Direct Displacement Based Seismic Design of Irregular CBFs

Suhaib Salawdeh[1]
[1]Department of Building and Civil Engineering, Galway-Mayo Institute of Technology, Dublin Road, Galway, Ireland
email: Suhaib.salawdeh@gmit.ie

ABSTRACT: Some types of structures can suffer from vertical irregularities due to aesthetic, functional, or economic reasons. A possible lateral force resisting solution for these types of structures is Concentrically Braced Frame (CBF) systems. These systems are gaining a lot of popularity as their design and fabrication are straight forward with low cost. Moreover, they are overcoming many limitations exist in other lateral force resisting systems.
In this paper, a direct displacement-based design (DDBD) procedure is developed for irregular multi-storey CBF structures. In this procedure, design displacements considered are decided upon the code and material drift limits, then the strength required to achieve this displacement is calculated and finally all structural elements are designed.
A case study of a twelve-storey CBF structure with vertical irregularity is designed using the developed DDBD procedure. The configuration of the vertical irregularity assessed is in the form of setbacks up the vertical axis of the building where the frames have more bays at the base of the building than at the top.
Non-linear time history analysis (NLTHA) using 7 different accelerograms with displacement response spectra matching the design displacement spectrum are used to record the behaviour of the irregular CBF structure when subjected to real earthquakes. It is found that the design displacements and storey drifts from the DDBD procedure for the case study with vertical irregularity matched relatively well with the displacements and storey drifts recorded through the NLTHA analyses and a new DDBD procedure for CBFs with vertical irregularity is validated.

KEY WORDS: Concentrically Braced Frames; Vertical irregularity; Direct Displacement-Based Design; NLTHA.

1 INTRODUCTION

For economic, functional and aesthetic reasons, many structures are built with vertical irregular configurations. Extensive research has been conducted to study the effects of vertical discontinuities in the distributions of strengths, stiffnesses and masses on the structural behaviour under seismic loads [1-10].

European seismic design code, Eurocode 8 [11] has defined the irregular configuration of buildings explicitly. For vertical irregularity, it has listed a criteria in which the building should satisfy to be categorised as regular in elevation. The criteria includes stiffness, strength and mass irregularities, discontinuity of the lateral load resisting systems, such as cores, structural walls, or frames and the presence of setbacks. If the building is categorised as irregular, a complex dynamic analysis procedure must be used instead of the equivalent lateral force method used in forced based design procedure.

In this paper, a simple direct displacement-based design (DDBD) methodology will be proposed to design CBF structures with vertical irregularity associated with setbacks and irregularity of mass distribution in the vertical plane.

In previous research [12-14] a performance-based design procedure for regular concentrically braced steel frame structures has been validated. To do so, full scale shake table tests [15-17] were performed to develop robust numerical models to simulate the behaviour of CBF structures under seismic loads [18-20].

The validity of the direct displacement-based design (DDBD) methodology will be assessed to design CBF structures with vertical irregularity. To do so, a case study CBF structure with vertical irregularity will be designed using the developed DDBD procedure. The structure will be designed to achieve a target displacement profile related to limit material strains or design drift limits specified in the seismic codes. The seismic energy produced by earthquakes is permitted to be dissipated by brace elements through yielding in tension and buckling in compression. All other elements such as beams and columns will be designed to behave elastically and are not expected to yield to dissipate energy.

Then, non-linear time history analysis (NLTHA) using real earthquakes will be used to record the behaviour of the designed irregular structure and compare it with the proposed response.

2 DDBD PROCEDURE FOR IRREGULAR CBFS

The Direct Displacement-Based Design (DDBD) procedure proposed by Priestly [21] characterizes the multi degree of freedom (MDOF) structure to an equivalent single degree of freedom (SDOF) system. The following paragraphs describe the proposed DDBD procedure for irregular CBFs.

The equivalent SDOF design displacement, Δ_D, can be found from the design storey displacements of the CBF, Δ_{Di} .

$$\Delta_D = \frac{\sum m_i \Delta_{Di}^2}{\sum m_i \Delta_{Di}} \quad (1)$$

where m_i is the mass at storey i, associated with the storey displacement, Δ_{Di}.

The design storey displacements, Δ_{Di}, is found by:

$$\Delta_{Di} = \delta_i(\frac{\Delta_1}{\delta_1}) \quad (2)$$

where Δ_1 is the displacement of the first storey or critical storey and δ_1 is the normalised inelastic mode shape of the first storey or critical storey.

The inelastic mode shape, δ_i, for CBF structures can be found by:

For $n \leq 4$:
$$\delta_i = \frac{H_i}{H_n} \quad (3)$$

For $n > 4$:
$$\delta_i = \frac{4}{3}(\frac{H_i}{H_n})(1 - \frac{H_i}{4H_n}) \quad (4)$$

where n is number of storeys, i is the storey number, H_i, is the height of the ith storey and H_n is the total height of the structure. The equivalent SDOF yield displacement, Δy, can be found from the storey yield displacement:

$$\Delta_y = \frac{\sum m_i \Delta_{yi}^2}{\sum m_i \Delta_{yi}} \quad (5)$$

where Δ_{yi} is the yield displacement at the ith floor. This displacement which can be obtained from the following equation proposed by Della Corte and Mazzolani [22] and Wijesundara [23].

$$\Delta_{yi} = \sum_{j=1}^{i} (\frac{\varepsilon_{br,y}}{\sin\alpha\cos\alpha} h_j + \varepsilon_{col,y} h_j \tan\alpha) \quad (6)$$

where $\varepsilon_{br,y}$ is the brace axial strain, α is angle of the brace with the horizontal axis, h_j is the storey height and $\varepsilon_{col,y}$ is the column axial strain.

Knowing the equivalent SDOF design displacement, Δ_D and the equivalent SDOF yield displacement, Δy ,the design displacement ductility, μ, can be found by:

$$\mu = \frac{\Delta_D}{\Delta_y} \quad (7)$$

The effective mass of the SDOF structure, m_e, can be found by:

$$m_e = \frac{\sum m_i \Delta_{Di}}{\Delta_D} \quad (8)$$

and the effective height, He, can be found by:

$$H_e = \frac{\sum m_i \Delta_{Di} H_i}{\sum m_i \Delta_{Di}} \quad (9)$$

This Equivalent Viscous Damping, ξ , model for CBFs proposed by Wijesundara [23] and verified by Goggins and Salawdeh [24] can be used as a function of non-dimensional slenderness ratio, $\bar{\lambda}$, and ductility, μ, in the following equations:

$$\xi = 0.03 + \left(0.23 - \frac{\bar{\lambda}}{15}\right)(\mu - 1) \qquad \mu \leq 2 \quad (10)$$

$$\xi = 0.03 + \left(0.23 - \frac{\bar{\lambda}}{15}\right) \qquad \mu \geq 2 \quad (11)$$

At this point of the design procedure, all of the substitute structure characteristics required for DDBD design methodology for irregular CBFs have been established and as such, the design displacement spectrum can be developed at the design level of damping. This can be done by applying a damping reduction factor R_ξ to the elastic displacement spectrum. The equation of the damping modifier, R_ξ , specified in EC8 [25] will be used as the following

$$R_\xi = \left(\frac{0.07}{(0.02 + \xi)}\right)^{0.5} \quad (12)$$

From the design displacement spectrum established, the effective period, Te, can be found. And the effective stiffness, K_e, can be calculated as:

$$K_e = \frac{4\pi^2 m_e}{T_e^2} \quad (13)$$

Knowing the effective stiffness, K_e, and the design displacement, Δ_d, base shear, V_{base}, can be obtained by the following Equation

$$V_{base} = K_e \Delta_D + \frac{m_e g}{H_e} \Delta_D \quad (14)$$

The second term is added to account for P-delta effects in CBF structures [21], where g is acceleration due to gravity.

The base shear will be distributed to all floor levels using the following equation suggested Priestley et al. [21].

$$F_i = F_t + 0.9 V_{base} \frac{m_i \Delta_i}{\sum m_i \Delta_i} \quad (15)$$

where $F_t = 0.1V_{base}$ at the roof level and $F_t = 0$ at all other levels, , m_i and Δ_i are the mass and design displacement of the ith floor. In this equation the higher modes effect, is taken into account by allocating 10% of V_{base} to the roof level and distributing the remaining 90% of V_{base} to all floor levels including the roof level.

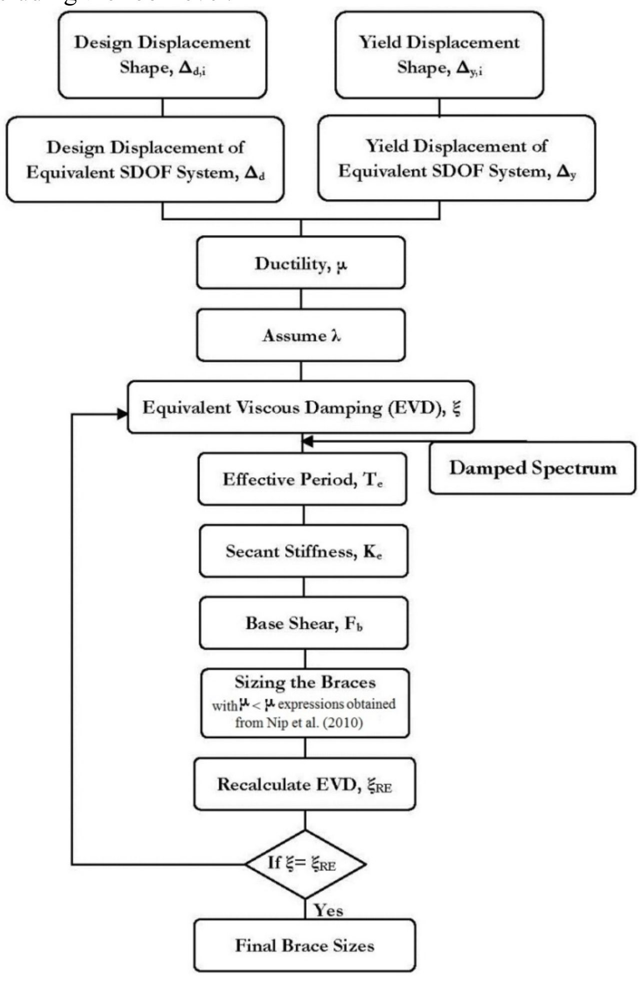

Figure 1. Flow chart of the DDBD procedure for CBFs, adapted from [13]

3 IREGULAR CBF CASE STUDY STRUCTURE DESIGNED USING DDBD PROCEDURE

A 12-storey CBF structure with vertical irregularity is designed using the DDBD. The irregularity is associated with setbacks up the vertical axis of the building where the frames have more bays at base of the structure than at the top. The structure dimension is 64X40 m in plan consisting of two CBFs with setbacks in the excitation direction with a uniform storey height of 3 m. The study of the response in the setback direction as the lateral resistant system is carried out in this paper as it is the scope of the study. The response of the transverse direction should be studied as part of a future research to take into account the torsional effects.

Plan view and elevation for the 12-storey structure are shown in Figure 2 and Figure 3.

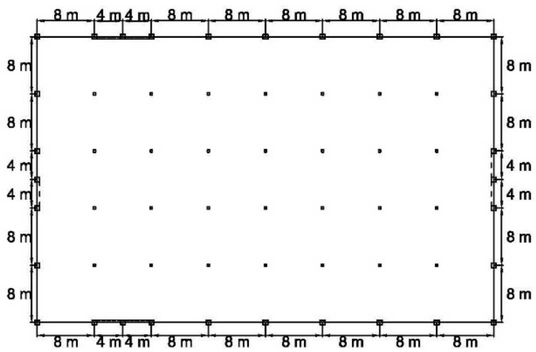

Figure 2. Plan view of the 12-storey structure associated with setbacks along the vertical axis of the building.

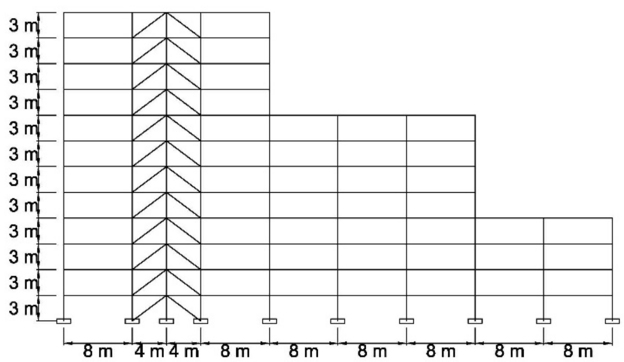

Figure 3. 12-story irregular CBF structure associated with setbacks on the frames at the fifth and ninth floor.

A pinned connection is used to connect columns and beams. The lateral seismic loads are assumed to be resisted by the braces only on the 4m bays represented in the dashed lines in Figure 2. Type 1 elastic response spectrum for soil type C from Eurocode 8 [11] is used. The peak ground acceleration (PGA) chosen for the design is 0.3g. In order to control damage of non-structural elements 2.5% design storey drift is chosen. Seismic load (1.0 Dead load + 0.3 Live load) of 8.75 KPa was selected using actions explained in provisions of EC1 [26]. The mass was different in every four floors, 896 tonnes for each of the floors from 1st to 4th floor, 700 tonnes for each of the floors from 5th to 8th floor and 420 tonnes for each of the floors from 9th to 12th floor.

Design storey displacements, Δ_{Di}, and yield storey displacements, Δ_{yi}, are found using Equations (2) and (6). The data for calculating Δ_{Di}, and, Δ_{yi}, are shown in Table 1.

Table 1. Design storey displacements, Δ_{Di}, and yield storey displacements, Δ_{yi}, calculations.

Level	H_i (m)	m_i (ton)/fr	δ_i	Δ_{id}	Δ_{iy}
12	36	420	1.00	0.69	0.17
11	33	420	0.94	0.65	0.16
10	30	420	0.88	0.61	0.14
9	27	420	0.81	0.56	0.13
8	24	700	0.74	0.51	0.11
7	21	700	0.66	0.46	0.10
6	18	700	0.58	0.40	0.09
5	15	700	0.50	0.34	0.07
4	12	896	0.41	0.28	0.06
3	9	896	0.31	0.22	0.04
2	6	896	0.21	0.15	0.03
1	3	896	0.11	0.08	0.01

The equivalent SDOF structure displacement, effective height, effective mass and ductility are found as the following

$$\Delta_D = \frac{\sum m_i \Delta_{Di}^2}{\sum m_i \Delta_{Di}} = \frac{1324.49}{2892.33} = 0.457m \tag{16}$$

$$\Delta_y = \frac{\sum m_i \Delta_{yi}^2}{\sum m_i \Delta_{yi}} = \frac{69.37}{643.73} = 0.108m \tag{17}$$

$$\mu_w = \frac{\Delta_D}{\Delta_y} = \frac{0.457}{0.108m} = 4.24 \tag{18}$$

$$H_e = \frac{\sum m_i \Delta_{Di} H_i}{\sum m_i \Delta_{Di}} = \frac{63144.77}{2895.32} = 21.81m \tag{19}$$

$$m_e = \frac{\sum m_i \Delta_{Di}}{\Delta_D} = \frac{2895.32}{0.457} = 6329.12\ tonnes \tag{20}$$

To find the equivalent viscous damping, ξ , from equation (11) the slenderness ratio, $\bar{\lambda}$, should be known. However, it is unknown at this stage of design. An initial design of braces using an arbitrary value of, $\bar{\lambda}$, is carried out to find the base shear and choose the braces and slenderness ratios. Then, a number of iterations are done in which new shear forces and brace sizes are calculated. These iterations will stop when the same designed braces are chosen for the last two trials.

The SDOF equivalent viscous damping, ξ , in the last iteration, n, is calculated by equation (21) using the data shown in Table 2 where at the i storey $\xi_{i,n}$ is the equivalent viscous damping, $T_{i,n}$ is the axial force for the chosen brace and Δ_{Di} is design displacement

$$\xi = \frac{\sum T_{i,n} \Delta_{Di} \xi_{i.n}}{\sum T_{i,n} \Delta_{Di}} = \frac{3062}{15340} = 0.20 \tag{21}$$

Knowing ξ, the reduction factor can be applied to the elastic displacement spectrum to find the design displacement spectrum. Then the effective period, T_e, can be read from the design displacement spectrum as shown in Figure 4 and the equivalent stiffness and base shear can be calculated as the following

Table 2. Equivalent viscous damping calculations.

Level	$\xi_{i,n}$	$T_{b,i,n}*\Delta_{iD}$	$T_{b,i,n}*\Delta_{iD}*\xi_{i,n}$
12	0.16	861	141
11	0.18	1174	208
10	0.18	1337	235
9	0.19	1434	268
8	0.20	1668	325
7	0.20	1675	328
6	0.21	1670	359
5	0.21	1608	345
4	0.21	1466	314
3	0.22	1170	253
2	0.22	834	185
1	0.23	445	101

$$K_e = \frac{4\pi^2 m_e}{T_e^2} = \frac{4\pi^2 * 6329}{(6.3)^2} = 6293\ kN/m \qquad (22)$$

$$V_{base} = K_e\Delta_D + \frac{m_e g}{H_e}\Delta_D = 4181\ kN \qquad (23)$$

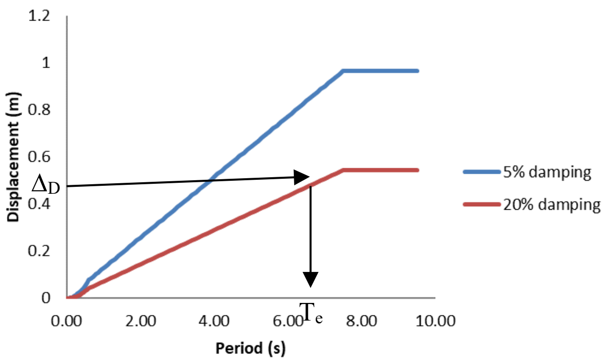

Figure 4. Displacement response spectra for the 5% and 20% damping

The base shear is distributed to storey forces using Equation (15) and storey shear forces, V_i, are found by summing the storey forces above the storey considered. As recommended by EC8 [11] the storey shear is assumed to be resisted by tension braces only. The brace area, Ab, is found by dividing the axial force in the brace by the yield strength, f_y, and the section sizes are chosen as shown in Table 3. All braces are chosen to be class 1 with a slenderness ratio $\bar{\lambda} \leq 2$, as suggested by EC8 [1], where $\bar{\lambda}$ is found by

$$\bar{\lambda} = \frac{L_{cr}}{i}\frac{1}{\lambda_1} \qquad (24)$$

where L_{cr} is the length of the brace, i is radius of gyration and $\lambda_1 = 93.9\varepsilon$, where $\varepsilon = \sqrt{235/f_y}$.

Columns and beams are designed to behave elastically. They are capacity designed as recommended by EC8 [11], to ensure that all seismic loads are dissipated by braces only.

Table 3. Calculations of distributed forces, shear and the design of brace element.

Level	Fi (kN)	Vi (kN)	A_b (cm^2)	section size
12	795.1	795.1	28.00	120X120X8
11	354.8	1150.0	40.49	140X140X10
10	331.3	1481.3	52.16	140X140X12.5
9	306.0	1787.3	62.93	160X160X12.5
8	465.0	2252.3	79.30	180X180X14.2
7	417.0	2669.3	93.99	200X200X14.2
6	366.2	3035.5	106.88	250X250X12.5
5	312.4	3347.9	117.88	250X250X14.2
4	327.3	3675.2	129.41	250X250X16
3	251.1	3926.3	138.25	260X260X16
2	171.1	4097.4	144.28	300X300X14.2
1	87.4	4184.8	147.35	350X350X12.5

4 NON-LINEAR TIME HISTORY ANALYSES (NLTHA)

In order to investigate the performance of the DDBD procedure for the case study structure, non-linear time history analyses (NLTHA) is carried out using the Open System for Earthquake Engineering Simulation, OpenSees, [27]. Seven different real earthquakes scaled to have displacement spectra that match the design displacement spectrum are used as shown in Figure 5 and Table 4 which gives the Earthquake location, ID used for the study, the magnitude, M, and the epicentre distance, r.

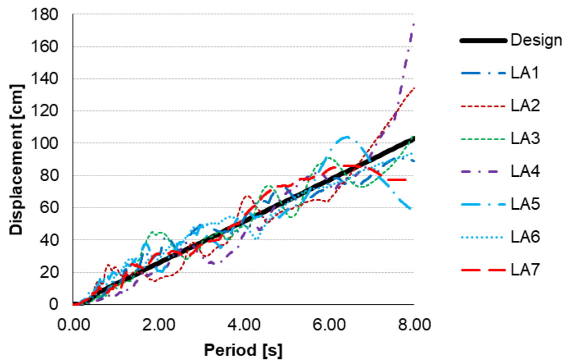

Figure 5. Displacement spectra for the seven real earthquakes used to verify the DDBD methodology using NLTHA scaled to the 5% design displacement spectrum.

Table 4. Ground motions applied to the NLTHA.

Earthquake	ID used	Magnitude, M	Distance, r (km)
Denali, Alaska	LA1	7.9	62
Chi-Chi, Taiwan	LA2	7.62	107
Chi-Chi, Taiwan	LA3	7.62	173
Chi-Chi, Taiwan	LA4	7.62	148
Darfield, NZ	LA5	7.1	-
Loma Prieta	LA6	6.93	84
Irpinia, Italy-01	LA7	6.9	33

In the numerical model, columns and beams are modelled to behave elastically with pinned connections. Braces are modelled with distributed plasticity and divided into elements using ten integration points per element. To allow global buckling of braces an initial camber of 1% of the length of the brace is introduced at the middle of the brace [18]. The fracture caused by low cyclic fatigue is captured by using a fatigue model wrapping the parental material to detect fracture in braces. The details of the fatigue model and the calibrated parameters can be found in Salawdeh and Goggins [18]. Hilber-Hughes-Taylor (HHT) method with constant Gamma equal to 0.5 is used to evaluate the dynamic response of the case study structure. The damping matrix is assumed to be proportional to the mass and stiffness matrices using the Rayleigh damping model.

The maximum floor displacements recorded during NLTHA for the seven earthquakes are compared with the design displacement profile as shown in Figure 6. Similarly, the average of the maximum recorded displacement for the seven earthquakes and the design displacement profile is shown in Figure 7. Furthermore, the maximum storey drifts recorded for the seven earthquakes during NLTHA compared with the maximum drift profile is shown in Figure 9 and the average of the maximum storey drifts recorded for the seven earthquakes compared with the maximum drift profile is shown in Figure 9. It is found that the average of the maximum floor displacements and storey drifts recorded from NLTHA are lying within the acceptable design limits. However, the average drifts were 3.6% and 8% more than the designed drift limit for the 11th and 12th storeys respectively as there was a very high drift for Earthquakes LA4 and LA6 in these floors.

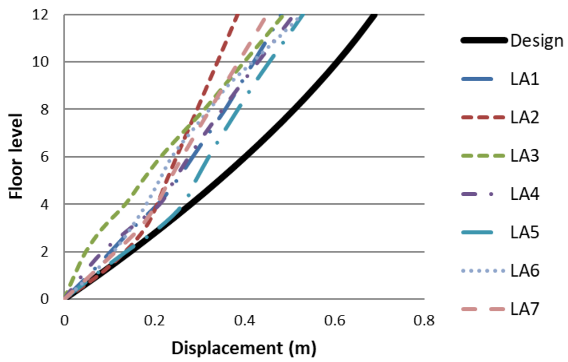

Figure 6. The maximum floor displacements recorded for the seven earthquakes compared with the design displacement.

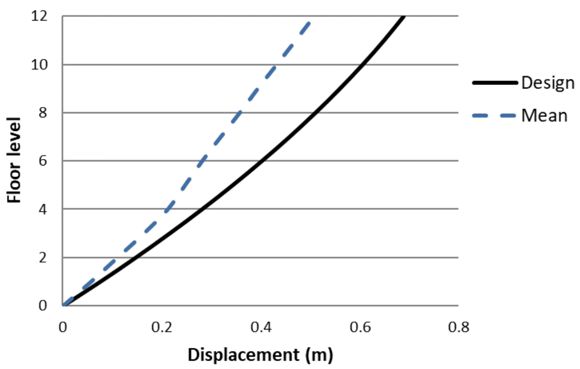

Figure 7. Average of the maximum floor displacements recorded for the seven earthquakes compared with the design displacements.

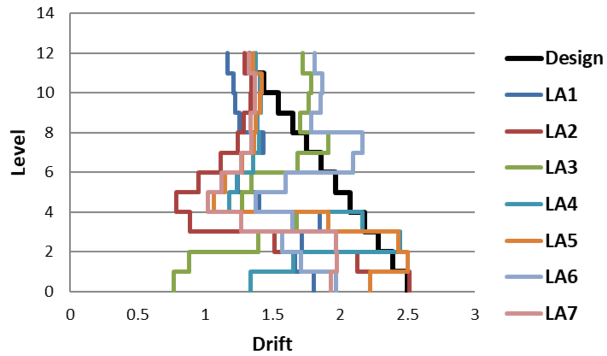

Figure 8. The maximum drifts recorded for the seven earthquakes compared with the design drifts.

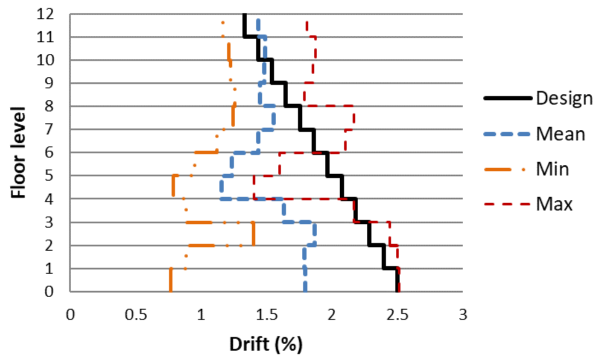

Figure 9. Average of the maximum drifts recorded for the seven earthquakes compared with the design drifts. The maximum and minimum drifts recorded from the seven earthquakes are included.

The maximum ductility values recorded from the NLTHA for the seven earthquakes are compared with the design ductility values obtained from the DDBD method, as shown in Figure 10. The average of the maximum ductility values observed from NLTHA are found to be less than the design ductility as shown in Figure 11.

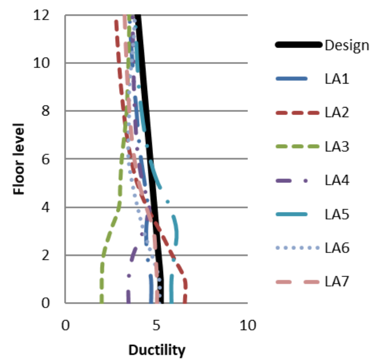

Figure 10. Maximum recorded ductility for the seven earthquakes from NLTHA compared with the design ductility for the case study structure.

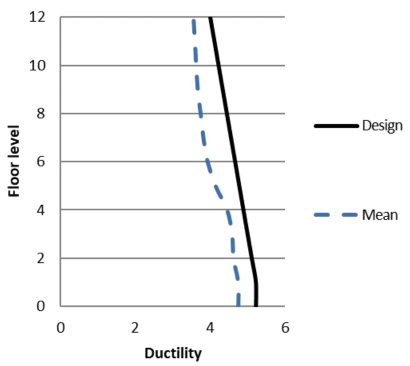

Figure 11. The average of the maximum recorded ductility for the seven earthquakes from NLTHA compared with the design displacement ductility for the case study structure

The lateral forces distributed to the irregular floors were assumed to be resisted by the tension brace members only and any contribution from the compression braces are neglected in the DDBD design procedure following the recommendation of EC8 [11]. However, in the NLTHA, as expected the compression member braces has resisted some of the lateral forces and, thus, the shear resistance from NLTHA was higher than the design shear causing the measured displacement and drift profiles to be conservative.

5 SUMMARY AND CONCLUSION

A DDBD methodology for vertically irregular CBF steel structures has been developed. The type of irregularity discussed was associated with setbacks up the height of the building where the frames have more bays at base of the structure than at top. All lateral forces are assumed to be resisted by tension braces only and columns and beams were capacity designed to behave elastically. The connections between columns, beams and braces were considered to be pinned in both ends.

A 12-storey case study CBF structure with vertical irregularity was carried out to verify the DDBD procedure. NLTHA was applied to the case study using seven different earthquakes scaled to have displacement spectra matching the design displacement spectrum. The results indicated that the DDBD procedure is conservative when compared to the recorded design displacements, storey drifts and brace ductility. The main reason is that both tension and compression members were found to resist seismic forces in NLTHA.

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