorter the structure is, the stiffer it becomes in the same design condition. Also, for structure with the same height, those able to stand higher seismic input and those opted for DLS is stiffer than with lower seismic load and ULS respectively. Therefore, the bottom left case (12 storey-0.1ag-ULS) is the softest, and the top case (4 storey-0.25ag-DLS) is the stiffest. Please refer to Annex with complete data of frame periods.

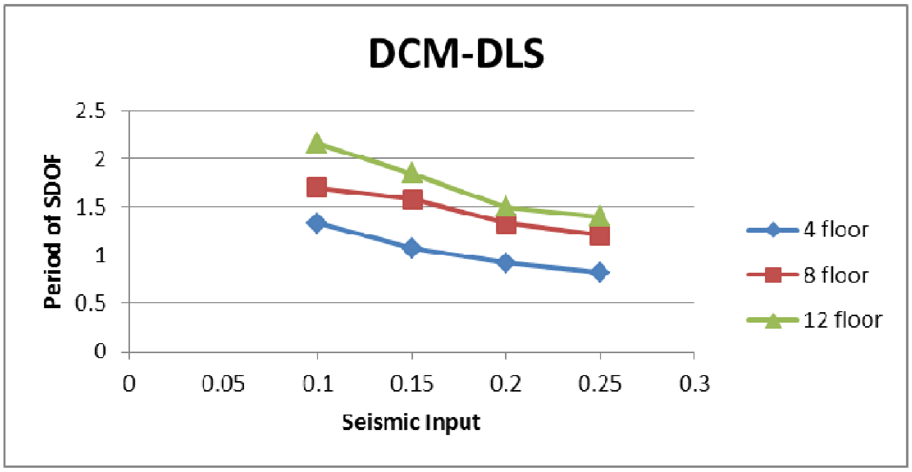


Figure 27: Stiffness comparison for DCM-ULS cases

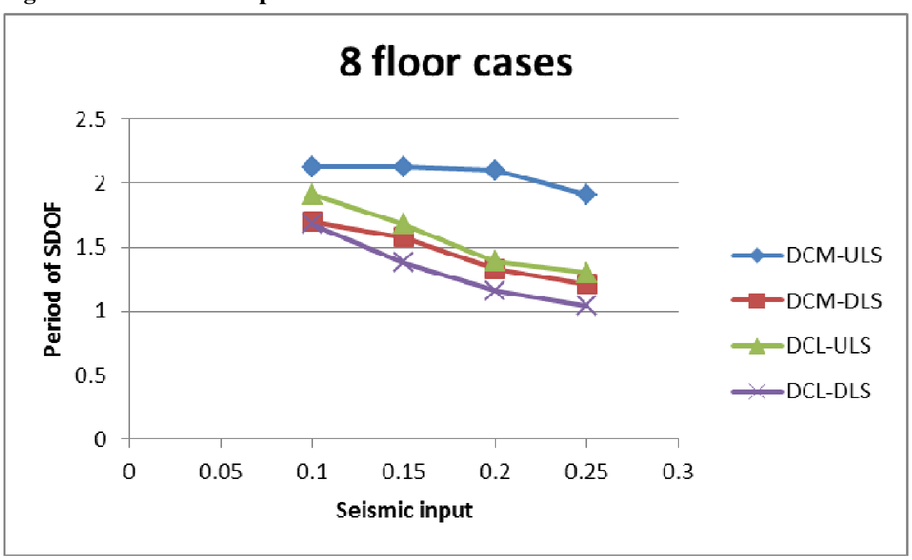


Figure 28: Stiffness comparison for all 8 floor cases.

- The behavior factor q show the clear trend of decreasing with increasing structure height. The seismic load, on the other hand, shows no effect on q.

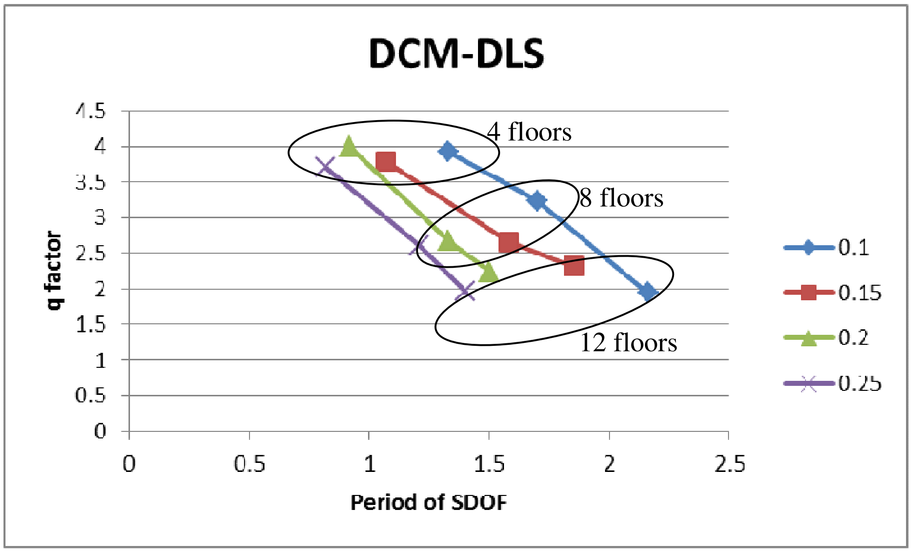


Figure 29: Q factor variation with T* period

- The overstrength factor (OS) goes up with the frame height but goes down with increasing seismic load.

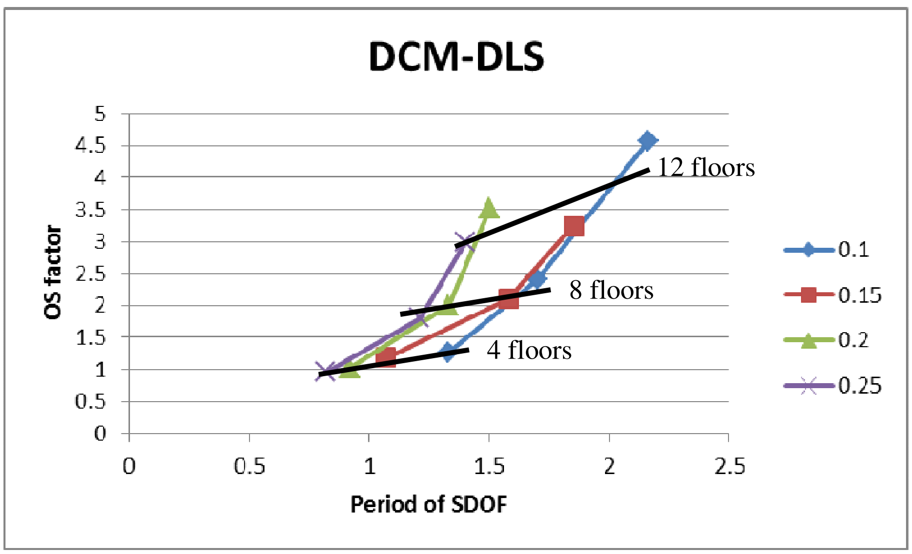
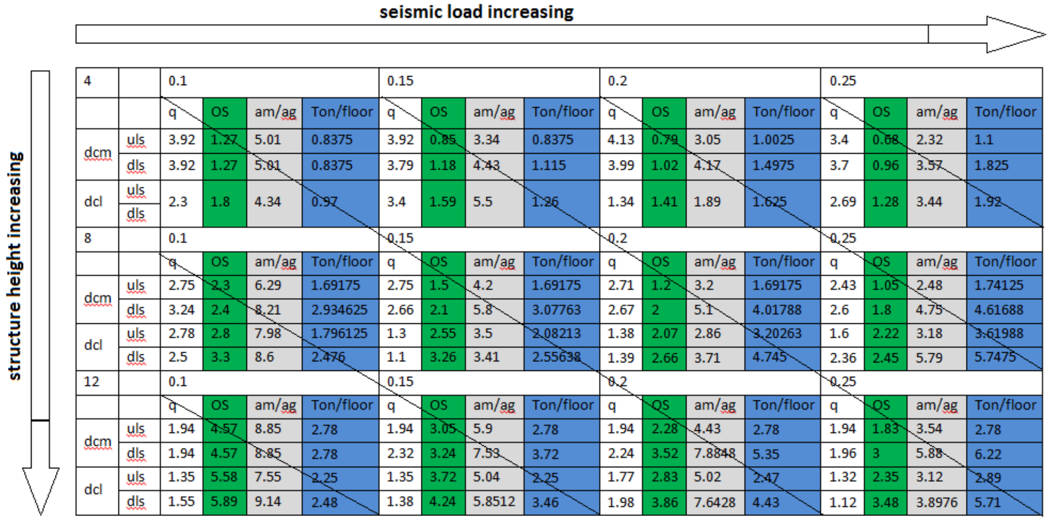


Figure 30: OS factor variation with T* period

- Therefore, the safety factor am/ag , the product of q and overstrength factor, also follow the pattern, which is illustrated below. All the cases on the same diagonal lines possess their safety factor in the same range.



4		0.1				0.15				0.2				0.25			
		q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor
dcm	uls	3.92	1.27	5.01	0.8375	3.92	0.85	3.34	0.8375	4.13	0.79	3.05	1.0025	3.4	0.68	2.32	1.1
	dls	3.92	1.27	5.01	0.8375	3.79	1.18	4.43	1.115	3.99	1.02	4.17	1.4975	3.7	0.96	3.57	1.825
dcl	uls / dls	2.3	1.8	4.34	0.97	3.4	1.59	5.5	1.26	1.34	1.41	1.89	1.625	2.69	1.28	3.44	1.92
8		0.1				0.15				0.2				0.25			
		q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor
dcm	uls	2.75	2.3	6.29	1.69175	2.75	1.5	4.2	1.69175	2.71	1.2	3.2	1.69175	2.43	1.05	2.48	1.74125
	dls	3.24	2.4	8.21	2.934625	2.66	2.1	5.8	3.07763	2.67	2	5.1	4.01788	2.6	1.8	4.75	4.61688
dcl	uls	2.78	2.8	7.98	1.796125	1.3	2.55	3.5	2.08213	1.38	2.07	2.86	3.20263	1.6	2.22	3.18	3.61988
	dls	2.5	3.3	8.6	2.476	1.1	3.26	3.41	2.55638	1.39	2.66	3.71	4.745	2.36	2.45	5.79	5.7475
12		0.1				0.15				0.2				0.25			
		q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor	q	OS	am/ag	Ton/floor
dcm	uls	1.94	4.57	8.85	2.78	1.94	3.05	5.9	2.78	1.94	2.28	4.43	2.78	1.94	1.83	3.54	2.78
	dls	1.94	4.57	8.85	2.78	2.32	3.24	7.53	3.72	2.24	3.52	7.8848	5.35	1.96	3	5.88	6.22
dcl	uls	1.35	5.58	7.55	2.25	1.35	3.72	5.04	2.25	1.77	2.83	5.02	2.47	1.32	2.35	3.12	2.89
	dls	1.55	5.89	9.14	2.48	1.38	4.24	5.8512	3.46	1.98	3.86	7.6428	4.43	1.12	3.48	3.8976	5.71

Figure 31: Safety factor variation pattern

For DCL cases, same conclusion can be made for q and OS with respect to the structure changing height. Also, compared to DCM cases, DCL possess higher OS but lower q factor. However, as the ductility is controlled, q factor varies randomly with the seismic load.

The table below summarizes the relationship of each performance indicator and design parameter.

	Height	Seismic load	ULS =>DLS	DCM =>DCL
q factor	-	0	0	-
OS	+	-	+	+
am/ag	+	-	+	+

Table 10: Relationship between parameter and performance indicator

4.2.2 Overstrength factor

4.2.2.1 OS vs Building Height

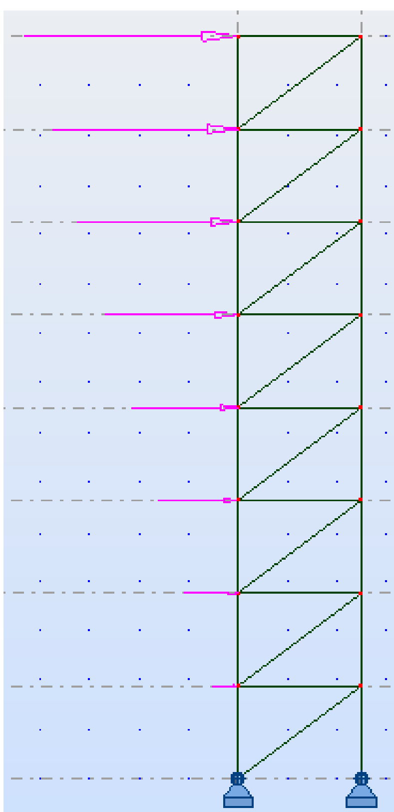
Observing the OS factor increasing with higher number of story, the following interpretation partly explains the reason. The table below presents the comparison of structure periods predicted by Annex B EC8 part 1 and those analysed by FinelG, together with their corresponding elastic response spectrum.

Number of floor	T_{EC8}	$x = \frac{S_e}{a_g}(for\ T_{EC8})$	T_{FinelG} $DCM - DLS - 0.25a_g$	$y = \frac{S_e}{a_g}(for\ T_{FinelG})$	$\frac{x}{y}$
4	0.37	3	1.06	1.42	2.11
8	0.64	2.34	1.89	0.79	2.96
12	0.87	1.72	2.87	0.36	4.77

Table 11: comparison between EC8 frequency prediction with FinelG prediction, and their consequence

It is clearly shown that the EC8 frequency prediction is increasingly conservative with rising height of the building, hence too the elastic response prediction. To a certain extent, this explains why the OS factor exaggerates with height. Further study to improve accuracy of eurocode prediction is recommended to address the excessive safety factor for tall structure under seismic load.

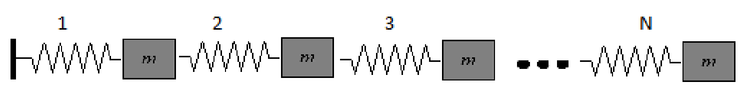
4.2.2.1 OS vs seismic load



It is also discerned that the OS factor varies in an inverse manner with seismic load. The following attempt is made to explain this trend.

D_y determination

The building is simplified into spring series which the combined stiffness respecting the series rules.



$$\frac{1}{K_N} = \frac{1}{K_N^1} + \frac{1}{K_N^2} + \frac{1}{K_N^3} + \cdots + \frac{1}{K_N^N}$$

Due to the proportional relation between F and K on each level, we have the following relation between K, assuming K_N^N is the top brace stiffness.

$$K_N^a = K_N^N\left(\frac{n + (n-1) + \cdots + a}{n}\right)$$

Substituting the relation into the combined stiffness formula, we have:

$$K_4 = \frac{K_4^1}{6.05} \qquad K_8 = \frac{K_8^1}{14.31} \qquad K_{12} = \frac{K_{12}^1}{23.53}$$

$where\ K_4 \quad K_8 \quad K_{12}\ is\ stiffness\ of\ the\ whole\ structure,$
$K_4^1 \quad K_8^1 \quad K_{12}^1\ is\ the\ bottom\ brace\ stiffness$

The first story brace stiffness is chosen here because, most of the cases, the bottom brace would yield first, at which point the whole structure is deemed to reach yield limit. At yield, assuming the bottom story drifts a distance of x. we have

$$K_4 x = \frac{K_4^1 x}{6.05} \qquad K_8 x = \frac{K_8^1 x}{14.31} \qquad K_{12} x = \frac{K_{12}^1 x}{23.53}$$

$$d_{y,4} = 6.05x \qquad d_{y,8} = 14.31x \qquad d_{y,12} = 23.53x$$

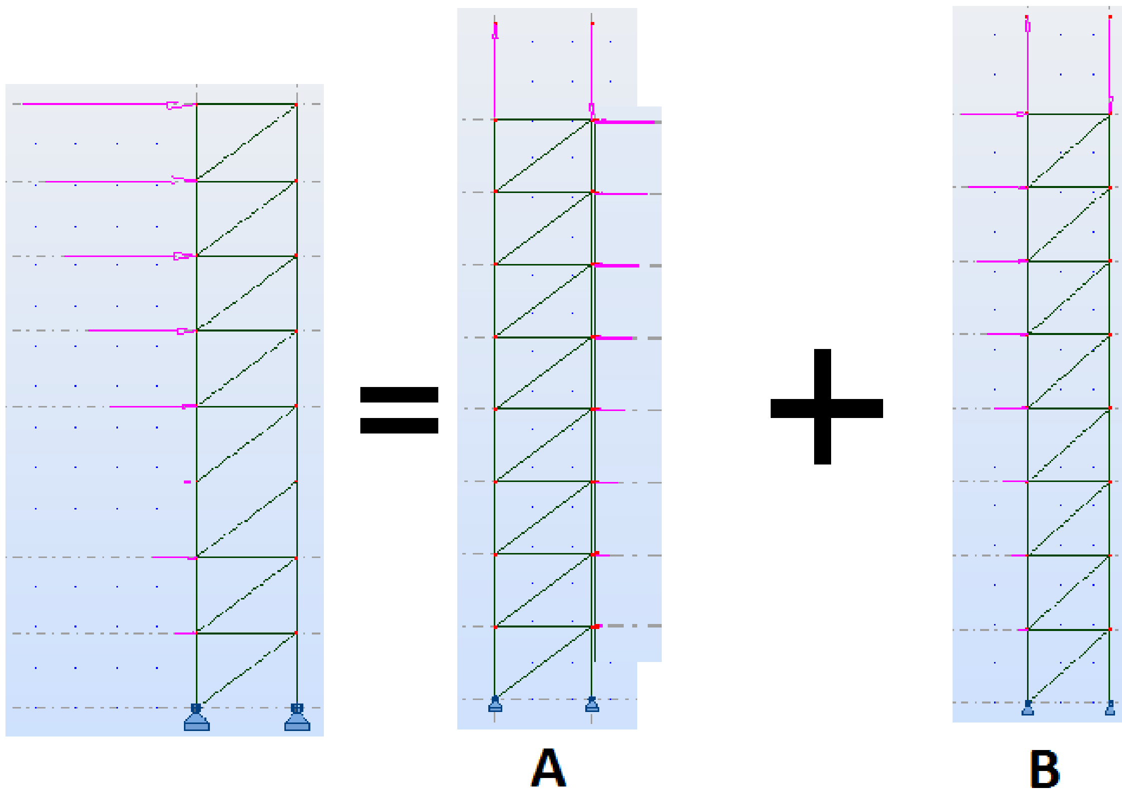
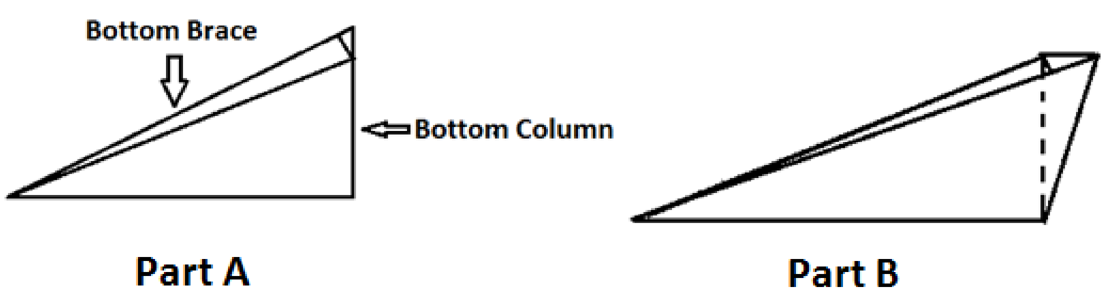


Figure 32: Frame action superposition

In the attempt to find the first story drift at yield, we divide the structure behavior into 2 parts. In part A, The structure has no horizontal movement, only vertical movement of column due to axial loads; and the braces are subjected to compressive strain due to that vertical movement. In Part B load configuration impose load only on beams and braces, with no net effect on column. The whole structure deflects purely horizontally, and the brace simply starts from no stress to yield point.

So totally, the bottom brace need to overcome both the compressive strain from part A and tension strain from part B to reach the yield point.



In Part A, assuming the column utilization factor is almost 1, the induced stress in the brace can be calculated as following (column height is 3.75m, brace length is 6.25m, the angle between them is 53o)

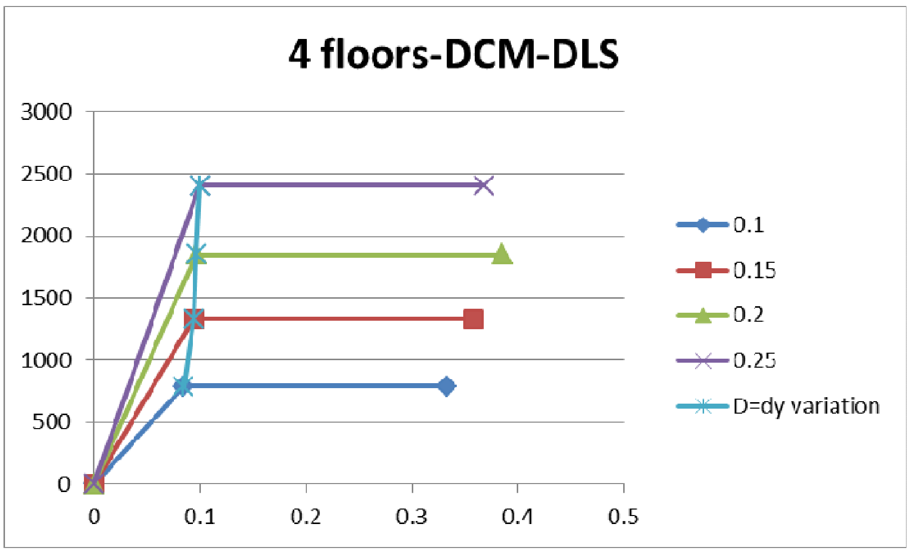
$$\frac{355}{210000} * 3750 * \frac{\cos(53)}{6250} * 210000 = 128MPa$$

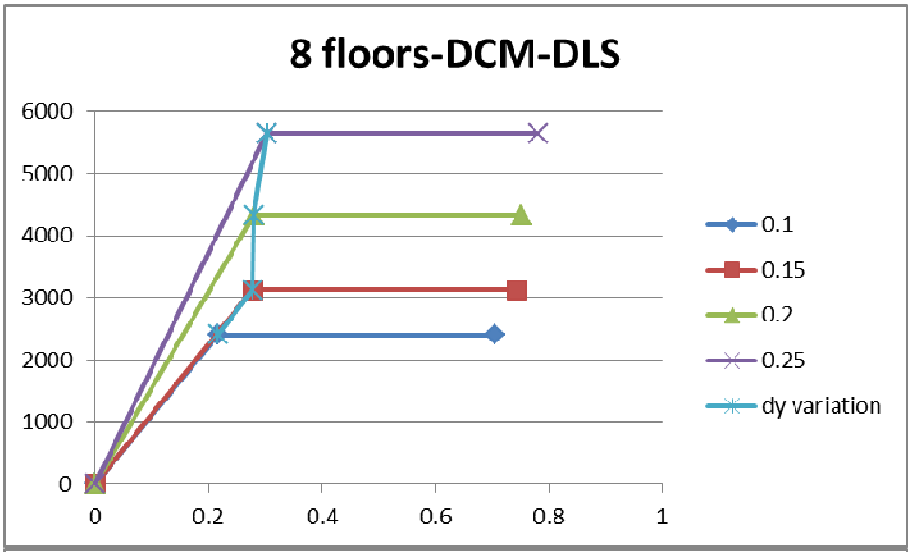
To reach yield in tension, the brace undergo elongation from -128MPa to +355MPa. This process incurs a horizontal displacement calculated as:

$$\frac{(355+128) * 6250}{210000 * \cos(90-53)} = 18mm = x$$

$d_{y,4} = 6.05x = 109mm \qquad d_{y,8} = 14.31x = 257.6mm \qquad d_{y,12} = 23.53x = 425mm$

This result generally agrees with the case study, taking example of the case DCM-ULS-0.25ag. $d_{y,4} = 93mm \qquad d_{y,8} = 261mm \qquad d_{y,12} = 450mm$





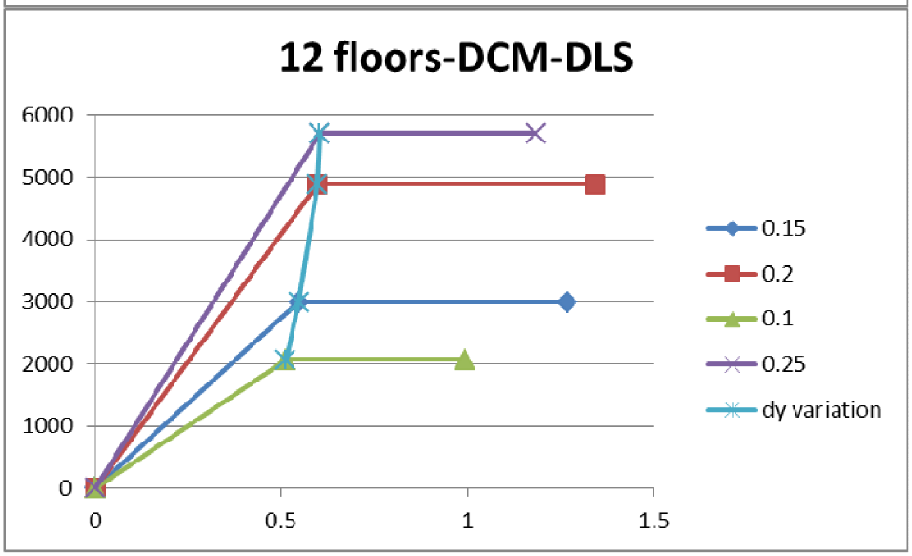


Figure 33: Comparison graph showing the dy consistency in 4/8/12 story cases

Therefore, it can be concluded that dy for each groups of building height is fixed. On the other hand, dt (target displacement) varies proportionally with ag.

$$d_{et}^{*} = S_e(T^*)\left[\frac{T^*}{2\pi}\right]^2$$

$$T_C \leq T \leq T_D : \ S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C}{T}\right]$$

$$T_D \leq T \leq 4s : \ S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right]$$

Therefore, OS= dy/ dt reduces with ag.

4.2.2.1 OS vs Design Limit State and Design Methodology

$$for\ T_C < T < T_D \quad d_{et} = a_g * S * \eta * 2.5 * \frac{T_C}{T} * \left(\frac{T}{2\pi}\right)^2 = a_g * S * \eta * 2.5 * \frac{T_C * T}{4\pi^2}$$

$$for\ T_D < T \quad d_{et} = a_g * S * \eta * 2.5 * \frac{T_C * T_D}{T^2} * \left(\frac{T}{2\pi}\right)^2 = a_g * S * \eta * 2.5 * \frac{T_C * T_D}{4\pi^2} = const$$

$if\ a_g\ is\ fixed$

$$so\ for\ T_C < T < T_D \quad d_{et} = \left(a_g * S * \eta * 2.5 * \frac{T_C}{4\pi^2}\right) * T < \left(a_g * S * \eta * 2.5 * \frac{T_C}{4\pi^2}\right) * T_D$$

$and\ d_{et}\ is\ linearly\ proportional\ to\ T$

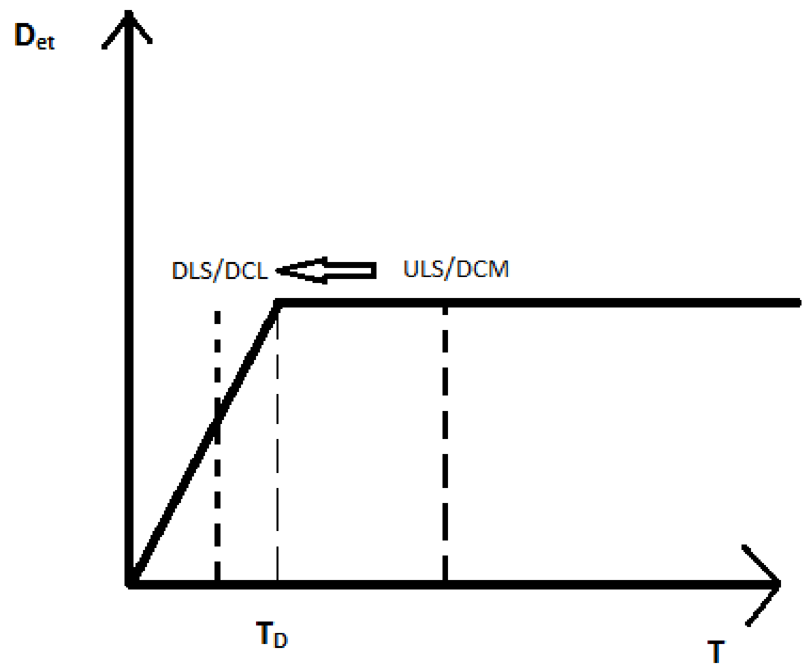


Figure 34: Variation of target displacement against structural stiffness

Changing the design limit state (ULS=>DLS) and design methodology (DCM=>DCL) amounts to increasing the frame stiffness, which may make the T* drop from $T>T_D$ to $T<T_D$, resulting in reduction of target displacement d_t for the same seismic action a_g. Taking into account that the yield displacement d_y is unchanged for a fixed value of a_g, OS= $d_{y/}\ d_t$ will increase.

4.2.3 Q factor

4.2.3.1 Q factor vs Height

The data show clear drop in q value for higher building. It is observed (in the output file) that, with higher number of floors, the braces yield in less uniform pattern. In other words, all four braces in 4-story frame are likely to reach the yielding and approach the LS limit in parallel when, in 12 story frame, some braces already reach LS limit and some others are still below yield point. Those yielding patterns explain partly the converse relation between q value and height.

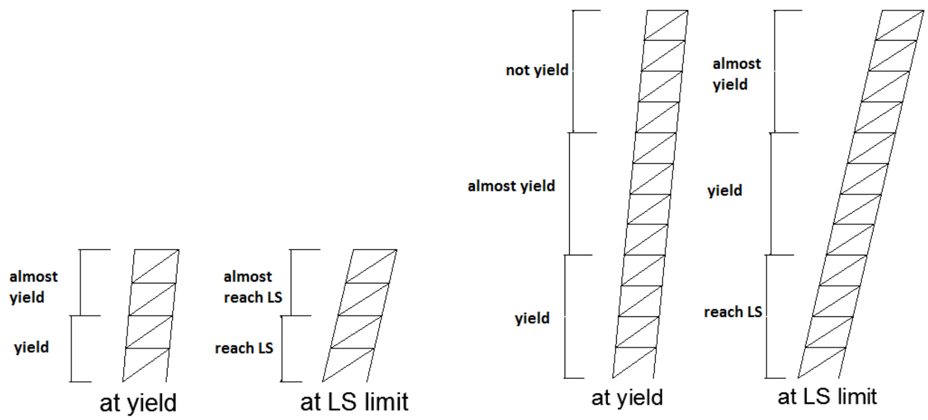


Figure 35: Illustration of brace yielding pattern of 4 and 12 story building.

4.2.3.1 Q factor vs Design Methodology

Changing the design methodology means allowing less ductility behaviour to be utilized and releasing rules and guidelines for guarantee of such ductility. Hence, the q factor drop can be attributable to 2 reasons:

- The non-uniformity of brace yield, as the homogeneity rule for brace overstrength is no longer conformed.
- Premature yielding of non-dissipative member, specifically columns. Those members, which yielding and failing point, determined according according to FEM356, corresponds to that of the whole structure, exhibit low ductility at LS limit.

4.2.4 Safety Factor

As the safety factor is the product of q factor and OS factor, it clumped together the characteristics of both factors, providing the overall safety level in a single indicator. Obviously, the safety factor on the top right corner (case 4 story-DCM-ULS-0.25a_g) has an agreeable value of 2.3, while that of the bottom right corner (case 12 story-DCL-DLS-0.1a_g) is unreasonably high, about 9.

4.2.5 Problematic design rules

Having perusing through the general case study, there are typical problems that design engineers found tricky and difficult to conform.

- In practical design situation, it's not always possible to abide to homogeneity rule 25% limit of overstrength.
- The slenderness limit λ<2 force many braces to be excessively over-strengthened. Combined with the homogeneity rules, the overstrength problem affect the whole structure, leading to unreasonable wastage of material tonnage.
- It is practical to simplify the whole frame with 4 spans into a frame with 1 span, of which only the relevant structural members are present. This simplification may inadvertently omit the contribution of number of columns to overall structural ductility and reliability.

The next section – parametric case study set, aims to testify the effect of those guidelines on the frame performance by adjusting and modifying the rules. At the same times, it also purposes to alleviate the unnecessarily excessive safety factor currently associated with soft structures.

5 Parametric study

5.1 Brace overstrength limit

In some cases, due to unavailability of the section from steel supplier, the design engineer found it challenging to restrict the brace overstrength within the 25% limit. In this case set, the brace overstrength homogeneity rule is altered. The ratio between max and min overstrength is allowed to vary up to 35% and 50%. This experiment is carried out for 4 –story frames under DCM-DLS design requirement with seismic of $0.2a_g$. The following assumptions are made to ensure the consistency.

- The load distribution along all stories remains triangular until LS limit, as the FinelG pushover analysis load distribution is fixed throughout the increment.
- LS limit of the frame is defined as when first dissipative member reaching its failure point (when the brace strain reached 7 times the yield strain).

In addition, 2 overstrength configurations are investigated, in which, one has much overstrength in first floor and third floor braces when the other has it in the second and top brace. Depending on the brace yielding sequence, each configuration manifests different ductility, hence safety factor.

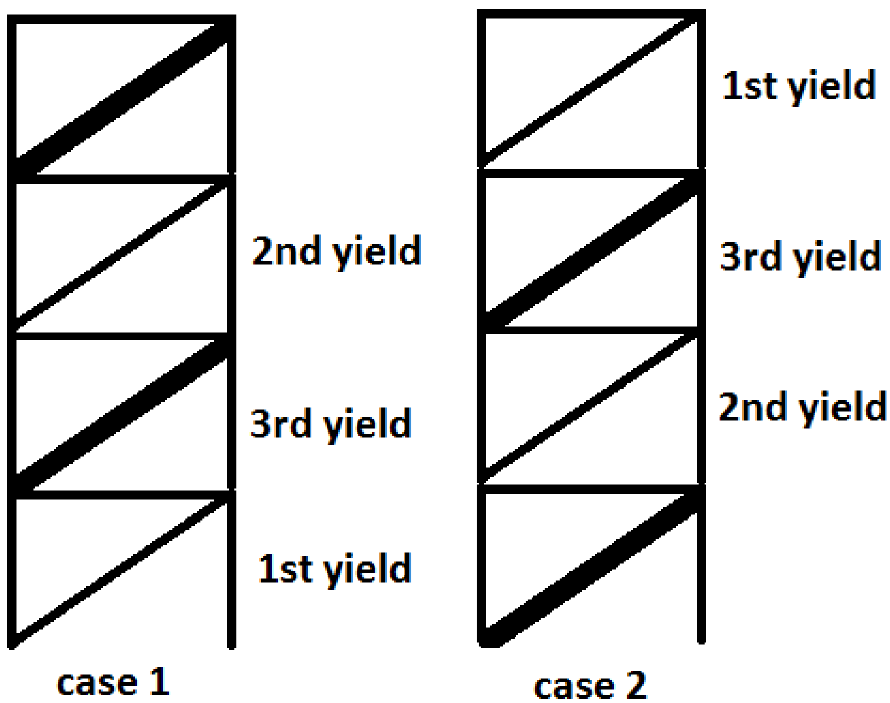


Figure 36: Two brace-overstrength configurations and yielding sequence as noted by order

Furthermore, the reduction of brace homogeneity generates new risk for columns design. There is no guarantee that non-dissipative members do not fail before the frame LS limit is reached. Therefore, this study take further step to include the parameter of Ω-overstrength ratio included in the column design, into consideration.

$$N_{\text{pl,Rd}}(M_{\text{Ed}}) \geq N_{\text{Ed,G}} + 1{,}1\gamma_{\text{ov}}\ \Omega . N_{\text{Ed,E}}$$

Instead of maintaining Ω as the minimum overstrength ratio for all braces, the average and maximum values are evaluated to gain insight into column responses in each case. Related back to literature review in section 2, this can help prevent “soft storey” mechanism created by overstrength non-homogeneity amongst floors.

case 1	25%					35%					case 1 - 50%				
Ω value	q	OS	am/ag	Ton	col yield*	q	OS	am/ag	ton	col yield*	q	OS	am/ag	ton	col yield*
min						2.16	1.22	2.65	1.48	Y	2.14	1.18	2.54	1.51	Y
average	3.99	1.02	4.17	1.5	N	2.84	1.15	3.3	1.7	N	2.71	1.21	3.29	1.75	Y
max						2.84	1.16	3.3	1.77	N	2.9	1.17	3.42	1.97	N
case 2	case 2 - 25%					case 2 - 35%					case 2 - 50%				
Ω value	q	OS	am/ag	Ton	col yield*	q	OS	am/ag	ton	col yield*	q	OS	am/ag	ton	col yield*
min						1.75	1.16	2.03	1.55	Y	1.74	1.25	2.18	1.61	Y
average	3.99	1.02	4.17	1.5	N	1.77	1.1	1.94	1.74	N	1.79	1.08	1.93	1.77	Y
max						1.68	1.17	1.97	1.81	N	1.72	1.09	1.88	1.95	N

Table 12: case study summary (DCM-DLS-0.25ag)

*Column yield denote whether the column has yielded before the frame LS limit is reached (Y=Yes N=No). "Yes" cases are highlighted in red.

Main observations are:

- Q factor drops in both case, when the homogeneity rule is relaxed, with case 2 dropping more than case 1
- OS value roughly stay the same throughout
- The column yielding possibility follows the pattern, with cases on top right of the table more likely to have yielding than bottom left. In other words, it's least conservative to design min Ω with 50% homogeneity limit.

The result approves the prediction that relaxation in overstrength homogeneity lead to the drop in q. It also suggests that the frame should be designed in such the way that the lowest overstrength factor should be assigned for the bottom brace (like in case 1) - the brace that has dominant capacity to dissipate seismic energy through deformation beyond yielding. This is also a common practice in reality, where the problem mostly lies in finding the small enough section for the top brace to satisfy the homogeneity requirement.

To help throw some light into ductility performance on both cases, the table below presents the number of braces actually exceeding yield point when the frame reach LS limit, with the yielding sequence flowing the order shown in above figure .

Number of yielded braces at LS limit		25%	35%	50%
Case 1	Min		3	2
	Average	4	3	3
	Max		3	3
Case 2	Min		2	1
	Average	4	2	2
	Max		2	2

Table 13: Number of brace exceeding yielding when the frame reach LS limit

Overall, case 1 configuration induce more braces to be on yielding plateau then case 2, which obviously result in higher ductility. Furthermore, it is interesting to notice that in case 1, the increasing stiffness of column by redefining Ω value strongly benefits the q factor (from 2.14 to 2.9), while in case 2, the effect is detrimental, but in the slight manner (from 1.75 to 1.68).

To summarize, the brace homogeneity case study provide two insights into design guidelines. Firstly, the brace overstrength distribution along the frame should be such that the bottom part gives way to yielding before the top part. Secondly, it is possible to relax the homogeneity limit up to 50%. But to conservatively avoid the yielding of column in global scheme before the structure reaches LS limit as well as the “soft storey” phenomenon, it is recommended to use Ω as the maximum, instead of minimum, overstrength of all dissipative members.

The possible shortcoming of this case study actually lies in its own assumption. In reality, when braces from certain floors reach yielding, the base shear distribution on each floors may not be triangular anymore, the phenomena that is not possible to precisely quantified and incorporate into the model. Therefore, the column yielding that occurs in the model may or may not happen under dynamic loads, and the suggestion about using Ω as the maximum is, in that sense, conservative.

5.2 Column design

Following the suggestion of the first parametric study, the relaxation of homogeneity limit should accompany with redefinition of the Ω value as maximum, instead of minimum, of all braces overstrength. Realizing that the suggestion is too conservative and too redundant in term of tonnage, one conceivable solution is designing each column with Ω as the corresponding brace overstrength of the same level.

Unfortunately, this idea cannot be implemented with pushover analysis. The reason is the exact shortcoming that is discussed in the previous case study. When one more braces are yielding, the system distribution of stiffness changes, which in turn alters the triangular force distribution. Further pushing of the structure does not bring the same proportional load increase to the levels at which the braces yield and those which braces are yet to yield. However, the load increment in FinelG keeps proportional for all levels of the frame as they are defined in the input file. Hence, at the level of yielding braces when braces can longer take higher load, the unduly excessive load increments are taken by columns only, which are designed with Ω as the corresponding brace overstrength. And because those braces are amongst the first to fail, their Ω value is also small, making the columns weaker. This understandably causes premature yielding of the columns on the same floor with weak braces.

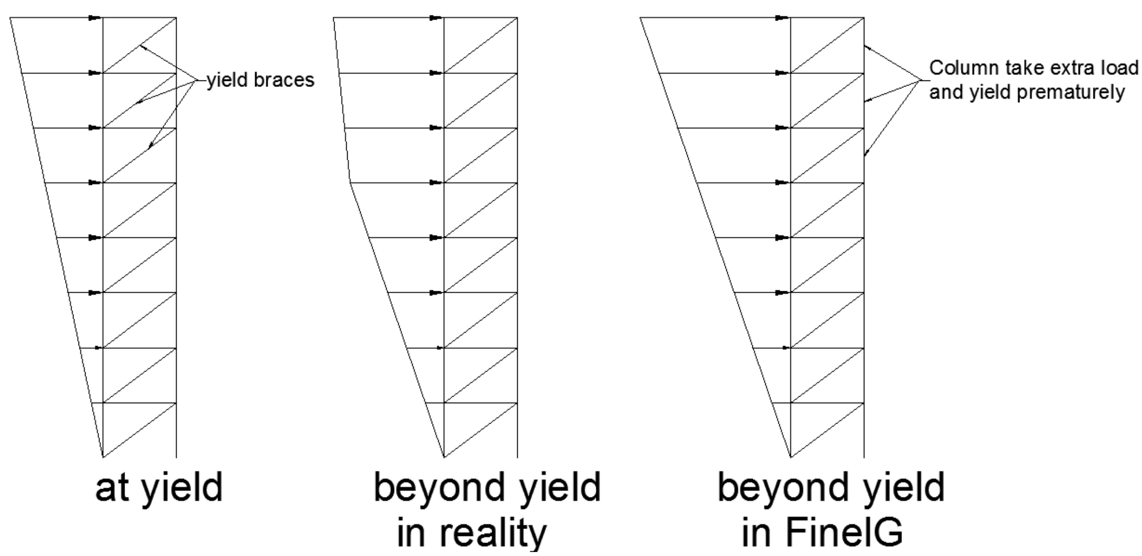


Figure 37: Load distribution beyond brace yielding in reality and in FinelG model

However, though it's outside the scope of this research, it is suggested that detailed dynamic analysis with incremental seismic action is carried out to investigate this possibility. In that case, the load distribution will shift according to the stiffness distribution, and the shortcoming of over analysis type is eliminated. And for each seismic input, base shear force and max top displacement can be extracted, and combined to plot out the behaviour until failure. Based on that, further interpretation of ductility, safety as well as feasibility of this new rule for column design can be determined.

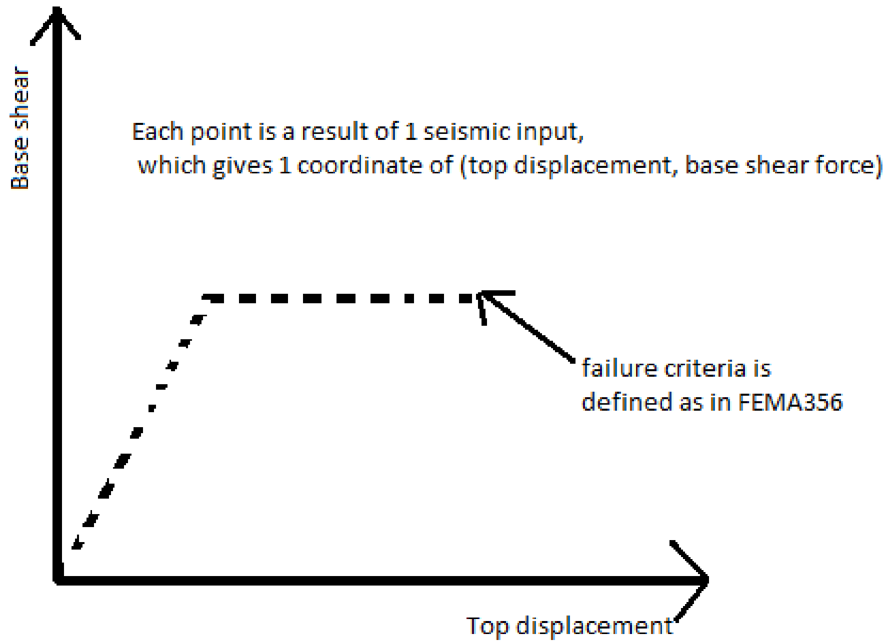


Figure 38: Predicted result of dynamic analysis with incremental seismic input

5.3 Homogeneity rules release

There is the recognized difficulty to impose homogeneity rule for the top part of the frame, since the application of which result in extremely small cross sections that breach the slenderness ratio limit. The designer can choose to uniformly increase the overstrength of all braces, thus the overstrength of the whole frame. Otherwise, it is desirable limit the scope of homogeneity to only the bottom part of the structure, leaving the top braces free in overstrength variation. And the "top braces" may possibly mean the top one-fourth or half of all braces, both of which will be investigated.

In this case study, 3 frames of 4/8/12 floors (1 each) from general case study of DCM-DLS design requirement under seismic load of $0.2a_g$ will be used for the experiment. In this case, the "top braces" are intentionally rendered over-strengthened up to 35% more than of the weakest brace.

top braces defined as	DCM-DLS-$0.2a_g$								
	4 floor			8 floor			12 floor		
	q	OS	am/ag	q	OS	am/ag	q	OS	am/ag
none	3.99	1.02	4.17	2.67	2	5.1	2.24	2.64	5.95
one-fourth	2.8	1.05	2.94	2.38	1.9	4.52	1.83	2.6	4.76
half	2.24	1.07	2.397	1.9	2	3.8	1.5	2.7	4.05

Table 14: case study summary

The result exhibits a clear trend of ductility reduction with the higher number of top braces over-strengthened. With no significance impact on OS value, the safety factor (a_m/a_g) follow the same trend as q value. The overstrength configuration in these cases also conform with the recommendation of previous parametric study, when the bottom part is weaker and will yield first, allowing high ductility performance. They are indeed the extension from case 1 of the previous case study- brace overstrength limit, with different configuration.

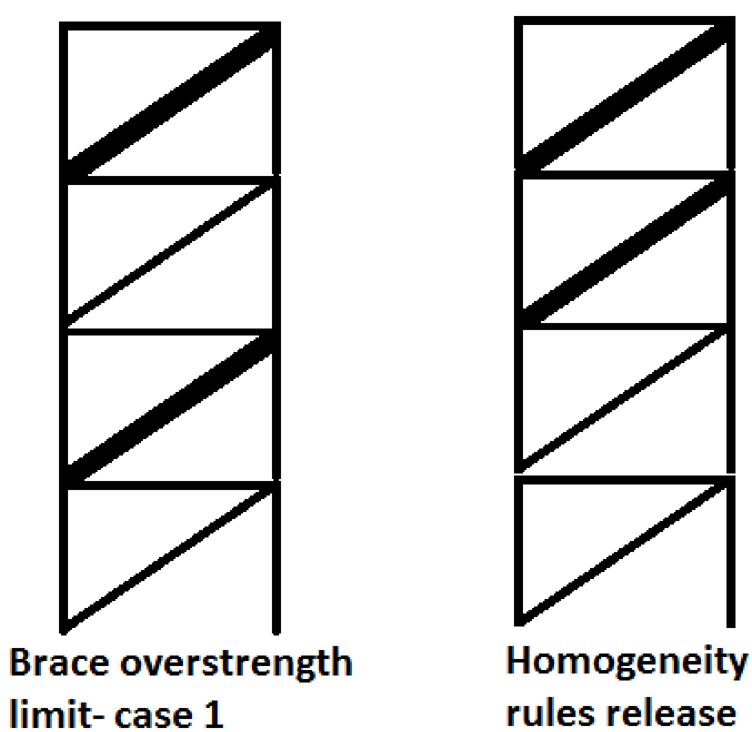


Figure 39: Comparison between 2 case studies, both has the weakest brace at the bottom floor

	q value	OS factor	am/ag
Brace overstrength limit case 1	2.16	1.22	2.65
Homogeneity rules release	2.24	1.07	2.397

Table 15: comparable result for both cases

Furthermore, the magnitude of the top brace overstrength should also be considered to make generalization on the alteration of homogeneity limit. As 35% is already tried out, 70% is tested to see if the magnitude has any effect on ductility and overstrength factor.

Homogeneity limit	DCM-DLS-0.2a_g								
	4 floor			8 floor			12 floor		
	q	OS	am/ag	q	OS	am/ag	q	OS	am/ag
Half braces-35%	2.24	1.07	2.397	1.9	2	3.8	1.5	2.7	4.05
Half braces70%	2.34	1.03	2.43	1.8	2.04	3.66	1.5	2.67	4.01

Table 16: Role of overstrength magnitude on frame seismic performance

Evidently, there is no significant change observed in the frame performance indicators. This phenomenon can only be explained that the displacement at yield and LS limit is dominantly due to bottom brace displacement. The top braces, those that hardly yield till the end, contribute little to overall drift.

However, it is also noted that, as the magnitude of ratio between max and min overstrength jumps without changing the column design, the column section at mid height, or the intersection between “bottom braces” and “top braces”, may fail before the frame reach LS limit. This is clearly observed in output file.

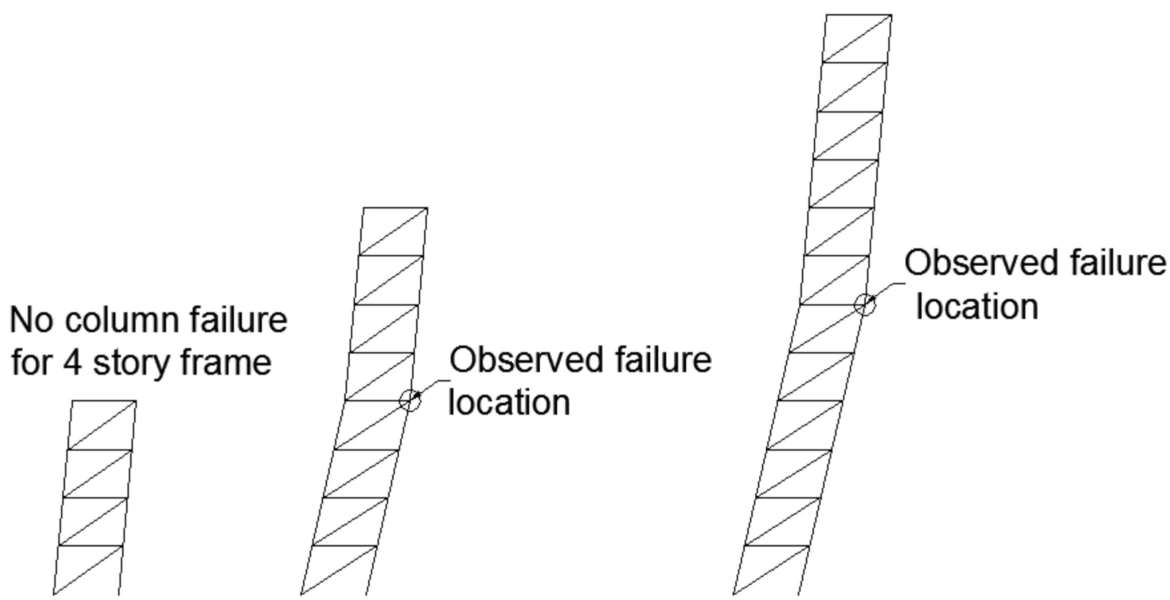


Figure 40: Frame behavior with column failure when Wmax/Wmin=1.7

One possible solution to this problem, as suggested before, is to redefine Ω value in non-dissipative member design as a maximum instead of minimum brace overstrength. The limitation is recognized though that it is conservative, as probably average or three-percentile value of Ω is adequate to guarantee no failure in columns; but the pushover type of analysis allow no possiblity to quantify such information.

To conclude, this experiment illustrates one possibility to fine-tune the design guideline in reality that address economical issue while optimize the safety factor if needed. Designers can increase top braces overstrength to meet the slenderness limit without increasing all other braces and consequently, other non-dissipative members. Referring to general case study, It is particularly applicable to soft building (12/8 story frames), where the load on top are generally too small to have appropriate cross section. The effect of this, of course, is the reduction of safety factor, which may not be the issue for those soft structures with excessive value of a_m/a_g.

5.4 Structural full scale pushover

It is fully established that in CBF, brace strength have dominant contribution to the overall frame performance in general, stiffness and ductility in particular. The columns are there mainly to resist axial loads induced by the pushover forces, which has impact on the frame ductility as explained in the section 4.2.2.1. However, the contribution of column flexural stiffness on the structure capacity to deform before yielding is largely unknown. This section aims to conduct a full scale model test on some typical cases to understand the column flexural stiffness contribution to ductility.

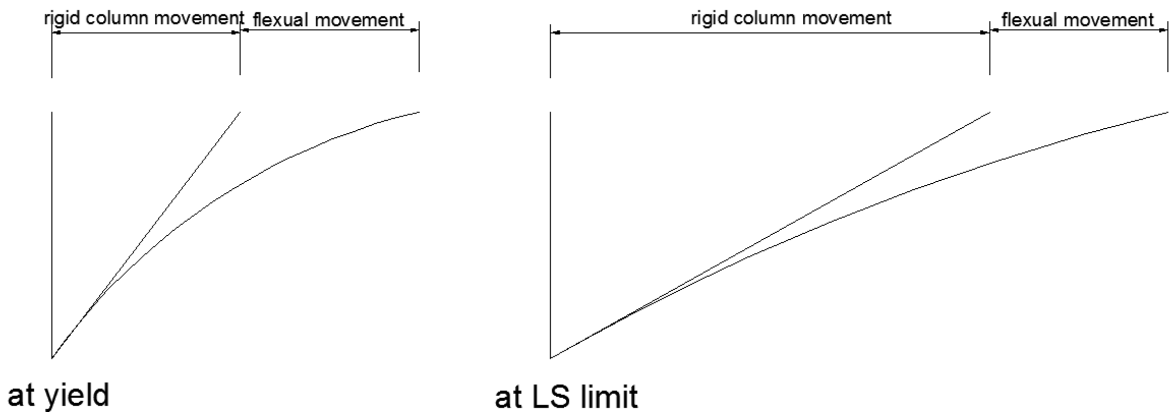


Figure 41: Total displacement contribution from rigid and flexural movement of the column

In cases when all the braces yield and reach LS limit in parallel, the flexural displacement of the column stops changing after the brace yield. Only rigid column movement take place then. With x as the rigid column movement, y as flexural displacement at yield; n*x as rigid column movement at LS limit, we have:

$$q = \frac{nx + y}{x + y} \qquad \lim_{x\to\infty} q = n \quad and \quad \lim_{y\to\infty} q = 1$$

Therefore, the higher the flexural movement is, the less ductility the frame exhibits. If the flexural stiffness of columns is somehow strengthened, which leads to reduction of flexural drift, the ductility is expected to increase according to the formula.

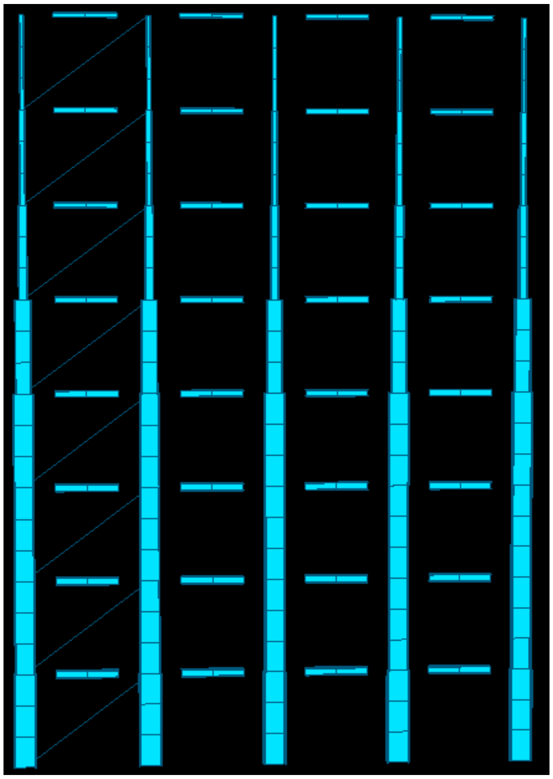


Figure 42: full scale model of building frame with 4 spans

In the full scale model, only the first two columns on the right are connected with braces and subject to axial loads. Therefore, the presence of extra column doesn't contribute axial stiffness of column members. However, as all the columns are laterally connected by beams, flexural stiffness is strengthened significantly. The predicted effect is, as stated previously, an increase in ductility of the frame under pushover action. Referring to general case study, 4 cases of 8 story frames under DCM-DLS design requirement with 0.1/0.15/0.2/0.25a_g seismic action will be considered. The comparison between results of simplified model, which is done previously, and full scale one is shown in the below table.

8 floor		0.1			0.15			0.2			0.25		
		q	OS	am/ag	q	OS	am/ag	q	OS	am/ag	q	OS	am/ag
DCM-DLS	simplified	3.24	2.4	8.21	2.66	2.1	5.8	2.67	2	5.1	2.6	1.8	4.75
	full scale	3.42	2.53	8.653	2.46	2.19	5.39	2.75	1.9	5.23	2.61	1.83	4.7763

Table 17: case study summary

It is obvious from the result that there is minimal change in q value and OS factor when the frames are tested out in full scale, which effectively means no improvement in safety factor. This simply proves the flexural displacement is minimal as compared to rigid column movement at yield point, and therefore at LS limits state. The inclusion of extra columns, present in the structure but not directly connected to braces member or relevant in resisting axial loads induce by earthquake, are unnecessary. This conclusion helps limit the adverse effect of structure simplification in this research, and also proves the reliability of the obtained result.

5.5 Slenderness limit

To resolve the unavoidable overstrength on the top part, the most conceivable way would be reconsidering the slenderness limit, currently set as 2. The requirement is there to help prevent excessive out of plane vibration of the brace, which create undue load on other direction of the frame. Also, there is a "whipping" effect generated by contribution of the frame second mode shape, which is not taken into account in frequency and load prediction. This effect is exaggerated for top braces with slenderness higher than 2.

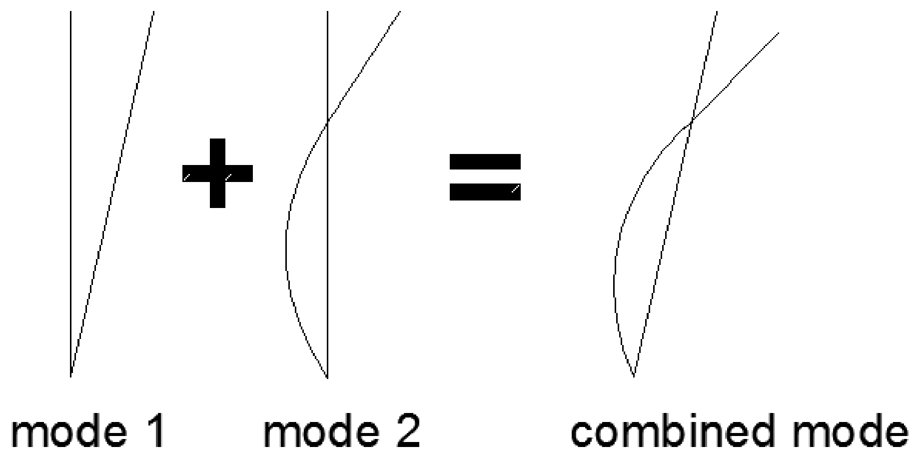


Figure 43: Illustration on how mode 2 contribution can increase the top displacement

This case study tries to detect the effect of slenderness limit relaxation, assuming that the problem of excessive out of plan vibration is to be prevented by other means (prestress, lateral brace). In general case study set, many 12 floor frames under DCM-ULS design condition have this problem of unavoidable overstrength due to slenderness limit, and are chosen to relax the rule. The comparison is made between target displacement d_t calculated by Annex B of EC8 and real displacement rendered by dynamic analysis with FinelG.

In dynamic analysis, the frame is basically subject to accelerogram with predetermined attribute of earthquake type, soil type,duration, damping; all of which is explained in section 2.6. To ease out the randomness of accellogram generated from Gosca, the dynamic analysis displacement should be either the maximum of any three or mean of seven independent analyses. In this case study, the maximum of three is to be performed.

DCM-ULS 12 floors	Period of mode 1	Period of mode 2	Top displacement from 3 analyses (m)
$0.1a_g$	4.15	1.23	0.1263 0.1042 0.1224
$0.15a_g$	3.5	1.17	0.2013 0.1989 0.173
$0.2a_g$	3.14	1.05	0.2577 0.222 0.218
$0.25a_g$	2.875	0.95	0.2661 0.2712 0.221

Table 18: Analysis result

	Number of top braces exceeding $\lambda = 2$ (counting from top)	Max l	Target displacement d_t	Real displacement from dynamic analysis (max of three)
$0.1a_g$	6	6.03	0.11	0.1263
$0.15a_g$	3	4.75	0.17	0.2013
$0.2a_g$	2	3.7	0.22	0.2577
$0.25a_g$	2	3.12	0.28	0.2712

Table 19: d_t and $d_{dynamic}$ comparison

The analysis reveals that generally, the real displacement that the seismic load imposes on the frame is indeed higher than the predicted displacement by Annex B of EC8.

6 Design guideline alternative

Reflecting on the original proposal of carving the intermediate way for low-medium seismic design, this section devote to investigate the effect of reducing the assumed q factor combined with several design guideline relaxations described and tested out in the previous section-parametric study, through which an evaluation of such new design method can be made.

Hypothetically, let us take the design case of DCM-DLS 0.1a_g. Assuming that the original q factor of 4 is reduced to 2, the effect is equivalent to increasing the seismic action from 0.1 to 0.2. Therefore it is considered that the frame that resist 0.2a_g with q=4 is adequate for the frame 0.1a_g with q=2.

DCM-DLS 0.1a_g	q	OS	am/ag	Ton/floor
q=4	3.92	1.27	5.01	0.8375
q=2	3.99	1.02x2=2.04	4.17x2=8.34	1.4975

Table 20: New design with lower q value

It is noted that for the OS factor, the value will double because target displacement now corresponds to 0.1a_g, half of the original 0.2a_g.

6.1 Brace overstrength limit

The relaxation of the ratio $\Omega_{max}/\Omega_{min}$ limit from 25% to 35% and 50% bring a drop to q value and keep OS relatively unchanged, as shown in the parametric study. This obviously leads to safety factor reduction. That effect is just complementary of reducing q value, where q value is kept unchanged and OS factor jumps up. Combining such relaxation with lowering q ductility may even out the combined effect, as hypothetically shown in the below table, take case 1 (cases where over-strengthened braces are placed on second and fourth floor in 4 story frames) as an example.

DCM DLS 0.1a_g	case 1	25%					35%					case 1 - 50%				
	Ω value	q	OS	am/ag	Ton	Col fail	q	OS	am/ag	ton	Col fail	q	OS	am/ag	ton	Col fail
q=2	min						2.16	2.04	4.4	1.48	Y	2.14	2.04	4.37	1.51	Y
	average	3.99	2.04	8.34	1.5	N	2.84	2.04	5.8	1.7	N	2.71	2.04	5.53	1.75	Y
	max						2.84	2.04	5.8	1.77	N	2.9	2.04	5.92	1.97	N
	average	3.99	2.04	8.34	1.5		2.61	2.04	5.33	1.65		2.58	2.04	5.27	1.74	

Table 21: Combined effect of brace overstrength limit relaxation and lowering q ductility, assuming OS kept unchanged

The table shows how the structural performance indicators change with the combined effect of two design guideline alterations. There is no significant change in overall safety factor (from 5.01 to 5.33/5.27), because the drop in q and jump in OS roughly cancel each other.

DCM DLS 0.1a_g	q=4	q=2, homogeneity limit 35%	q=2, homogeneity limit 50%
Safety factor	5.01	5.33	5.27

Table 22: Safety factor comparision

Even though the effect is shown for only 1 case, the trend of variation can still be predicted for other cases. As the q value and OS factor change in the opposite direction, in this case relatively with the same magnitude, we expect the ratio of am/ag still stay in the same range. The relaxation of homogeneity rule is simply compensated by higher tonnage of the structure due to lower load reduction factor q. Therefore, such design guidelines amendment is possible and safe in reality

6.2 Homogeneity rules release

Similarly, limiting the scope of homogeneity rules to only a bottom portion of the structure will bring down the q values. Combining that with lower load reduction factor q is expected to have the same effect.

DCM DLS $0.1a_g$	4 floor			8 floor			12 floors		
	q	OS	am/ag	q	OS	am/ag	q	OS	am/ag
q=4	3.92	1.27	5.01	3.24	2.4	8.21	1.94	4.57	8.85
q=2	3.99	1.02x2=2.04	4.17x2=8.34	2.67	2x2=4	5.1x2=10.2	2.24	3.52x2=7.04	7.88x2=15.76

Table 23: New design with lower q value

Taking into account the fact that the adjustment of homogeneity rules plummets the q value while keeping OS factor unchanged, the table below shows the possible combined effect when combining it with lowering q value in design.

top braces defined as	DCM-DLS-$0.1a_g$ q=2								
	4 floor			8 floor			12 floor		
	q	OS	am/ag	q	OS	am/ag	q	OS	am/ag
none	3.99	2.04	8.34	2.67	2	10.2	2.24	2.64	5.95
one-fourth	2.8	1.05x2=2.1	2.94x2=5.88	2.38	1.9x2=3.8	4.52x2=9.04	1.83	2.6x2=5.2	4.76x2=9.52
half	2.24	1.07x2=2.14	2.397x2=4.8	1.9	2x2=4	3.8x2=7.6	1.5	2.7x2=5.4	4.05x2=8.1
Average of one-fourth and half	2.52	2.12	5.34	2.14	3.9	8.32	1.67	5.3	8.81

Table 24: Combined effect of homogeneity rule relaxation and lowering load reduction q value

Comparing the overall safety factor of original and altered design, the structure remains equally safe. Again, the ease of design is compensated by higher frame weight.

am/ag	4 floor	8 floor	12 floor
q=4	5.01	8.21	8.85
q=2+homogeneity rule release	5.34	8.32	8.81

Table 25: safety factor comparison

Also, based on the supplementary study done in section 4.3, it is also evident that the magnitude of top braces overstrength, as long as they are over 25%, has no effect on seismic performance of frame. Therefore, it can be concluded that the relaxation of homogeneity rule as above, combined with reducing q value, is an acceptable alternative of CBF design.

7 Conclusion

In this study, a systematic procedure of how a CBF is designed strictly according to Eurocode requirements is presented, and accordingly executed for 48 cases in general case study.

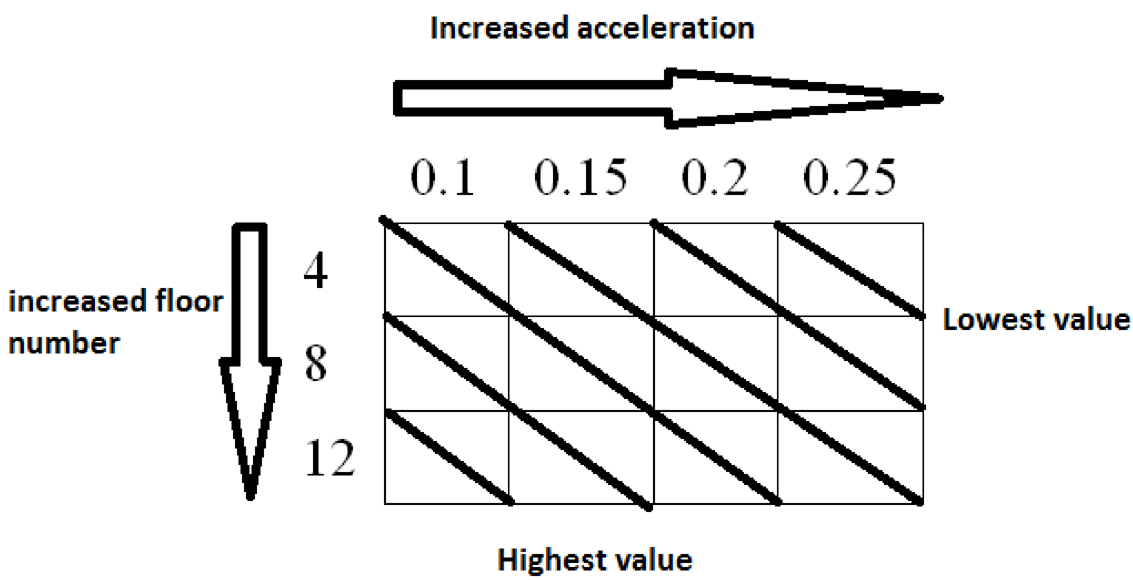
In the above-mentioned case study, structures are of varying frame height, seismic load, design methodologies and limit state, therefore exhibit different performance properties such as ductility, overstrength factor and overall safety factor. Still, there is a clear trend of variation for each performance indicator, which is summarised in the following table.

	Height	Seismic load	ULS =>DLS	DCM =>DCL
q	(-) Brace non uniform yielding	(0)	(0)	(-) Brace non uniform yielding + Column failure with low ductility
OS	(+) Eurocode increased conservativeness with height	(-) Yield displacement Is fixed for each floor height	(+) Increase in stiffness lead to reduction in target displacement	(+) Increase in stiffness lead to reduction in target displacement

(-) negatively correlated (+) positively correlated (0) neutral

Table 26: General case study trend summary

Safety factor is simply the product of q and OS, and follow the trend as shown below.



The general case study, in the mean times, reveals some design feasibility and constructability issue. Firstly, the homogeneity rule imposes strict limit on the ratio of $\Omega_{max}/\Omega_{min}$, which can prohibit sensible design solution due to section unavailability. Secondly, the slenderness limit λ<2 renders many top braces with low imposed load inevitably over-designed. Combined with homogeneity rules, it can cause global over-designed of whole structure, dissipative and non-dissipative members alike. Lastly, the simplification of the structural model, by eliminating unconnected non-dissipative members from pushover analysis, stands a risk of inaccurate prediction of the pushover curve due to unknown contribution from those remote members.

The parametric case study is carried out to individually investigate the addressed issue can is summarized in the following table.

Increase limit of $\Omega_{max}/\Omega_{min}$ from 25% to 35/50%	The frame **ductility is reduced**, depending on overstrength distribution, while the **OS factor largely unchanged**. To ensure the columns not to yield before LS limit, redefining value of **Ω in column design as max** instead of min is conservatively recommended. It is also advised **not to put overstrength braces on the first floor** of frames.
New column design rule, using Ω brace overstrength on the same floor	**The idea is logical** but **not feasible with pushover analysis**, because the type of analysis unalterably has same proportional load distribution on floors, while this distribution proportion may change when some braces yields before others.
Relax the homogeneity rule on some top braces	Similarly, frame **ductility is reduced**, when **OS factor unchanged**. The more top braces are outside 25% limit, the more q value drops. However, the **magnitude of overstrength** on top, as long as bigger than 25%, does **not affect the frame performance**. Column premature failure can be prevented by redefining value of Ω
Include all element in the design	A complete structure is modelled for pushover, which **increases flexural**, not axial, **stiffness of columns**. The result show **no significant change** in frame performance, indicating **little contribution from column flexural deflection** to overall displacement.
Slenderness limit relaxation	Assuming that the vibration problem of slender braces is taken care by other ways, **the inclusion of such slender braces** in structure increases the 2 mode contribution and leads to **higher displacement than predicted** by EC8 part 1 Annex B.

Table 27: Parametric case study summary

Further study also shows that there are 2 possible alternatives to design CBF frame that may be found easier to implement in real design case.

- Reduce the load reduction factor q from 4 to 2, and relax the homogeneity rule limit from 25% to 50%
- Reduce the load reduction factor q from 4 to 2, and relax the homogeneity rule limit from half of the top braces, allowing them to be over-designed above 25% limit.

ANNEX

This section provides detailed pushover curve attributes for all frames in general and parametric case study

4		0.1				0.15				0.2				0.25			
attributes		F_b (kN)	D_y (mm)	D_m (mm)	T* (s)	F_b (kN)	D_y (mm)	D_m (mm)	T* (s)	F_b (kN)	D_y (mm)	D_m (mm)	T* (s)	F_b (kN)	D_y (mm)	D_m (mm)	T* (s)
dcm	uls	787.6	85	333	1.33	787.6	85	333	1.33	1033.7	88	36.3	1.17	1281	94	317	1.09
	dls	787.6	85	333	1.33	1330	94	358	1.07	1846	97	385	0.92	2405	100	368	0.82
dcl	uls / dls	1374	106	245	1.12	2064	115	397	0.95	2762	117	156	0.82	3429	121	325	0.75
8		0.1				0.15				0.2				0.25			
dcm	uls	1521	246	678	2.13	1521	246	678	2.13	1613	254	689	2.1	2016	262	637	1.91
	dls	2398	217	704	1.6	3129	278	742	1.58	4322	280	751	1.33	5636	304	780	1.21
dcl	uls	2186	298	828	1.91	3264	346	477	1.68	4398	317	436	1.39	5500	346	564	1.3
	dls	3263	346	477	1.68	4869	350	474	1.38	6606	337	470	1.16	8578	347	822	1.04
12		0.1				0.15				0.2				0.25			
dcm	uls	2062	513	994	2.16	2062	513	994	2.16	2062	513	994	2.16	2062	513	994	2.16
	dls	2062	513	994	2.16	2990	546	1271	1.85	4881	597	1346	1.5	5702	603	1185	1.4
dcl	uls	2055	630	852	2.37	2055	630	852	2.37	2386	637	1131	2.22	3023	662	877	2.01
	dls	2501	664	1031	2.2	3471	678	1031	1.89	4882	707	1400	1.62	6166	715	802	1.45

Table 28:General case study curve attributes (note that 12 floor cases has their load divided by half)

F_b-base shear force in kN

D_y- yield displacement in mm

D_m- LS limit displacement in mm

T*-period of frames converted to SDOF

case 1	25%				35%				case 1 - 50%			
Ω value	F_b (kN)	D_y (mm)	D_m (mm)	T* (s)	F_b (kN)	D_y (mm)	D_m (mm)	T* (s)	F_b (kN)	D_y (mm)	D_m (mm)	T* (s)
min	1846	97	385	0.92	2112	115	250	0.95	2265	103	221	0.88
average					2115	105	297	0.9	2247	108	293	0.89
max					2130	105	298	0.9	2265	102	296	0.87
case 2	case 2 - 25%				case 2 - 35%				case 2 - 50%			
min	1846	97	385	0.92	2069	106	185	0.9	2198	114	199	0.91
average					2022	96	171	0.87	2024	94	167	0.85
max					2147	105	176	0.88	2083	94	161	0.85

Table 29: Parametric study- brace overstrength limit cases (4 floor-DCM-DLS-0.2ag)

top braces defined as	DCM-DLS-0.2a_g											
	4 floor				8 floor				12 floor			
	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)
none	1846	97	385	0.92	4322	280	751	1.33	4881	597	1346	1.5
one-fourth	1875	96	271	0.92	4345	277	660	1.32	4918	593	1087	2.13
half	1921	98	220	0.92	4537	290	554	1.33	5064	612	962	2.13

Table 30: Parametric study- homogeneity rule relaxation (12 floor cases has their load divided by half)

top braces defined as	DCM-DLS-0.2a_g											
	4 floor				8 floor				12 floor			
	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)
half-35%	1921	98	220	0.92	4537	290	554	1.33	5064	612	962	2.13
half-70%	1927	93	218	0.9	4644	293	5251	1.33	5092	598	899	2.11

Table 31: Parametric study- homogeneity rule relaxation-supplementary(12 floor cases has their load divided by half)

8 floor		0.1				0.15				0.2				0.25			
		F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)
DCM-DLS	simplified	2398	217	704	1.6	3129	278	742	1.58	4322	280	751	1.33	5636	304	780	1.21
	full scale	2399	216	739	1.59	3111	282	694	1.6	4322	277	761	1.32	5637	301	788	1.2

Table 32: Parametric study- full scale structure pushover curve

12 floor	0.1				0.15				0.2				0.25			
	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)	F_b (kN)	D_y(mm)	D_m (mm)	T* (s)
DCM-ULS	454	380	936	4.02	681	416	741	3.4	905	440	1001	3.03	1109	453	1012	2.78

Table 33: Parametric study- 12 floor cases design with no slenderness limit

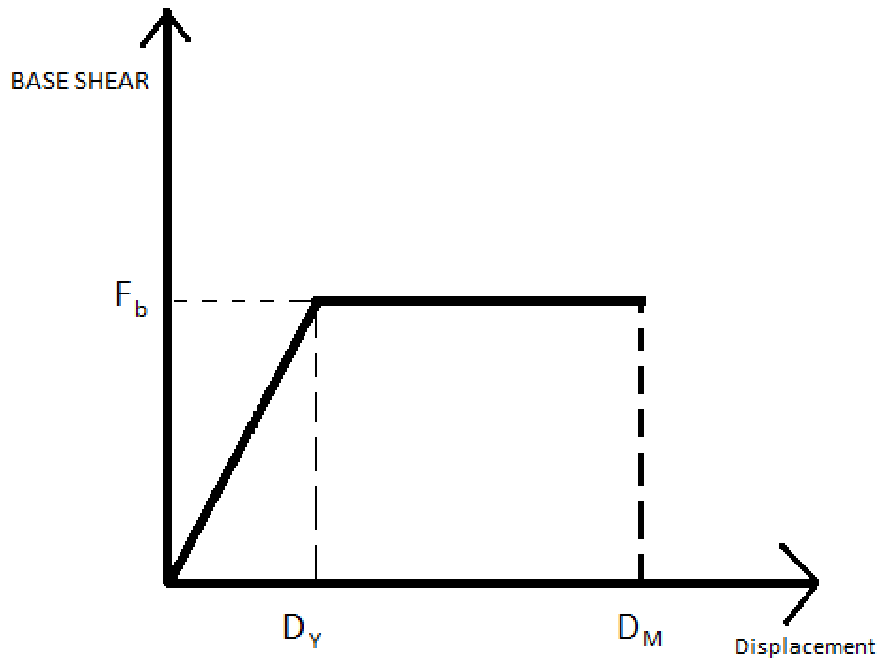


Figure 44:Pushover curve attributes

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