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# PRELIMINARY DESIGN OF HIGH RISE BUILDINGS FOR GLOBAL ADAPTATION USING THE LINEAR STATIC METHOD

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## ABSTRACT
***Preliminary sizing of the members of high-rise buildings for adaptation in Nigeria and other countries with similar earth tremor data is carried out in this work using the linear static (lateral force) method. The studied building model comprises a regular, symmetric 50 storey Steel Dual-Concentric (chevron) Brace Frame, SD-CBF. European wide flange beam section of HE220M, column section HE260M and brace section HE180B were realised as initial design sections which are structurally safe. Results indicate that the aforementioned sections, though structurally safe can be made more robust for greater safety by applying a factor of safety ranging from 1.25 to 1.5 depending on available investment and seismicity of the environment. This is to justify safety of lives and properties.***

***Keywords***: *High-rise, Earth-tremor, Linear Static Method and SD-CBF.*

## 1. INTRODUCTION
Steel is one of the most widely used materials for building construction in the world .The inherent strength; toughness and high ductility of steel are characteristics that are ideal for seismic analysis and design. Moment resisting frames (MRFs) have low elastic stiffness therefore; can require large member sizes to keep lateral drifts within obligatory limits demanded by seismic codes. Load-deflection (P-Δ) effect is another problem associated with such structures in high rise buildings and so could not fulfil serviceability requirements. Structural response is increased in Steel MRFs by introducing steel bracings in the structural system. Bracing can be applied as concentric bracing or eccentric bracing. There are 'n' number of possibilities to arrange steel bracings, such as cross bracing 'X', diagonal bracing 'D', 'K' and 'V' type bracing. These bracings are arranged to form vertical trusses and then lateral loading is resisted by truss action; ductility is developed through inelastic action. Failure occurs because of yielding of truss under tension or buckling of truss under compression [1].

Because of the obstructions caused by cross-braces, chevron braces are often preferred to allow for door and windows openings. Conventional chevron frames consist of two braces forming an inverted V-shape and meeting the upper storey beam at mid-span; while the fulfilment of serviceability limit state requirements is easy to obtain by means of such structural typology, some uncertainties arise about its adequacy to resist strong seismic actions by undergoing severe excursions in the non-linear range. The energy dissipation capacity of CBFs is in fact, almost completely related to non-linear hysteretic behaviour of diagonal braces under alternate tension and compressive internal forces [2]. Due to the inherent drawbacks of both MRFs and CBFs, MRF-CBF dual systems are more and more attracting the interest of researchers and practitioners as they constitute a reliable structural scheme which combines the advantages of both structural typologies, because of the exploitation of the local ductility supply of the beams of the moment resisting part and of the lateral stiffness provided by the diagonal members of the braced part. Therefore, dual systems constitute an effective structural solution able to satisfy ultimate and serviceability limit state requirements [3].

Four alternative analytical procedures are available for use in performance evaluation of steel moment-frame buildings; the first is the linear static procedure which is a method of estimating the response of a structure to earthquake ground shaking by representing the effects of this response through the application of a series of static lateral forces applied to an elastic mathematical model of the structure and its stiffness. The forces are applied to the structure in a pattern that represents the typical distribution of inertial forces in a regular

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structure responding in a linear manner to the ground shaking excitation, factored to account, in an approximate manner, for the probable inelastic behaviour of the structure. The linear static procedure inherently has more uncertainty associated with its estimates of the response parameters because it accounts less accurately for the dynamic characteristics of the structure [4]; so, it is used for:

- The preliminary analysis and design of multilevel buildings.
- Accounting for torsional incidence in multilevel structures.

## 2. BACKGROUND OF THE STUDY

A multi-story steel frame building with braces and shear walls, which was subjected to a simulated combined earthquake and dead loads was modelled by [5] using SAP2000. The building was assumed not too close to the seismic source (September 11th 2009 earth tremor in Abeokuta, Nigeria) [6], 22.5km from the site; however, if a large magnitude event is produced at the source, then the building can be affected by the earthquake. The soil is stiff with a shear wave velocity of 250m/s.

So, this present study attempts the preliminary design of the members of a high-rise building with, SD-CBF (chevron braces) for implementation in Nigeria and other countries with similar earth tremor data using the linear static (lateral force) method. This is because the seismicity of other similar countries like Nigeria rarely exceeds that determined for Abeokuta.

## 3. METHODOLOGY

### 3.1 Description of the Building

The structure is a fifty-storey regular symmetric office building composed of a dual steel system (MRF and chevron frames in the middle outer bays) .The gravity loads resisting system consists of composite floor system which is made up of 130mm lightweight concrete of dry density 19.00kN/m$^3$ over trapezoidal profiled steel decking of 0.11kN/m$^2$ unit weight leading to a permanent load of 3.60kN/m$^2$, while the variable load is 3.00kN/m$^2$, comprising 2.25kN/m$^2$ imposed load (category B) and 0.75kN/m$^2$ movable partition. The roof permanent load is assumed to be 0.90kN/ m$^2$, and its live load is 0.6kN/m$^2$; the roof is only accessible for normal maintenance and repair.

The steel profile used is classified as S355 European structural steel and has a Young's modulus of 210,000 N/mm$^2$ and yield strength of 355 N/mm$^2$ [7]. The plan and first floor 2-dimensionl elevation is as shown in Figure 1.

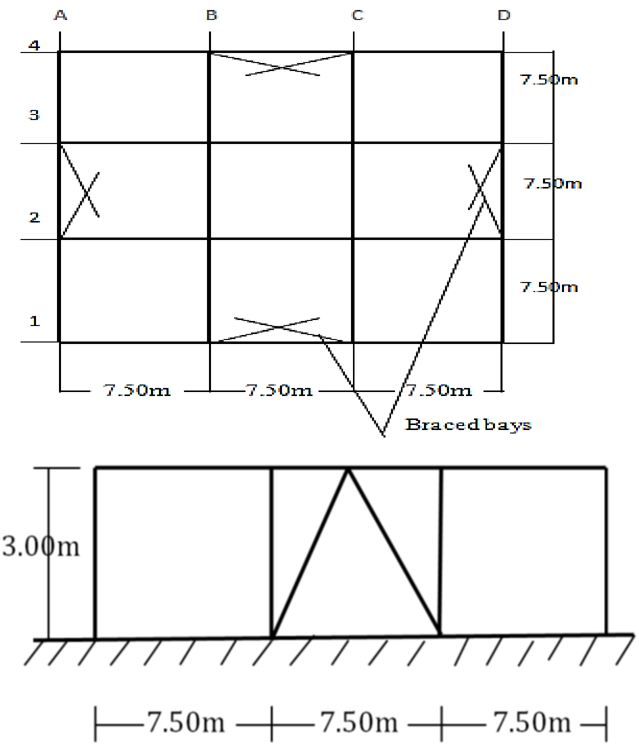


*Figure1: Plan and first floor 2-dimensional view of the building*

### 3.2. Considered Loadings

All loadings are in accordance to [7]; While the design is to [8] and [9] provisions.

The beams to be used for the section are determined initially by using two checks:

The moment resistance check and the deflection criteria. Beams are assumed to be fixed at both ends (rigid frames).

Permanent loads (G) on building:

Depth of slab = 130mm

Unit weight of lightweight concrete = 19.00kN/m$^3$

Weight of slab = 0.13 x 19.00 = 2.47kN/m$^2$

Weight of profiled steel decking = 0.11kN/m$^2$

Assumed weight of ceiling, raised floor and services = 0.75kN/m$^2$

Total weight = 2.47+0.11+0.75 = 3.33kN/m$^2$, say 3.60kN/m$^2$

G = 3.60 x 7.50m = 27.00kN/m

Imposed loads (Q): The structure is an office building: category B (clause 6.3.1.1 of [7])

Imposed floor load for offices (Category B) = 2.25kN/m$^2$

Assumed weight of movable partitions = 0.75kN/m$^2$

Bay length = span (L) = 7.50m

Elastic modulus of steel (E) = $2.1\ x\ 10^5 N/mm^2$

Steel profile grade ($f_y$) = S355$N/mm^2$

Partial factor for steel $(\gamma_y) = 1$

## 4. RESULTS AND DISCUSSIONS

### 4.1. Selection of Beams Section

*i. Moment Resistance Check*

Designed load (w) = 1.35G +1.5Q = (1.35x27.00) + (1.5 x 22.50) = 70.20kN/m

Since beams are fixed at both ends,

$$M_{sd} = \frac{wL^2}{12} = \frac{70.20 \times 7.50^2}{12} = 329.06kNm \quad (1)$$

$$W_{pl.min.} = \frac{M_{sd}}{f_y} = \frac{329{,}06 \times 10^6}{355} = 927\ x\ 10^3 mm^3 \quad (2)$$

Minimum beam section = HE 220M with $W_{pl} = 1135\ x\ 10^3 mm^3$

*ii. Deflection Check*

Deflection of a section with uniformly distributed load:

$$\delta = \frac{wL^4}{384EI} \quad (3)$$

Deflection limit: $\delta = \frac{L}{300}$

$$I_{req} = \frac{300wL^3}{384E} = \frac{300 \times 70{,}20 \times 7.50^3 \times 10^6}{384 \times 2.1 \times 10^8} = 11018cm^4 \quad (4)$$

Minimum beam section = HE 220M with $I_{req} = 14600cm^4$

Therefore, for gravity loading only (excluding self-weight of the beam and lateral loads) HE 220M was the minimum beam section selected for both the X and Y direction.

### 4.2. Determination of Column Section

The section for an interior column (worst case) was determined assuming that all storeys have equal masses using the weak beam, strong column (WBSC) check:

For the H220M beam

$W_{ply} = 1419cm^3$ and $W_{plz} = 679cm^3$

The WBSC check is expressed as:

$$\sum M_{RC} \geq 1.3 \sum M_{RB} \quad (5)$$

This can also be expressed as:

$$\sum f_{yc}\ x\ W_{plc} \geq 1.3 \sum f_{yb}\ x\ W_{plb} \quad (6)$$

Same steel grade ($f_y$) was chosen for both beams and columns that is S355, so the WBSC checks becomes:

$$\sum W_{plc} \geq 1.3 \sum W_{plb} \quad (7)$$

At interior nodes, there are two beams and two columns intersecting, so the WBSC check is:

$$W_{plc} \geq 1.3\ W_{plb} \quad (8)$$

At exterior nodes, there is one beam and two columns intersecting, so the WBSC check becomes:

$$2W_{plc} \geq 1.3\ W_{plb} \quad (9)$$

Considering the worst case at an interior node, say the intersection of line B2, B3and C2, C3 as shown in Figure1

$$W_{plc,weak\ axis} \geq 1.3\ W_{plb,HE\ 220M} \quad (10)$$

$$W_{plc,weak\ axis} \geq 1.3\ x679\ x\ 10^3 mm^3$$
$$\geq 883\ x\ 10^3 mm^3$$

For HE 240M, $W_{plz} = 1006cm^3 \geq 883cm^3$

Hence with a HE240M column, the WBSC criteria is satisfied, but HE260M column size was adopted in order to make allowance for the self-weight of the beam and column.

### 4.3 Sizing of Bracing

I. *Design summary:*

i. The seismic loading at each floor is transferred to the vertically braced central bays on all sides of the building by the floors acting as diaphragms.

ii. The braced bays, acting as vertical pin-jointed frames, transfer the lateral seismic load to the ground.

iii. The beams and columns that make up the bracing system have already been designed for gravity loads. Therefore, only the diagonal members have to be designed and only the forces in these members have to be calculated.

iv All the diagonal members are of the same section, thus, only the most heavily loaded member has to be designed.

I. *Evaluation of the total mass of the building*

'kg' is the unit used for mass.

Permanent loads

$G_{\text{floor, ceilings and services}}$ = 360kg/ m² x 56.25m² = 20250kg /storey

$G_{\text{frame}}$:

Column HE260M: 3m x 16 x 172Kg/m = 8256 kg

Beams HE220M: 7.5m x 3 x 8 x 117 Kg/m = 21060 kg

Total $G_{\text{frame}}$: 29316 kg/storey

$G_{\text{roof}}$ = 75kg/ m² x 56.25m² = 4218.75 kg/storey

Total permanent load of the building, G: G = 49 ($G_{\text{floor}}$ + $G_{\text{frame}}$) + $G_{\text{roof}}$ = 49(20250 + 29316) + 4218.75 = 2432.953 x 10³kg

Imposed load

$Q_{\text{imposed}}$ = 225 kg/ m² x 56.25 m² = 12656.25 kg /storey

$Q_{\text{partition}}$ = 22.5 x 8 x 75kg/m² = 13500kg/storey

$Q_{\text{roof}}$ = 60kg/m² x 56.25 = 3375kg/ storey

Total variable load of the building: Q = 49 (12656.25 + 13500) + 3375 = 1285.03x 10³ kg/storey

Total mass of the building, M = G + $\psi_{Ei}$ Q (11)

M= 2432953 + 0.3 x 1285.03= 2818.46 x10³ kg

II. *Evaluation of seismic design shear using the 'lateral forces' method*

In this section the approximate linear static 'lateral forces' method is considered.

The fundamental period of the structure (clause 4.3.3.2.2, [9]) is given as:

$$T = C_t H^{0.75} \tag{12}$$

Where: $C_t$, a coefficient, is 0,085 for moment resistant space steel frames; (clause 4.3.3.2.2 (3))

H is the height of the building, from the foundation or from the top of a rigid basement

$C_t = 0.085$, H = 50 x 3 = 150m

$T = 0.085\ x\ 150^{0.75} = 3.64$

For the design pseudo acceleration $S_d$ (T),

$$S_d\ (T) = \begin{cases} a_g\ .S.\dfrac{2.5}{q}\left[\dfrac{T_C T_D}{T^2}\right] \\ \geq\ \beta\ a_g \end{cases} \tag{13}$$

Where: 'q' is the behavior factor of the structure

'β' is the lower bound factor for the horizontal design spectrum (recommended as 0.2).

'$T_B$' 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.

Values of the parameters describing the recommended Type 2 elastic response spectrums are:

(s) =1.5, $T_B$ (s) = 0.1, $T_C$ (s) = 0.25, $T_D$ (s) =1.2

$T_D\ (1.2) < T(3.64)$

$a_g = 0.1g = 0.98m/s^2$

$$S_d\ (T) = \begin{cases} 0.98\ x\ 1.5\ x\ \frac{2.5}{4}\left[\frac{0.25\ x\ 1.2}{3.64^2}\right] = 0.021m/s^2 \\ 0.2\ x\ 0.98 = 0.196m/s^2 \end{cases}$$

Since, $S_d\ (T) calculated\ <\ S_d(T)\ recommended$, that is, 0.021 < 0.196, $0.196m/s^2$ was adopted

Seismic design shear

$$F_b = MS_d\ (T)\ \boldsymbol{\lambda} \tag{14}$$

$= 2818.46 \text{ x } 10^3 \text{ x } 0.196 \text{ x } 0.85 =$
$2046.96\text{x } 10^3\boldsymbol{N} = 470\text{kN}$

'M' is the total mass of the building and 'λ' is a correction factor, given as 0.85 (clause 4.3.3.2.2. (1), [9]).

Account is taken of torsion by amplifying the base shear by the factor $\delta$ (clause 4.3.3.2.4, [9]).

$$\delta = 1 + 0.6\ \frac{X}{L} \tag{15}$$

Where: L is the horizontal dimension of the building perpendicular to the earthquake direction considered = 22.5m. X is the center of rigidity of the frame in which the effects of torsion are to be evaluated. The greatest effect is obtained for the greatest X at 0.5L = 0.5 x 22.5 = 11.25. So, $\delta = 1 + 0.6\left(\frac{11.25}{22.50}\right) = 1.3$

The design base shear including torsional effects is therefore: $F_{bt} =\ \delta\ x\ F_b = 1.3\ x\ 470 = 611kN$

The design seismic base shear force applied on each MR frame in either the X or Y direction is

$$F_{btx} = \frac{F_{bt}}{x} \tag{16}$$

Where x is the number of frames in each direction of the building;

$F_{btx} = \frac{611}{4} = 152.75kN$

To calculate the lateral forces at each floor level Equation 4.11in [9] is used

$$F_i =\ F_{btx}\ \frac{Zimi}{\sum Zjmj} \tag{17}$$

Table 1 below shows the spreadsheet for the sequence of calculation. It is obvious that the greatest lateral force of 152kN is obtained at the first floor level. $Z_i, Z_j$ are the heights of the masses $M_i, M_j$ above the level of application of the Seismic action.

*Table 1: Brace Design: Equivalent Lateral Forces at Each Level*

| Floor level | $Z_i(m$ | $M_i(kN$ | $Z_iM_i(kNm$ | $\frac{Z_iM_i}{\sum Z_iM_i}$ | $F_i(kN$ | $\sum F_i$ |
|---|---|---|---|---|---|---|
| Roof | 150 | 5.23 | 784.5 | 0.0037 | 0.5668 | 0.5668 |
| 49 | 147 | 57.41 | 8439.27 | 0.03958 | 6.05574 | 6.62254 |
| 48 | 144 | 57.41 | 8267.04 | 0.03877 | 5.93181 | 12.55435 |
| 47 | 141 | 57.41 | 8094.81 | 0.03797 | 5.80941 | 18.36376 |
| 46 | 138 | 57.41 | 7922.58 | 0.03716 | 5.68548 | 24.04924 |
| 45 | 135 | 57.41 | 7750.35 | 0.03635 | 5.56155 | 29.61079 |
| 44 | 132 | 57.41 | 7578.12 | 0.03554 | 5.43762 | 35.04841 |
| 43 | 129 | 57.41 | 7405.89 | 0.03474 | 5.31522 | 40.36363 |
| 42 | 126 | 57.41 | 7233.66 | 0.03393 | 5.19129 | 45.55492 |
| 41 | 123 | 57.41 | 7061.43 | 0.03312 | 5.06736 | 50.62228 |
| 40 | 120 | 57.41 | 6889.2 | 0.03231 | 4.94343 | 55.56571 |
| 39 | 117 | 57.41 | 6716.97 | 0.0315 | 4.8195 | 60.38521 |

| Floor level | $Z_i(m$ | $M_i(kN$ | $Z_iM_i(kNm$ | $\frac{Z_iM_i}{\sum Z_iM_i}$ | $F_i(kN$ | $\sum F_i$ |
|---|---|---|---|---|---|---|
| 38 | 114 | 57.41 | 6544.74 | 0.0307 | 4.6971 | 65.08231 |
| 37 | 111 | 57.41 | 6372.51 | 0.02989 | 4.57317 | 69.65548 |
| 36 | 108 | 57.41 | 6200.28 | 0.02908 | 4.44924 | 74.10472 |
| 35 | 105 | 57.41 | 6028.05 | 0.02827 | 4.32531 | 78.43003 |
| 34 | 102 | 57.41 | 5855.82 | 0.02747 | 4.20291 | 82.63294 |
| 33 | 99 | 57.41 | 5683.59 | 0.02684 | 4.10636 | 86.7393 |
| 32 | 96 | 57.41 | 5511.36 | 0.02585 | 3.95505 | 90.69435 |
| 31 | 93 | 57.41 | 5339.13 | 0.02504 | 3.83112 | 94.52547 |
| 30 | 90 | 57.41 | 5166.9 | 0.02423 | 3.70719 | 98.23266 |
| 29 | 87 | 57.41 | 4994.67 | 0.02343 | 3.58479 | 101.8175 |
| 28 | 84 | 57.41 | 4822.44 | 0.02262 | 3.46086 | 105.2783 |
| 27 | 81 | 57.41 | 4650.21 | 0.02181 | 3.33693 | 108.6152 |
| 26 | 78 | 57.41 | 4477.98 | 0.021 | 3.213 | 111.8282 |
| 25 | 75 | 57.41 | 4305.75 | 0.02019 | 3.08907 | 114.9173 |
| 24 | 72 | 57.41 | 4133.52 | 0.01939 | 2.96667 | 117.884 |
| 23 | 69 | 57.41 | 3961.29 | 0.01858 | 2.84274 | 120.7267 |
| 22 | 66 | 57.41 | 3789.06 | 0.01777 | 2.71881 | 123.4455 |
| 21 | 63 | 57.41 | 3616.83 | 0.01696 | 2.59488 | 126.0404 |
| 20 | 60 | 57.41 | 3444.6 | 0.01616 | 2.47248 | 128.5129 |
| 19 | 57 | 57.41 | 3272.37 | 0.01535 | 2.34855 | 130.8614 |
| 18 | 54 | 57.41 | 3100.14 | 0.01454 | 2.22462 | 133.0861 |
| 17 | 51 | 57.41 | 2927.91 | 0.01373 | 2.10069 | 135.1868 |
| 16 | 48 | 57.41 | 2755.68 | 0.01292 | 1.97676 | 137.1635 |
| 15 | 45 | 57.41 | 2583.45 | 0.01212 | 1.85436 | 139.0179 |
| 14 | 42 | 57.41 | 2411.22 | 0.01131 | 1.73043 | 140.7483 |
| 13 | 39 | 57.41 | 2238.99 | 0.0105 | 1.6065 | 142.3548 |
| 12 | 36 | 57.41 | 2066.76 | 0.00969 | 1.48257 | 143.8374 |
| 11 | 33 | 57.41 | 1894.53 | 0.00889 | 1.36017 | 145.1975 |
| 10 | 30 | 57.41 | 1722.3 | 0.00808 | 1.23624 | 146.4338 |
| 9 | 27 | 57.41 | 1550.07 | 0.00727 | 1.11231 | 147.5461 |
| 8 | 24 | 57.41 | 1377.84 | 0.00646 | 0.98838 | 148.5345 |
| 7 | 21 | 57.41 | 1205.61 | 0.00565 | 0.86445 | 149.3989 |
| 6 | 18 | 57.41 | 1033.38 | 0.00485 | 0.74205 | 150.141 |
| 5 | 15 | 57.41 | 861.15 | 0.00404 | 0.61812 | 150.7591 |
| 4 | 12 | 57.41 | 688.92 | 0.00323 | 0.49419 | 151.2533 |
| 3 | 9 | 57.41 | 516.69 | 0.00242 | 0.37026 | 151.6235 |
| 2 | 6 | 57.41 | 344.46 | 0.00162 | 0.24786 | 151.8714 |
| 1 | 3 | 57.41 | 172.23 | 0.00081 | 0.12393 | 151.9953 |
| Total | 3825 | 2818.32 | 211766.3 | 1 | 151.995 | |

Resolving forces horizontally at first floor level is sufficient to calculate the force in the lowest (most highly loaded) bracing member.

Horizontal component of force in bracing member = 152kN

Vertical component of force in bracing member = $\frac{152}{0.5\ x\ 7.5}\ X\ 3 = 122kN$

Brace axial load $= \sqrt{152^2 + 122^2} = 195kN$

Trial section: HE180B grade S355 flanged section

Section Properties: Area, A = 63.50 cm², Depth of section, d = 180mm; Second moment of area, I = 3831cm⁴, Radius of gyration, r = 7.66cm, flange Thickness, $t_f$ = 14.00 mm, web thickness, $t_w$=8.5mm.and ratio for local buckling d / t = 12.86

Material properties
As $t \leq 16$ mm, for S355 steel, Yield strength $f_y = 355$ N/mm$^2$
Modulus of elasticity E = 210 kN/mm$^2$
Section classification
Class 1 limit for section in tension,

$$d/t \leq 50\varepsilon^2 \quad (18)$$

$$\varepsilon = \left(235/f_y\right)^{0.5}, f_y = 355N/mm^2 \Rightarrow \varepsilon = 0.82$$

$$d/t \leq 50\varepsilon^2 = 50 \, x \, 0.82^2 = 33.6$$

Since 12.86 < 33.6, the section is Class 1 for axial tension
Design of member in tension
Cross sectional resistance to axial tension
Basic requirement,

$$\frac{N_{Ed}}{N_{t,RD}} \leq 1.0 \quad (19)$$

$N_{Ed}$ is the design value of the applied axial force $N_{Ed}$ = 852kN; $N_{t,Rd}$ is the design resistance of the cross-section for uniform tension

$$N_{t,Rd} = \frac{A \, x \, f_y}{\gamma_{mo}} \; (for\ class 1, 2\ and\ 3) \quad (20)$$

$$\gamma_{mo}\ is\ the\ partial\ factor\ for\ resistance$$

$$N_{t,Rd} = \frac{6350 \, x \, 355 \, x \, 10^3}{1} = 2254kN$$

$$\frac{N_{Ed}}{N_{t,Rd}} = \frac{195}{2254} = 0.09 < 1.0$$

Therefore, the capacity of the cross section is adequate; a much smaller section can also be tried. When the seismic force is applied in the opposite direction, the bracing member considered above will be loaded in compression. By inspection, the compressive resistance is equal to the cross-sectional resistance, 2254kN, > 195 kN, OK.

## 5. CONCLUSION

With these initial sizes of beams = HE220M, columns = HE260M and braces HE180B the design of the structure is safe. These sections give the minimum members to provide structural safety. However, for greater structural safety, reliability and cost/benefit implications, a factor of about 1.25 to 1.50 could be applied safely to the member sizes in order to accommodate higher loads and other accidental loads which may not be readily seen, or estimated, since seismicity may be exceeded in many cases. A structural analysis and design software could also be used to validate and improve the results obtained.

With this presentation, the skyline in Nigeria and other countries with similar seismic data can be used, especially in cities, while conserving available land for future generations of such countries. It will also increase vertical development and technology base for modular building constructions and systems. The city environment will be neater, more friendly and with green technological investment.

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