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Recent Advances in Structural Design in Regions of Low-to-Moderate Seismicity

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Seismic Design of RC Shear Wall-Frame Structures in Singapore

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ABSTRACT Since Singapore is located on a stable part of the Eurasian Plate, with the nearest earthquake fault 400 km away in Sumatra, before 2013-2015 buildings in Singapore were gravity-load designed (GLD) structures designed according to BS8110, which does not have any seismic provision. However, they were occasionally subjected to tremors due to the far-field effects of earthquakes in Sumatra (Balendra et al. 1990, 1999). The research on seismic performance including capacity of GLD reinforced concrete structures had been carried out in Singapore context since the last two decades (Balendra et al. 1999, 2001, Kong et al. 2003). A microscopic model calibrated for shear walls was used to determine the capacity of the full scale shear wall structures (Kong 2004). Also a macroscopic model for capacity evaluation of shear wall-frame structures was presented on pushover analysis of a 25 storey shear wall-frame point block (Balendra et al. 2007). Past experience during earthquakes reveal that buildings which are designed for seismic loads are able to withstand earthquakes of magnitude several times larger than that for which they have been designed. This is largely due to overstrength and ductility of the structure. It is found that the buildings in Singapore, which are not designed for earthquake loads, possess overstrength varying from 4 to 12 times the design strength depending on the type of buildings (Balendra et al. 2012). All these research works amongst others contributed a part to the development of BC3: 2013 and SS EN 1998-1:2013 (Singapore's National Annex to Eurocode 8). **This presentation will discuss and focus on the seismic action considerations and requirements of Singapore's BC3 guide and SS EN 1998-1 (2013) which became mandatory as of 2015 with building example calculation for practitioners.**

BC3: 2013

Note: Para* refer to relevant paragraphs in BC3: 2013.

In Singapore, the "Guidebook for Design of Buildings in Singapore to Requirements in SS EN 1998-1" referred to as BC3: 2013 gives provisions for the structural design against seismic actions and is to be read in conjunction with the Singapore National Annex to SS EN 1998-1: 2012.

1.2 As Singapore is in a low seismicity region, Ductility Class Low (DCL)[D] design and detailing can be adopted for reinforced concrete, precast concrete, structural steel or composite buildings.

[D] DCL steel reinforcement detailing for reinforced concrete structures would follow the requirements of SS EN 1992-1-1 (Design of concrete structures – General rules and rules for buildings) and in conjunction with Clause 5.3.2(1)P of SS EN 1998-1. For buildings of structural steel or composite construction, the element design shall follow the relevant parts of SS EN 1993 (Design of steel structures) or SS EN 1994 (Design of composite structures) respectively.

- In Singapore residential building context, most of the seismic analysis shall be computed with 3D Modal Analysis with response spectrum
- Based on EC8 to Singapore code, for a reasonably regular building, seismic loading most likely will govern building design upto 30 stories
- Beyond 30 stories, Wind Load or Notional Load most likely governing building design

SS EN 1998-1: 2012.

Cl. 2.1

Fundamental Requirements (1)

<u>**No-Collapse Requirement (NCR)**</u>

- **To withstand the design seismic action without local or global collapse,** thus retaining its structural integrity and a residual load bearing capacity after the seismic events.
- <u>**Design**</u> **seismic action** is expressed in terms of:

  (a) **reference seismic action** associated with a reference probability of exceedance, P_{NCR}, in **50 years** (or reference return period T_{NCR}), and

  (b) **importance factor** γ_I of building ("reliability differentiation")
- Recommended P_{NCR} **= 10% in 50 years** corresponds to T_{NCR} **= 475 years** – this is **adopted in Singapore National Annex (NA)**

Seismic Hazard Levels Considered

Exceedance Probability (P_N)	Time Span (N)	Return Period (T)	Hazard Levels Considered In Seismic Design Codes
20%	10 years	45 years	
10%	**10 years**	**95 years**	**"Damage Limitation Requirement" in EC8****
20%	50 years	225 years	
10%	**50 years**	**475 years**	**"No-Collapse Requirement" in EC8**** Design EQ in UBC97 Design EQ in IBC2012++
5%	50 years	975 years	
10%	100 years	950 years	Max. Capable EQ in UBC97
2%	50 years	2475 years	Max. Considered EQ in IBC2012

** Based on the recommended value of **10%** for P_{DLR} and P_{NCR} in EC8.

++ Approximate equivalence (DE = 2/3 * MCE in IBC2012)

SS EN 1998-1: 2012.
Cl. 2.2

Compliance Criteria for Seismic Design

- To satisfy the fundamental requirements, two limit states shall be checked:

(1) **Ultimate limit state (ULS)** – associated with collapse or other forms of structural failure which might endanger safety of people.

(2) **Damage limitation state (DLS)** – associated with damage beyond which specified service requirements are no longer met.

- To limit uncertainties and promote good behaviour of structures under seismic actions more severe than the design seismic action, some specific measures are taken as per 2.2.4 (e.g. simple and regular forms, ductile behaviour, good detailing, appropriate analysis model, etc).
- For **low seismicity**, simpler rules can be applied for certain structures (low ductility class), e.g. **detailing follows EC2, EC3 and EC4**.
- For very low seismicity, EC8 needs not be observed.
- **Singapore NA:** **Low seismicity** if $a_g S \leq 0.98$ m/s²
  Very low seismicity if $a_g S \leq 0.39$ m/s²

  where a_g = design ground acceleration on type A ground, and S = soil factor.

  (This check is taken care of by ground type and building type in Singapore NA.)

- **Reference peak ground acceleration (a_{gR})** corresponds to the reference return period T_{NCR} of the seismic action for the no-collapse requirement –

a_{gR} = 0.175 m/s² **for Singapore**

1. General Philosophy

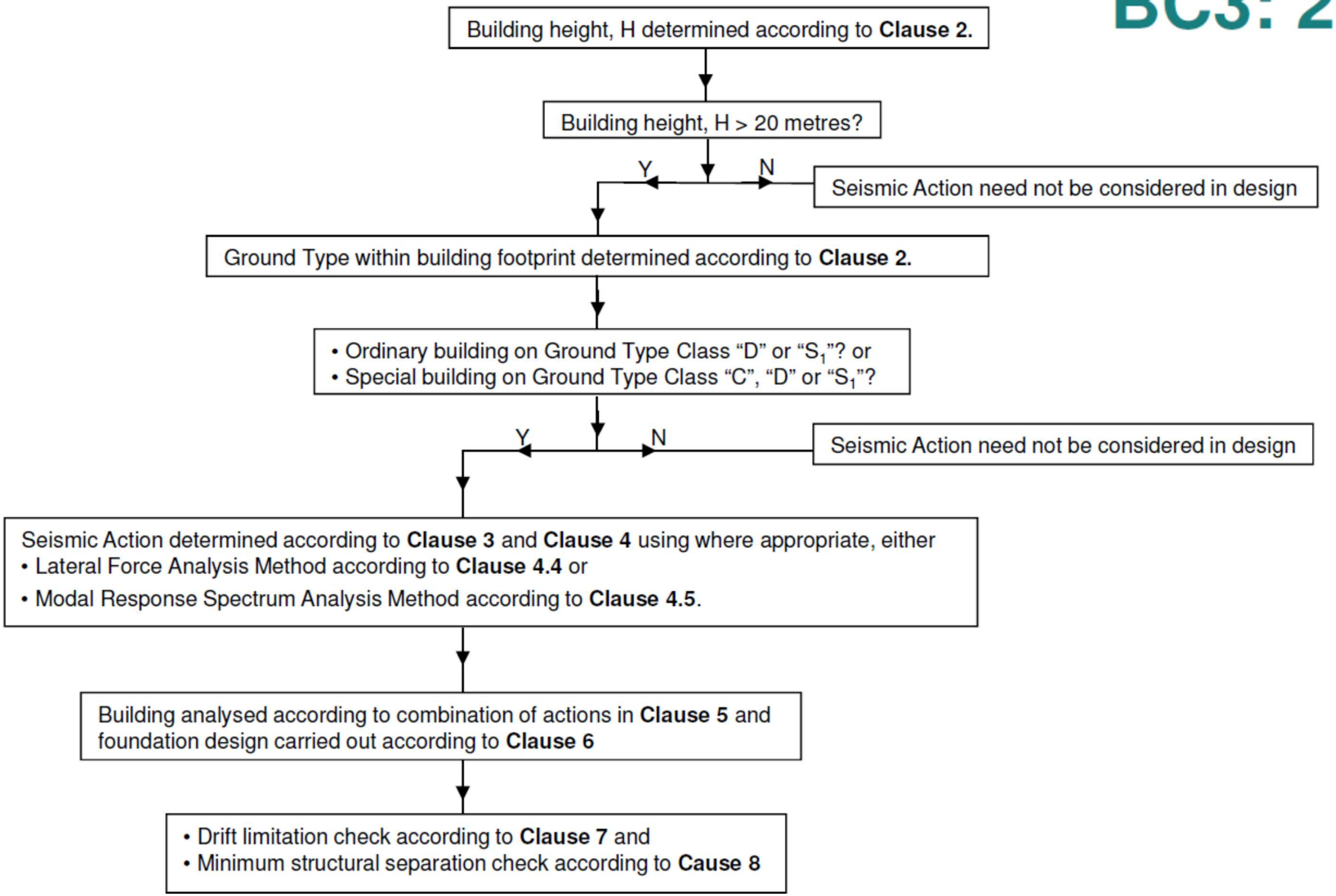
1.1 New buildings that are above 20 metres in height and existing buildings undergoing very major addition or alteration works[A] founded on certain Ground Types shall be checked for an enhanced robustness consideration to cater for impact of Seismic Actions due to distant earthquakes with a probability of exceedance of 10% in 50 years according to the methodology outlined in the flowchart in Figure 1. This requirement will apply to the following types of building and the corresponding Ground Types (determined as per paragraph 2.3):

- Special buildings[B] on Ground Types "C", "D" or "S_1" and
- Ordinary buildings[C] on Ground Types "D" or "S_1"

[A] Very major addition or alteration works refers to:

- addition of floors on existing buildings that results in the building attaining a height greater than 20 metres, or
- structural works affecting key structural elements supporting total tributary area of more than 60% of the total structural floor area of a building of height greater than 20 metres, or
- additions of new structural floor areas of more than 60% of the existing total structural floor area of a building of height greater than 20 metres.

BC3: 2013



Building height, H determined according to **Clause 2.**
Building height, H > 20 metres?
Y
N
Seismic Action need not be considered in design
Ground Type within building footprint determined according to **Clause 2.**
• Ordinary building on Ground Type Class "D" or "S_1"? or
• Special building on Ground Type Class "C", "D" or "S_1"?
Y
N
Seismic Action need not be considered in design
Seismic Action determined according to **Clause 3** and **Clause 4** using where appropriate, either
• Lateral Force Analysis Method according to **Clause 4.4** or
• Modal Response Spectrum Analysis Method according to **Clause 4.5**.
Building analysed according to combination of actions in **Clause 5** and foundation design carried out according to **Clause 6**
• Drift limitation check according to **Clause 7** and
• Minimum structural separation check according to **Cause 8**

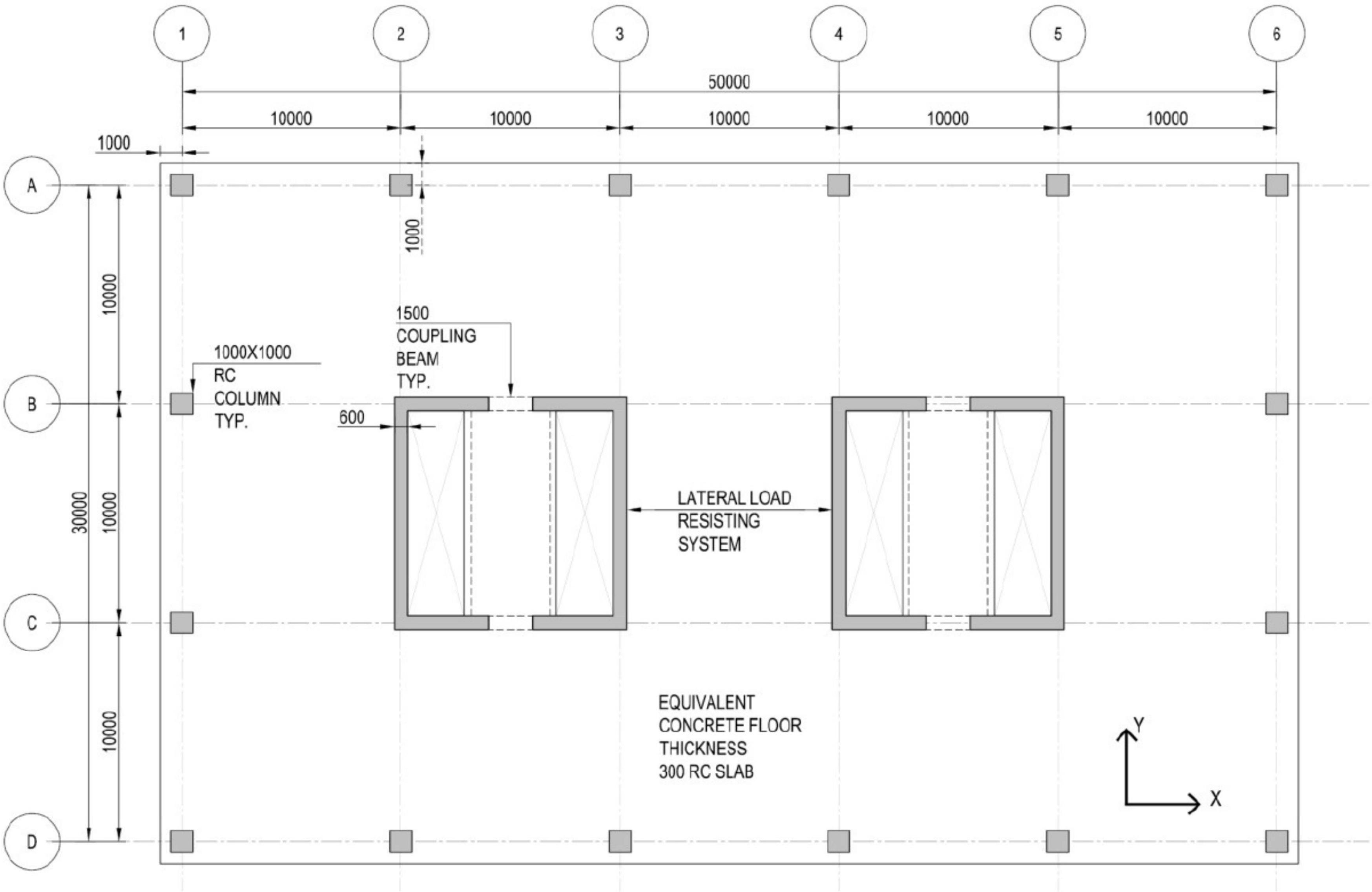
1. An RC Shear-Wall Frame Building Example Introduction

In this work example, a step- by- step procedure for the elastic analysis of a building is described for Seismic Actions highlighting the key recommendations of BC3, using Ductility Class DCL for practising engineer’s guide.

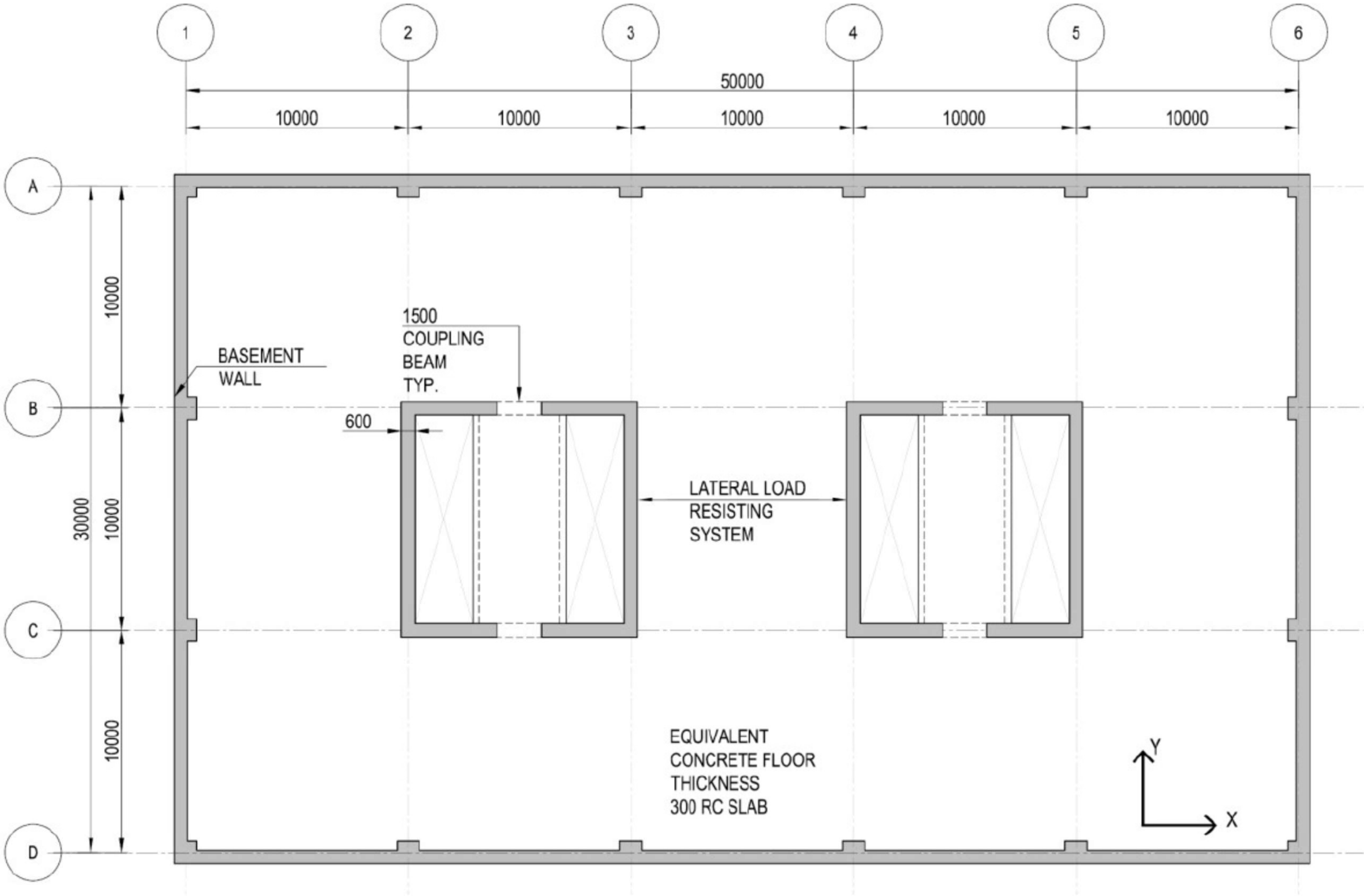
The example building is a multi-storey reinforced concrete shear-wall frame structure.

Two floor plans (typical and basement) and the elevation of the building are shown in next Figures.

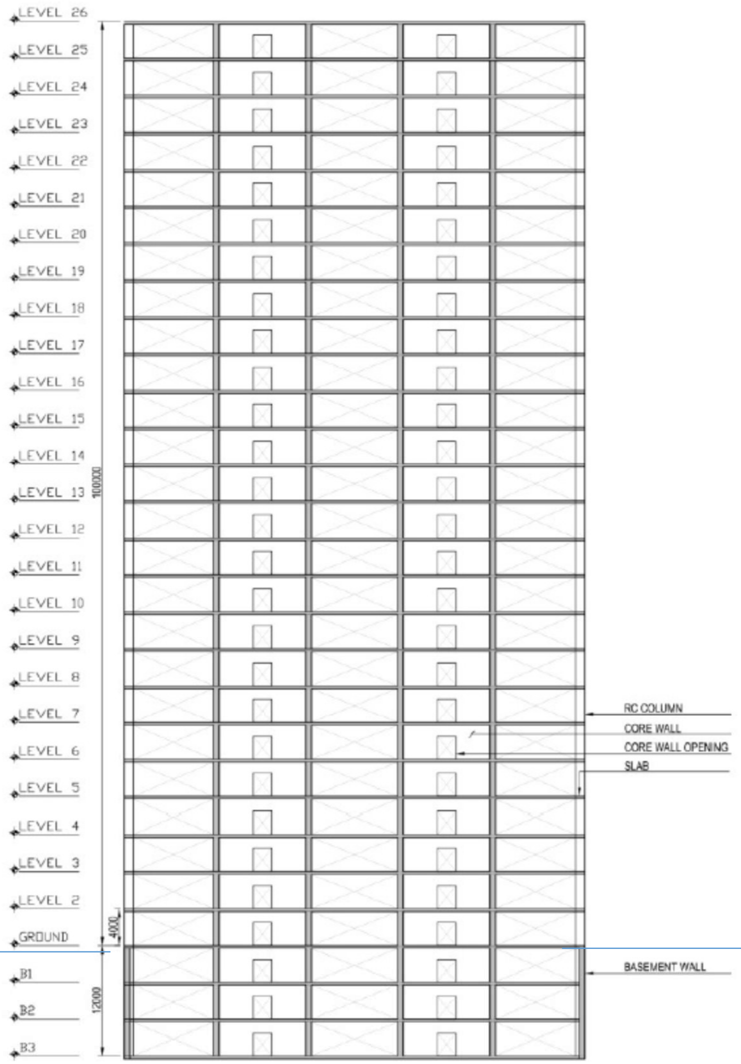
The building properties are listed too.



Example Building Floor Plan

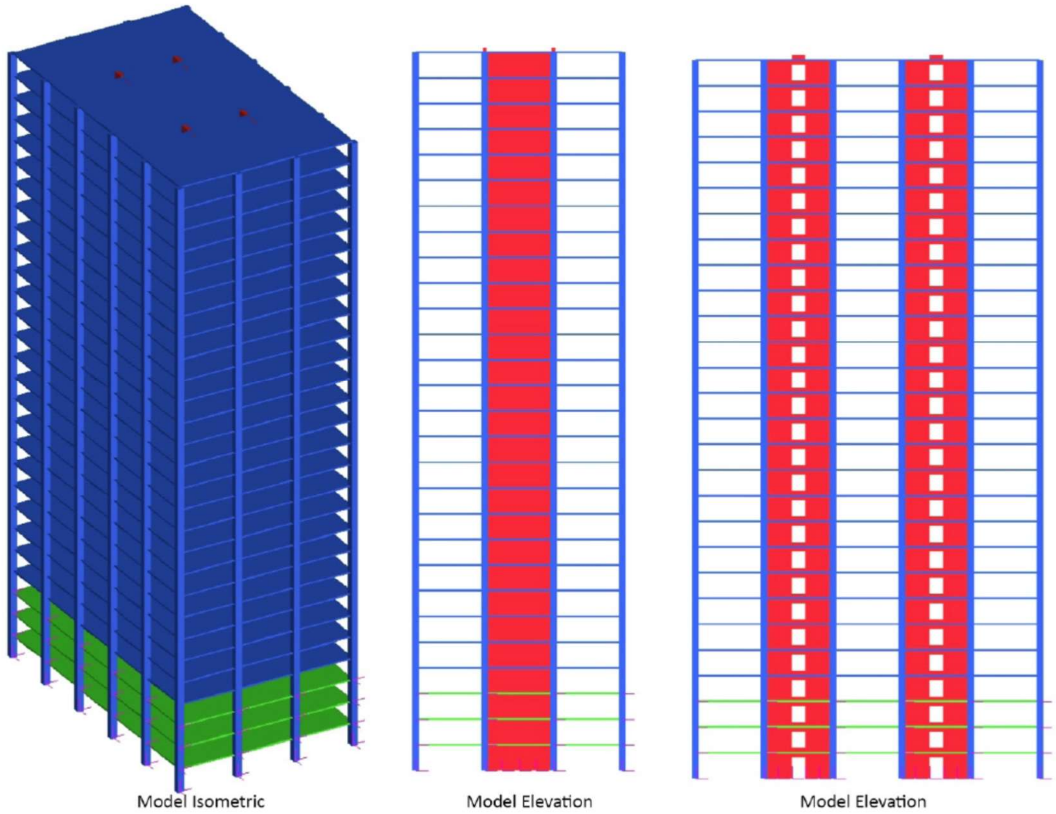


Example Building Basement Floor Plan



Example Building Elevation

A three-dimensional structural model is used for the analysis. The structural model fulfills all the requirements of SS EN 1998-1, Clause 4.3.1, and is shown in Figure **below**



Analysis Model

4.2 Structural model. The model of the building used in the structural analysis shall fulfil all requirements in clause 4.3.1 of SS EN 1998-1. Refer to clause 4.3.1 (6) & (7)[L] of SS EN 1998-1 for guidance on modelling of cracked behaviour of concrete or composite buildings.

2. Building Properties Used in Example

Note: Para* refer to relevant paragraphs in BC3: 2013.

- Building Type: Ordinary (per *Para* 1.2)*
- Importance Factor: 1.0 (per SS EN 1998-1, Clause 4.2.5 (5))
- Ground Type: D (assumed per *Para* 2.3)*
- Number of above ground storys: 25
- Number of below ground storys: 3
- Typical Story Height: 4m
- Typical Floor Area: (50+2) x (30+2) = 1664m^2
- Typical Loading:
  - SDL = 2.5 kPa
  - LL = 3.5 kPa
- Key structural element sizes: Shown on plan.
- Lateral Load Resisting System: Central Core Walls (Coupled along X-Directon, Uncoupled along Y-Direction, per SS EN 1998-1, Clause 5.1.2)

2. Building Properties Used in Example

- Behaviour Factor: q = 1.5 (per *Para* 3.3*)
- Typical Office
  - Ψ_{2i} = 0.3 (per *Para* 4.3*)
  - Φ = 0.8 (storeys with correlated occupancies assumed, per *Para* 4.3*)
  - Ψ_{Ei} = 0.8 x 0.3 = 0.24
- Typical Basement (Traffic Areas assumed)
  - Ψ_{2i} = 0.6 (per *Para* 4.3*)
  - Φ = 1.0
  - Ψ_{Ei} = 0.6 x 1.0 = 0.6
- Roof
  - Ψ_{2i} = 0.3 (per *Para* 4.3*)
  - Φ = 1.0
  - Ψ_{Ei} = 0.3 x 1.0 = 0.3

3.3 Determining the behaviour factor q. The q factor depends on the structural system, regularity in elevation and plan, and ductility class. After accounting for any enhancements or reductions as per paragraphs 3.3.1 and 3.3.2, a **minimum value of 1.5 can be adopted for the q factor** in determining the design seismic action **for all building types** (i.e. concrete, steel and composite steel-concrete structures).

4.3 Storey weight, W_i. The storey weight, W_i at storey i, taken when calculating the seismic actions should comprise the full permanent (or dead) plus the variable (or imposed) load multiplied by a factor Ψ_{Ei}. Clause 4.2.4(2)P of SS EN 1998-1 quantifies Ψ_{Ei} as the factor Ψ_{2i} multiplied by a reduction factor φ that allows for the incomplete coupling between the structure and its imposed load.

$W_{i\,(\text{at storey } i)}$ = dead load + superimposed dead load + (Ψ_{Ei} . imposed load)

where $\Psi_{Ei} = \Psi_{2i}.\varphi$

BC3: 2013

BC3: 2013

Category of use	Specific use	Ψ_{2i}	φ		
			Roof	Storeys with correlated occupancies	Independently occupied storeys
A	domestic, residential (eg. rooms in residential buildings and houses; bedrooms and wards in hospitals; bedroom in hotels and hotel kitchens and toilets)	0.3	1.0	0.8	0.5
B	offices	0.3	1.0	0.8	0.5
F	traffic areas (vehicle weight ≤ 30kN)	0.6	1.0		

3. Evaluation of Structural Regularity (Para* 3.3.1)

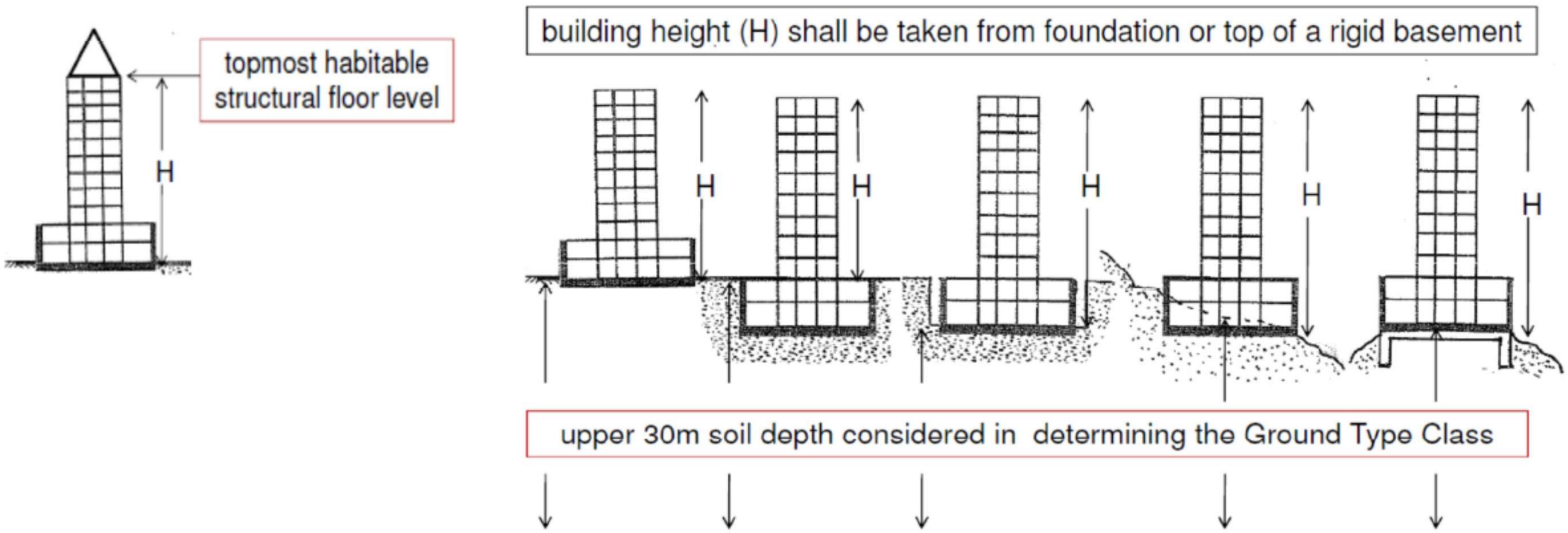
From inspection, the structure for the design example building can be categorized as being regular in both elevation and in plan.

If a structure is classified as irregular in plan, a three-dimensional structural model analysis is necessary, per Clause 4.2.2.1 (3) of SS EN 1998-1.

4. Establishment of Basic Parameters (Paras* 2 and 4.4.3)

- Identify the Ground Type for your site per *Paras* 2.1 to 2.4*.
  - For this example building, the ground type is D.
- Determine the height of the building per *Para* 2.1*.
  - For this example, the height is 25x4 = 100m
- Determine the fundamental period of the building, T_1, using conventional analysis software or any of the appropriate methods in SS EN 1998-1, Clause 4.3.3.2.2. (Note that if the height of the building is less than 40m, SS EN 1998-1 Clause 4.3.3.2.2 (3) could be used).
  - T_1 = 3.3 sec from computer analysis
- Determine the base shear percentage per Para* 4.4.3
  - $F_b = (S_d/g).W.\lambda$ (*where g is the gravitational constant* = $9.81\ m/s^2$)
  - $S_d = S_e.\gamma/q$ = 5.5%g x 1.0 / 1.5 = 3.7%g
  - λ = 1.0 (*Para* 4.4.3*, $T_1 > 2T_c$)
  - F_b = 3.7%.W.1.0 = 3.7%W

2.1 The building height, H shall be taken from the foundation or top of a rigid basement to the topmost habitable structural floor level[E], as shown in Figure below



BC3: 2013

2.2 The Ground Type within the footprint of structurally independent building shall be determined firstly by computing the value of P using either soil parameter of shear wave velocity ($v_{s,30}$), standard penetration test ($N_{SPT(blows/300mm)}$) or undrained shear strength (c_u) in the upper 30m soil depth as:

$$P = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{P_i}}$$

where $\sum_{i=1}^{n} d_i$ is equal to 30m;

P_i is the soil parameter ($v_{s,30}$, $N_{SPT(blows/300mm)}$ or c_u); and

d_i is the thickness of layer i between 0 and 30m.

2.3 The computed value of P is then used to determine the Ground Type from Table 1 below.

Value of P as computed from paragraph 2.2 for soil in the upper 30m			Ground Type	Description of stratigraphic profile
Shear-Wave Velocity. $v_{s,30}$ (m/s)	**N_{SPT} (blows/30cm)**	**Undrained Shear Strength, c_u (kPa)**		
> 800	Not applicable	Not applicable	**A**	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.
360 - 800	> 50	> 250	**B**	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.
180 - 360	15 - 50	70 - 250	**C**	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.
< 180	< 15	< 70	**D**	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soils.
< 100	< 5	10 - 20	**S_1**	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content.

Table 1 – Determining Ground Type from computed value of P

2.4 In determining the Ground Type,

(a) the top 30m soil depth is taken from the existing ground level even if the development requires excavations for basement construction;

(b) if more than one of the 3 soil parameters mentioned in table above are available, the most onerous Ground Type determined from these soil parameters shall be adopted;

(c) the most onerous Ground Type shall be adopted if there are different Ground Types spatially distributed as determined from various site investigations within the footprint of a building; and

(d) these rules shall apply regardless of whether the building is founded on piles that extend to hard soil stratum or not.

T (sec)	Spectral Acceleration $S_e(T)$ (%g)	T (sec)	Spectral Acceleration $S_e(T)$ (%g)
0.0	4.50	1.8	10.00
0.1	5.25	2.0	9.00
0.2	6.00	2.2	8.18
0.3	6.75	2.4	7.50
0.4	7.50	2.7	6.67
0.5	8.25	3.0	6.00
0.6	9.00	3.5	5.14
0.7	9.75	4.0	4.50
0.8	10.50	4.6	3.91
0.9	11.25	5.2	3.06
1.0	11.25	6.0	2.30
1.1	11.25	7.0	1.69
1.2	11.25	8.0	1.29
1.4	11.25	9.0	1.02
1.6	11.25	10.0	0.83

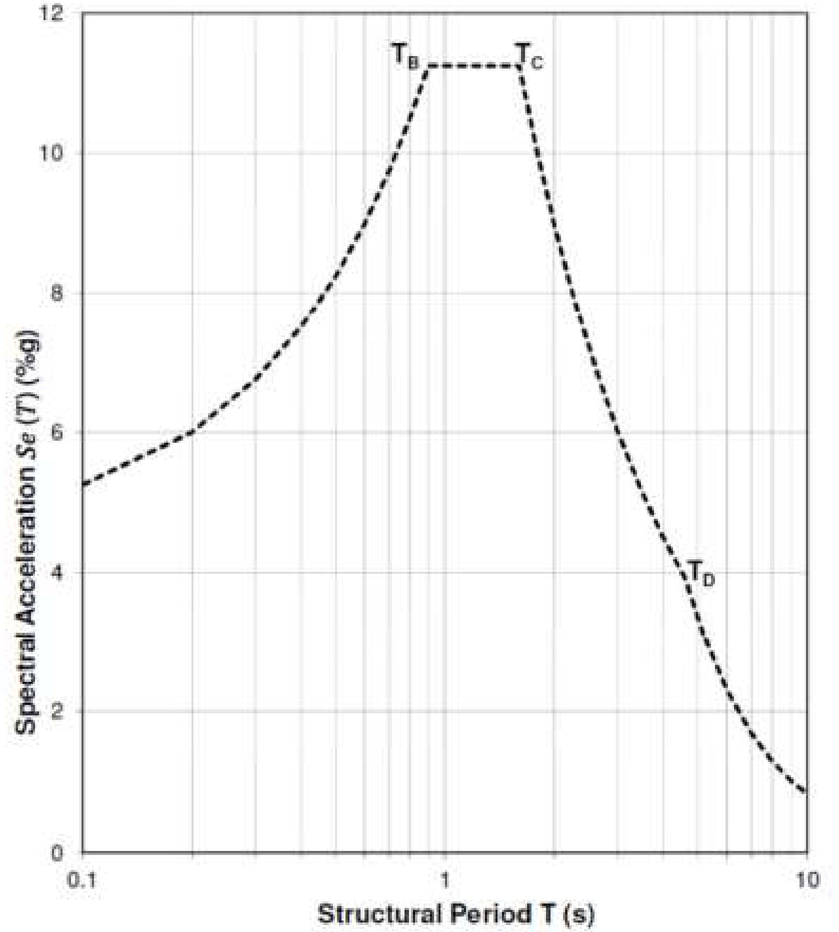


Figure 4 - Spectral accelerations, $S_e(T)$, for **Ground Type D** at 5% structural damping

BC3: 2013

4.4.3 Derivation of base shear due to Seismic Action

- Estimate natural period of building, T_{1x} and T_{1y}, in two main directions based on any of the appropriate equations in Clause 4.3.3.2.2 of SS EN 1998-1;
- Determine $S_d(T_{1x})$, and $S_d(T_{1y})$, which are derived from the equation in paragraph 3.2;
- Determine λ, the correction factor (refer to Clause 4.3.3.2.2 of SS EN 1998-1) which is equal to 0.85 (T_1 has to be less than $2T_C$); and
- Base shear force in the two main directions:

$F_{b,x} = \frac{S_d(T_{1x})}{g} . W. \lambda$ and

$F_{b,y} = \frac{S_d(T_{1y})}{g} . W. \lambda$

where, W is the total weight of the building (refer to paragraph 4.4.2) and g is the gravitational constant = 9.81m/s^2.

5. Storey Weight (Para* 4.3)

Determine the floor by floor weights of the tower. This is shown in Figure below. Per Para* 2.1, basement weights need not be considered. The seismic weight of the building is thus 582,973 kN.

Floor Weight Tabulation of Building Example

Storey	Elevation (m)	Flr to Flr (m)	Floor Area (m^2)	Cols (m^3)	Core (m^3)	Slab (kPa)	SDL (kPa)	LL (kPa)	Φ	Ψ	LL Dyn Factor	Slab Weight (kN)	SDL Weight (kN)	LL Weight (kN)	Core + Cols (kN)	Floor Dynamic Weight (Dead + SDL + LL_{dyn}) (kN)	Cummulative Weight (kN)
26	100	4	1664	64	192	7.05	2.5	3.5	0.3	1	0.3	11,731	4,160	5,824	6,016	23,654	23,654
25	96	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	46,959
24	92	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	70,264
23	88	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	93,569
22	84	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	116,874
21	80	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	140,179
20	76	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	163,484
19	72	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	186,789
18	68	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	210,094
17	64	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	233,399
16	60	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	256,704
15	56	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	280,009
14	52	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	303,314
13	48	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	326,619
12	44	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	349,924
11	40	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	373,229
10	36	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	396,534
9	32	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	419,839
8	28	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	443,144
7	24	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	466,449
6	20	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	489,754
5	16	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	513,059
4	12	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	536,364
3	8	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	559,668
2	4	4	1664	64	192	7.05	2.5	3.5	0.3	0.8	0.24	11,731	4,160	5,824	6,016	23,305	**582,973**
Ground	0	4	1664	64	192	7.05	2.5	3.5	0.6	1	0.6	11,731	4,160	5,824	6,016	25,402	608,375
B1	-4	4	1664	64	192	7.05	2.5	3.5	0.6	1	0.6	11,731	4,160	5,824	6,016	25,402	633,777
B2	-8	4	1664	64	192	7.05	2.5	3.5	0.6	1	0.6	11,731	4,160	5,824	6,016	25,402	659,178
B3	-12																
				1,792	5,376							328,474	116,480	163,072		659,178	9,484,178

6. Lateral Force Analysis Method (Para* 4.4.2)

Since the fundamental time period of the building is greater than 2.0 sec, per Para* 4.4.1, the Lateral Force Analysis method cannot be adopted for this building.

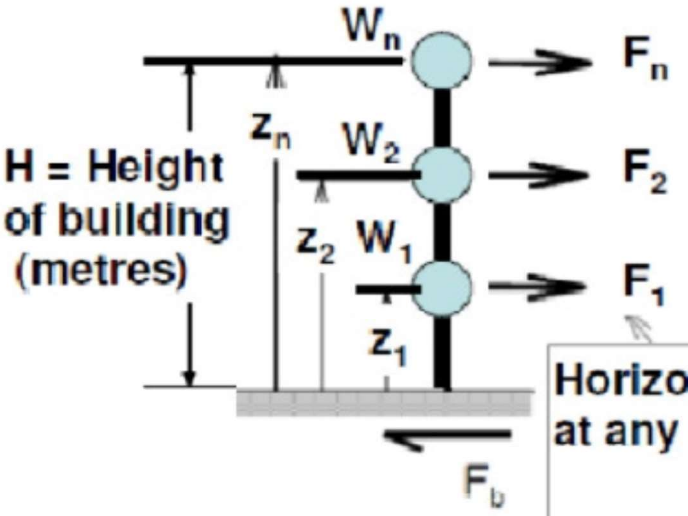
4.4 Lateral Force Analysis Method

BC3: 2013

4.4.1 The lateral force analysis method is only applicable to buildings with fundamental periods (T_1) of vibration in the two main directions smaller than 2.0s (refer to Clause 4.3.3.2.1(2)a) of SS EN 1998-1) and to buildings that are regular in elevation (refer to Clause 4.2.3.3 of SS EN 1998-1 for definition of regularity in elevation).

6. Lateral Force Analysis Method (Para* 4.4.2)

However, for an understanding of the steps necessary to carry out this procedure, the Lateral Force Analysis method is explained below for illustration purposes only.
After determining the storey and the total mass/weight of the building, the base shear can be distributed per the formulation in Para* 4.4.2.



W_i, W_j are the floor level Storey Weights determined as in paragraph 4.3;

z_i, z_j are the heights of the Storey Weights W_i, W_j above the level of application of the Seismic Action (foundation level or top of a rigid basement);

Horizontal force at any floor level i, $F_i = F_b \dfrac{W_i z_i}{\sum_{j=1}^{n} W_j z_j}$

$i-1$

F_b is the base shear due to Seismic Action

total storey weight of building W $= \sum_{i=1}^{n} W_i$

n is the number of storeys;

W is the total weight of the building

6. Lateral Force Analysis Method (Para* 4.4.2)

The distribution of lateral forces is computed. The base shear percentage has been determined above to be 3.7%.

Thus the design base shear is (3.7% x 582,973) = 21,570 kN.

It is checked that the sum of the computed lateral force is equal to the initial computation of base shear.

Lateral Force Analysis Method- Lateral Force Distribution

Storey	Elevation (m)	Flr to Flr (m)	Cummulative Weight (kN)	(Floor Weight) x Elevation	(Weight x Z) / Sum(Weight x Z)	Lateral Force (kN)
26	100	4	23,654	2,365,440	0.08	1,682
25	96	4	46,959	2,237,276	0.07	1,591
24	92	4	70,264	2,144,056	0.07	1,525
23	88	4	93,569	2,050,836	0.07	1,458
22	84	4	116,874	1,957,617	0.06	1,392
21	80	4	140,179	1,864,397	0.06	1,326
20	76	4	163,484	1,771,177	0.06	1,260
19	72	4	186,789	1,677,957	0.06	1,193
18	68	4	210,094	1,584,737	0.05	1,127
17	64	4	233,399	1,491,517	0.05	1,061
16	60	4	256,704	1,398,298	0.05	994
15	56	4	280,009	1,305,078	0.04	928
14	52	4	303,314	1,211,858	0.04	862
13	48	4	326,619	1,118,638	0.04	796
12	44	4	349,924	1,025,418	0.03	729
11	40	4	373,229	932,198	0.03	663
10	36	4	396,534	838,979	0.03	597
9	32	4	419,839	745,759	0.02	530
8	28	4	443,144	652,539	0.02	464
7	24	4	466,449	559,319	0.02	398
6	20	4	489,754	466,099	0.02	331
5	16	4	513,059	372,879	0.01	265
4	12	4	536,364	279,660	0.01	199
3	8	4	559,668	186,440	0.01	133
2	4	4	**582,973**	93,220	0.00	66
Ground	0	4	608,375	0	0.00	0
B1	-4	4	633,777			0
B2	-8	4	659,178			0
B3	-12					
			9,484,178	30,331,392		21,570

6. Lateral Force Analysis Method (Para* 4.4.2)

The distributions of the storey shears and storey moments are computed

Storey	Elevation (m)	Flr to Flr (m)	Lateral Force (kN)	Storey Shear (kN)	Storey Moment (kN-m)
26	100	4	1,682		
25	96	4	1,591	1,682	6,729
24	92	4	1,525	3,273	19,821
23	88	4	1,458	4,798	39,013
22	84	4	1,392	6,256	64,039
21	80	4	1,326	7,649	94,633
20	76	4	1,260	8,974	130,530
19	72	4	1,193	10,234	171,466
18	68	4	1,127	11,427	217,175
17	64	4	1,061	12,554	267,392
16	60	4	994	13,615	321,851
15	56	4	928	14,609	380,288
14	52	4	862	15,537	442,438
13	48	4	796	16,399	508,035
12	44	4	729	17,195	576,813
11	40	4	663	17,924	648,509
10	36	4	597	18,587	722,856
9	32	4	530	19,183	799,590
8	28	4	464	19,714	878,445
7	24	4	398	20,178	959,157
6	20	4	331	20,576	1,041,459
5	16	4	265	20,907	1,125,088
4	12	4	199	21,172	1,209,777
3	8	4	133	21,371	1,295,261
2	4	4	66	21,504	1,381,276
Ground	0	4	0	21,570	1,467,556
B1	-4	4	0	21,570	1,553,836
B2	-8	4	0	21,570	1,640,117
B3	-12			21,570	1,726,397
			21,570		

6. Lateral Force Analysis Method (Para* 4.4.2)

In addition to lateral forces, *Para* 5.3* requires a consideration for accidental torsion effects. The lateral force is required to be offset 0.05 the horizontal dimension of the floor plate. These force offsets can be applied as a point torque at the center of mass of each level fo analysis purposes.

For this example building, the X dimension is 52m and the Y dimension is 32m. The offset dimension is thus 0.05x52 = 2.6m and 0.05x32 = 1.6m respectively.

These computations are shown in the Table.

Lateral Force Analysis Method- Accidental Torsion Effects *(Para* 5.3)*

Storey	Elevation (m)	Flr to Flr (m)	X Building Dimension (m)	Y Building Dimension (m)	X Dimension to Offset Force (m)	Y Dimension to Offset Force (m)	X Torque (kN-m)	Y Torque (kN-m)
26	100	4	52	32	2.6	1.6	4,374	2,691
25	96	4	52	32	2.6	1.6	4,137	2,546
24	92	4	52	32	2.6	1.6	3,964	2,440
23	88	4	52	32	2.6	1.6	3,792	2,334
22	84	4	52	32	2.6	1.6	3,620	2,227
21	80	4	52	32	2.6	1.6	3,447	2,121
20	76	4	52	32	2.6	1.6	3,275	2,015
19	72	4	52	32	2.6	1.6	3,103	1,909
18	68	4	52	32	2.6	1.6	2,930	1,803
17	64	4	52	32	2.6	1.6	2,758	1,697
16	60	4	52	32	2.6	1.6	2,585	1,591
15	56	4	52	32	2.6	1.6	2,413	1,485
14	52	4	52	32	2.6	1.6	2,241	1,379
13	48	4	52	32	2.6	1.6	2,068	1,273
12	44	4	52	32	2.6	1.6	1,896	1,167
11	40	4	52	32	2.6	1.6	1,724	1,061
10	36	4	52	32	2.6	1.6	1,551	955
9	32	4	52	32	2.6	1.6	1,379	849
8	28	4	52	32	2.6	1.6	1,207	742
7	24	4	52	32	2.6	1.6	1,034	636
6	20	4	52	32	2.6	1.6	862	530
5	16	4	52	32	2.6	1.6	689	424
4	12	4	52	32	2.6	1.6	517	318
3	8	4	52	32	2.6	1.6	345	212
2	4	4	52	32	2.6	1.6	172	106
Ground	0	4	52	32	2.6	1.6	0	0
B1	-4	4						
B2	-8	4						
B3	-12							

5.3 Accidental torsional effects. In order to account for uncertainties in the location of masses and spatial variations of the Seismic Action, the calculated centre of mass at each floor level *i* shall be considered in the combination of actions in paragraph 5 as being displaced from its nominal location in each direction of analysis by an accidental eccentricity, $e_{\mathrm{ai}} = \pm 0.05\, L_i$

where	e_{ai}	*is the accidental eccentricity of storey mass i from its nominal location, taken in the same direction at all floor levels;*
	L_{i}	is the floor-dimension perpendicular to the direction of the Seismic Action.

6. Lateral Force Analysis Method (Para* 4.4.2)

The computed lateral force should be applied at the center of mass for the floor to appropriately capture any inherent torsional effects due to the differences between the center of mass and the center of rigidity.

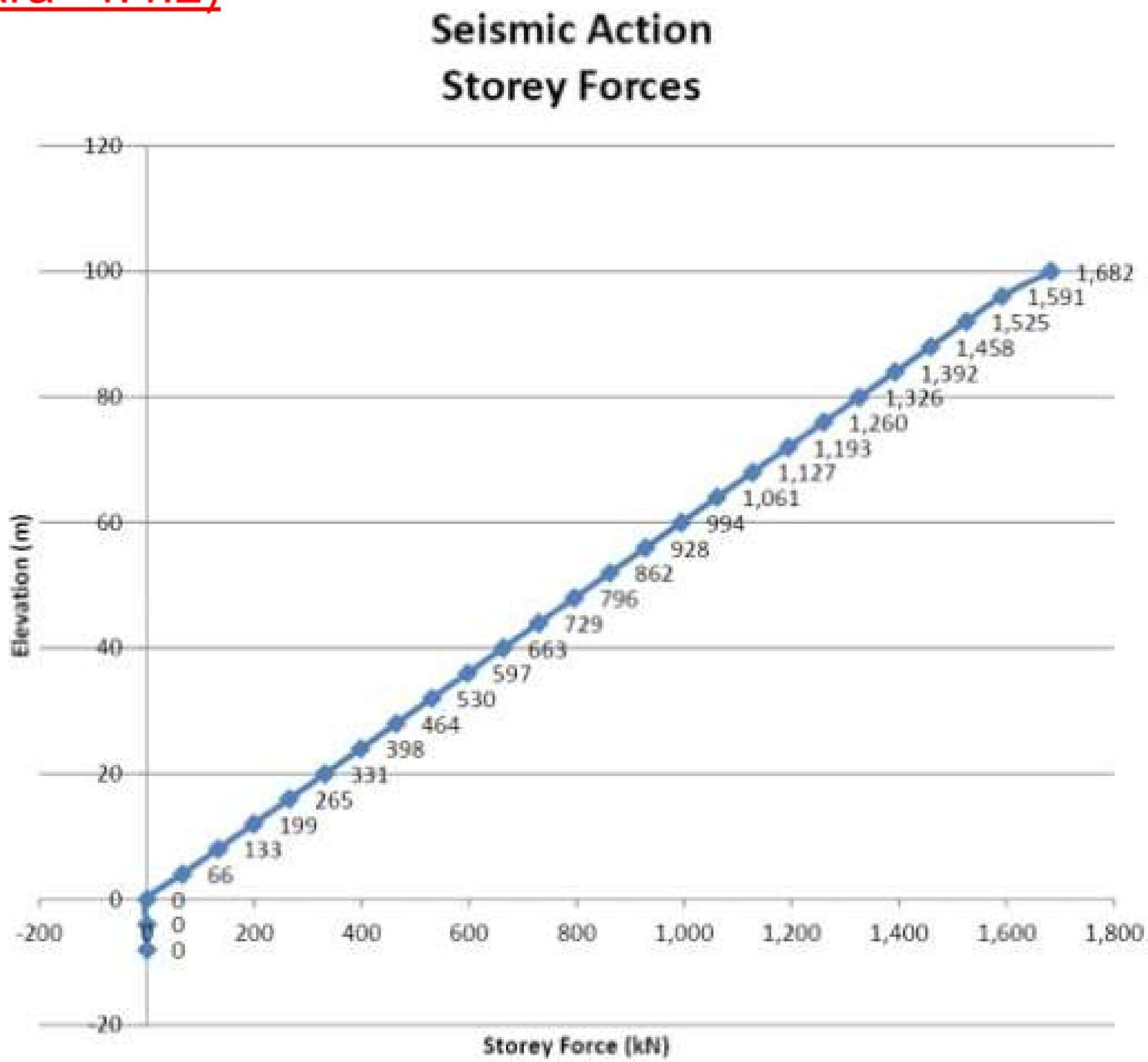
For completeness, the building story torques are computed and tabulated

Lateral Force Analysis Method-Building Torques

Storey	Elevation (m)	Flr to Flr (m)	X Torque (kN-m)	Y Torque (kN-m)	Storey Torque X (kN-m)	Storey Torqu (kN-m)
26	100	4	4,374	2,691		
25	96	4	4,137	2,546	4,374	2,691
24	92	4	3,964	2,440	8,510	5,237
23	88	4	3,792	2,334	12,475	7,677
22	84	4	3,620	2,227	16,267	10,010
21	80	4	3,447	2,121	19,886	12,238
20	76	4	3,275	2,015	23,333	14,359
19	72	4	3,103	1,909	26,608	16,374
18	68	4	2,930	1,803	29,711	18,284
17	64	4	2,758	1,697	32,641	20,087
16	60	4	2,585	1,591	35,399	21,784
15	56	4	2,413	1,485	37,984	23,375
14	52	4	2,241	1,379	40,397	24,860
13	48	4	2,068	1,273	42,638	26,239
12	44	4	1,896	1,167	44,706	27,512
11	40	4	1,724	1,061	46,602	28,678
10	36	4	1,551	955	48,326	29,739
9	32	4	1,379	849	49,877	30,694
8	28	4	1,207	742	51,256	31,542
7	24	4	1,034	636	52,462	32,285
6	20	4	862	530	53,497	32,921
5	16	4	689	424	54,358	33,451
4	12	4	517	318	55,048	33,876
3	8	4	345	212	55,565	34,194
2	4	4	172	106	55,910	34,406
Ground	0	4	0	0	56,082	34,512
B1	-4	4			56,082	34,512
B2	-8	4			56,082	34,512
B3	-12				56,082	34,512

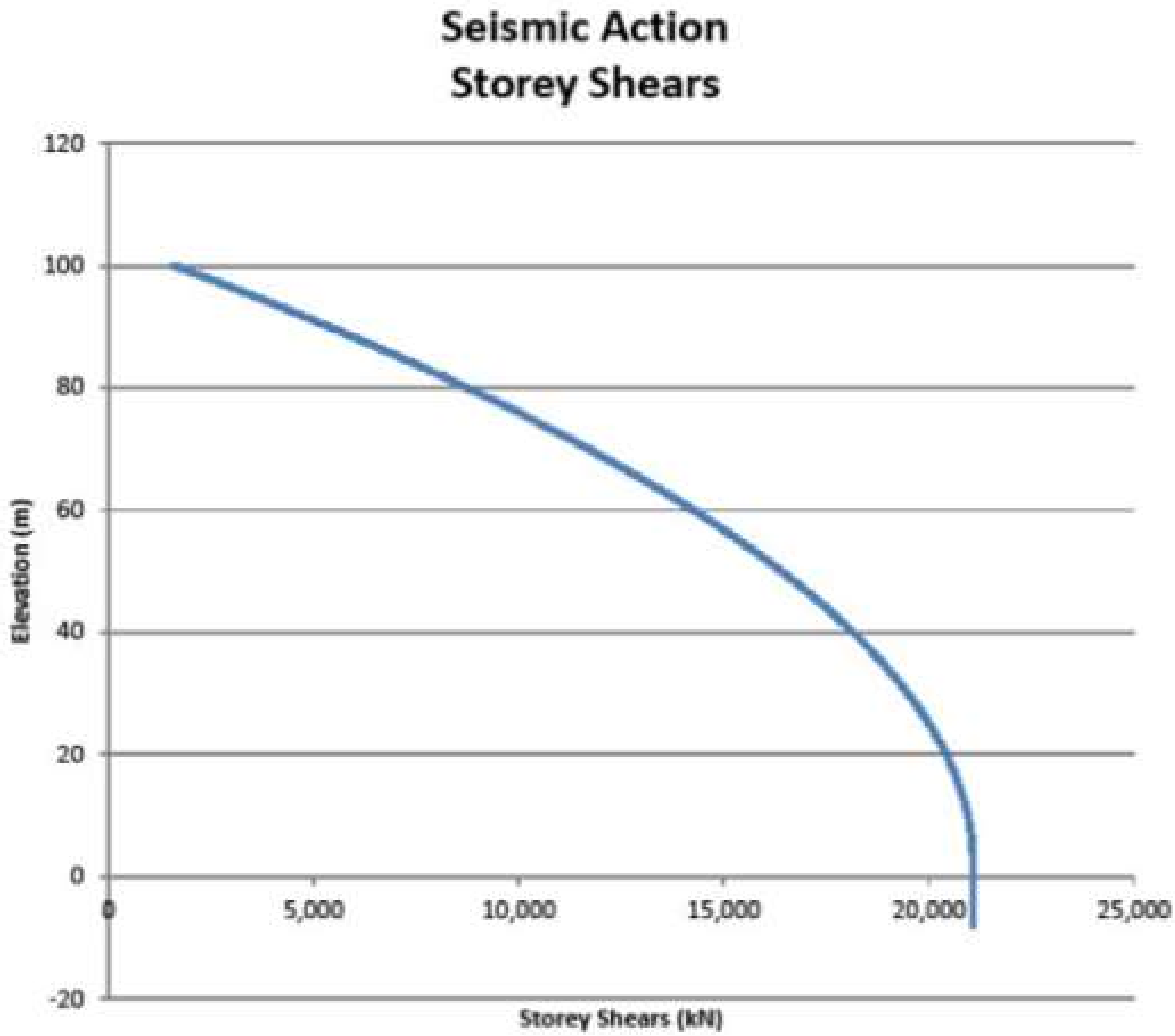
6. Lateral Force Analysis Method (Para* 4.4.2)

Lateral Force Analysis Method- Story Forces and Shears



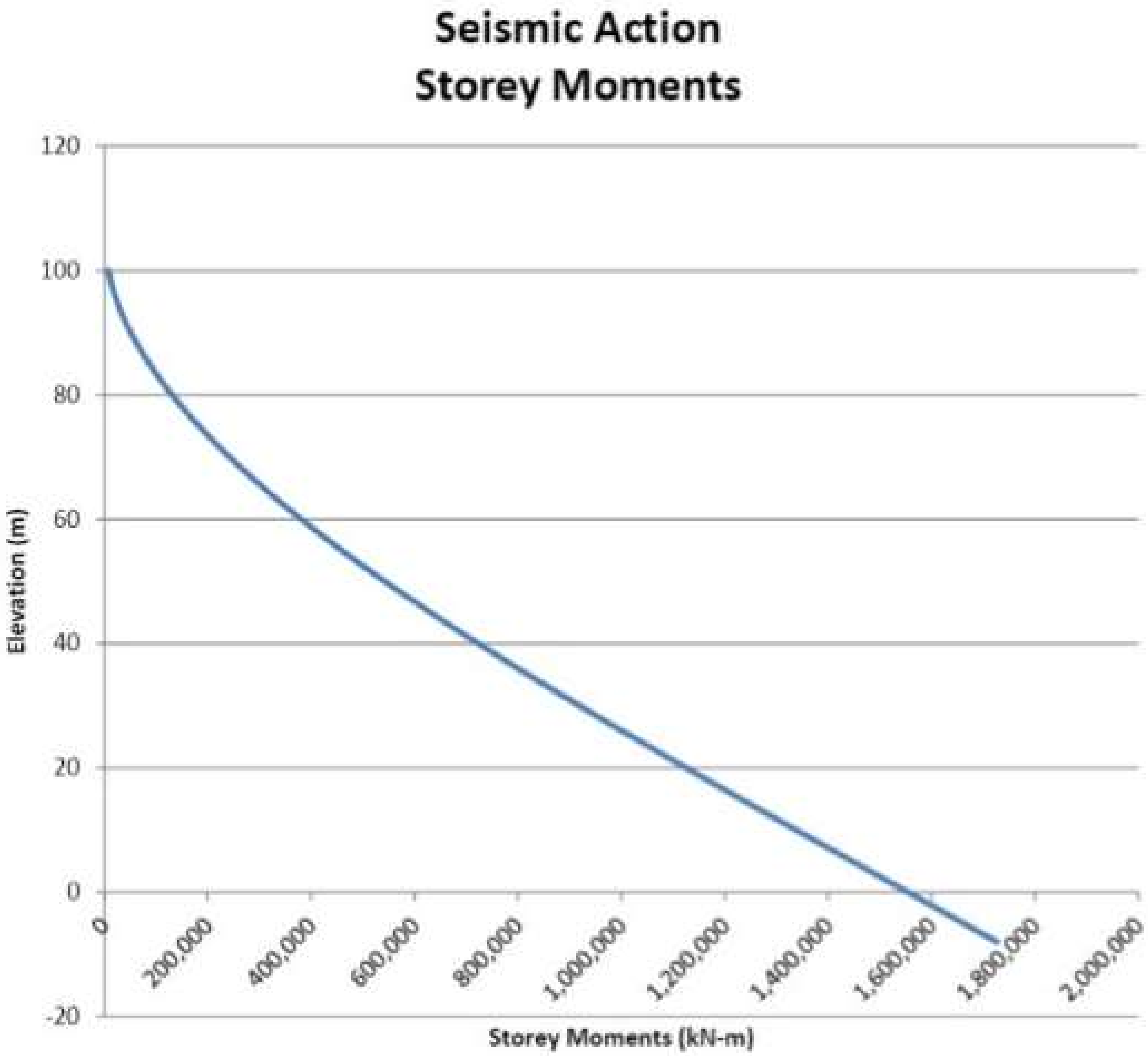
6. Lateral Force Analysis Method (Para* 4.4.2)

Lateral Force Analysis Method- Story Forces and Shears



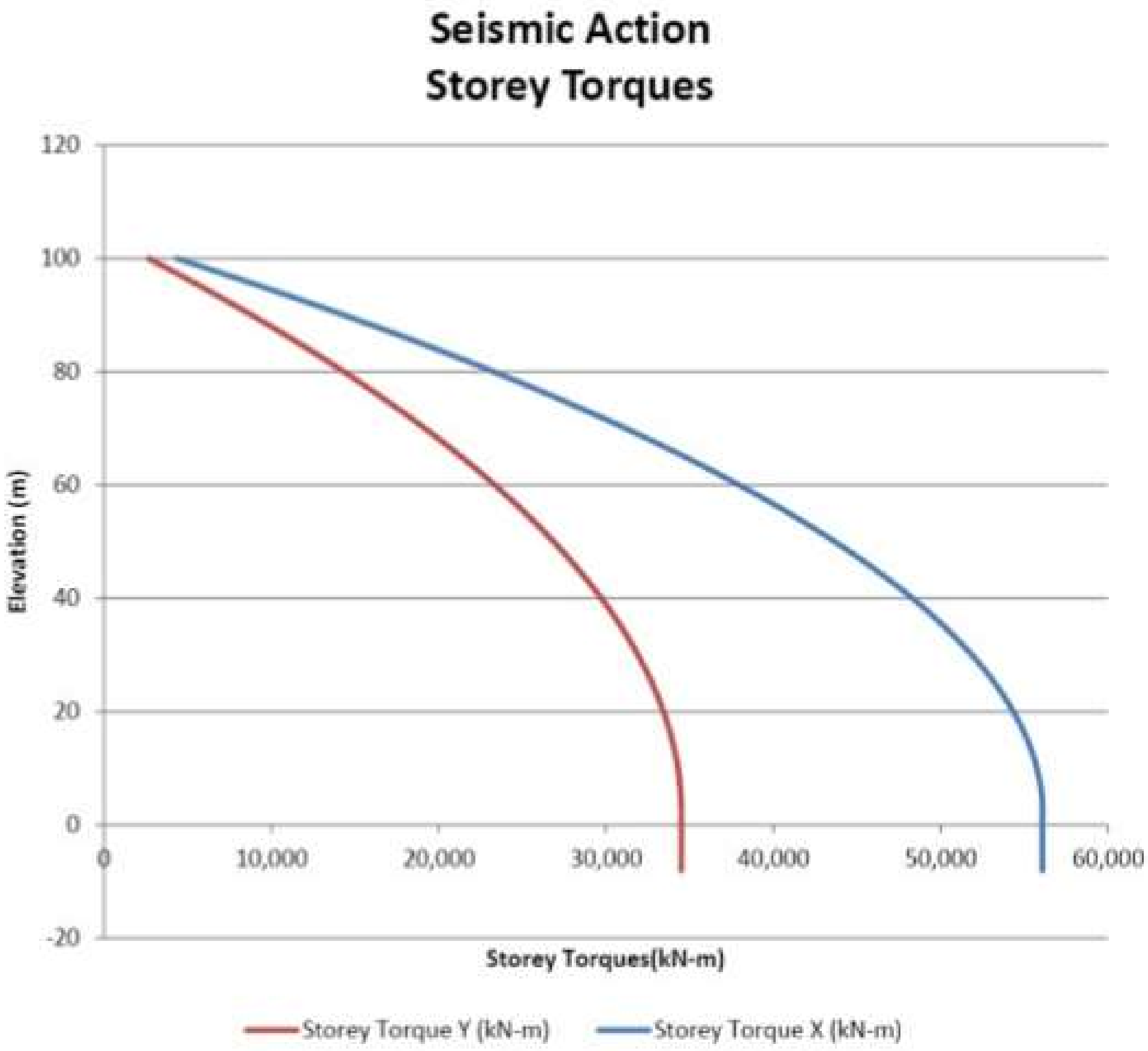
6. Lateral Force Analysis Method (Para* 4.4.2)

Lateral Force Analysis Method-Story Moments and Torques



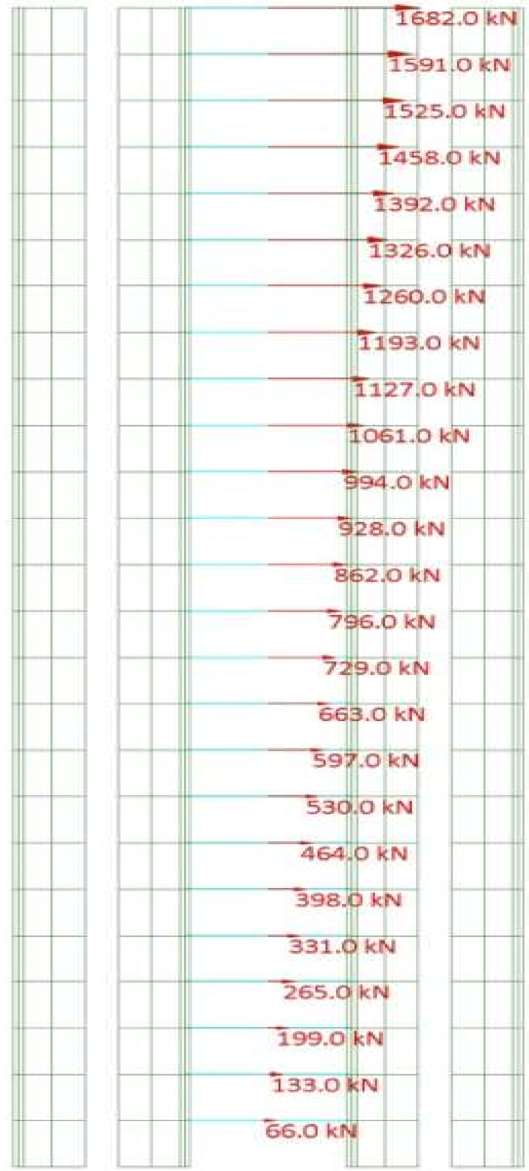
6. Lateral Force Analysis Method (Para* 4.4.2)

Lateral Force Analysis Method- Story Moments and Torques



6. Lateral Force Analysis Method (Para* 4.4.2)

Hence, the lateral force distribution profile along the building height for this example is shown



7. Modal Response Spectrum Analysis Method (Para* 4.5)

Alternatively, a more rigorous dynamic analysis approach is to more accurately capture the vertical distribution of forces along the height of the building.

The steps for a dynamic analysis are summarized below.

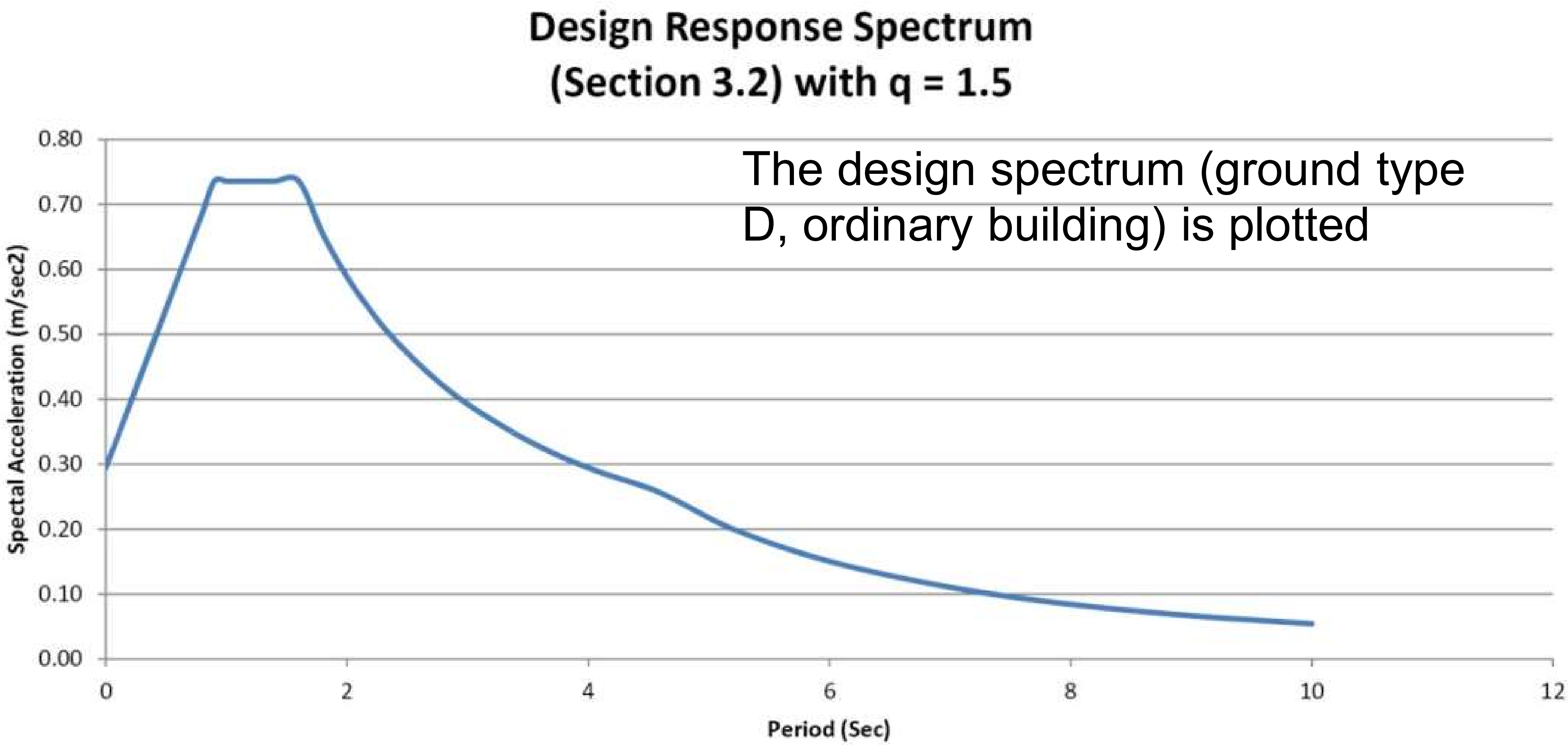
1. Solve for the building's period and mode shapes.

2. Ensure sufficient modes are used in the dynamic analysis by inspecting the cumulative modal participation.

3. Determine base shears obtained through response spectrum in each direction under consideration.

7. Modal Response Spectrum Analysis Method (Para* 4.5)

Determine Design Spectrum (Para* 3.2)



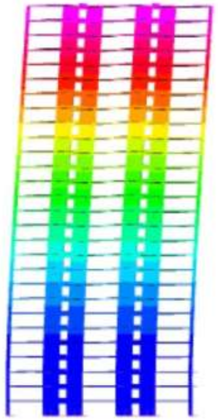
The design spectrum (ground type D, ordinary building) is plotted

Design Spectrum (m/sec^2)

7. Modal Response Spectrum Analysis Method (Para* 4.5)

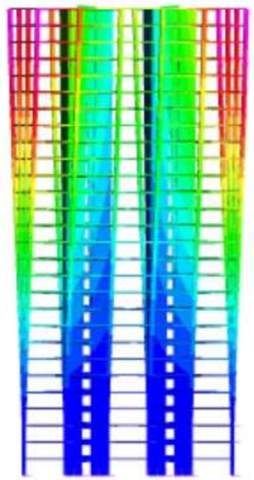
The above design spectrum can be entered into any commercial analysis software capable of free vibration and response spectrum analysis (e.g. ETABS).

For the example building, the first three time periods and mode shapes are shown

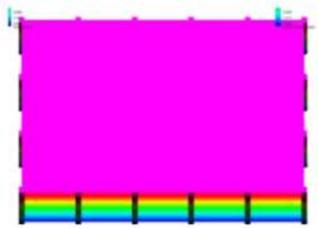


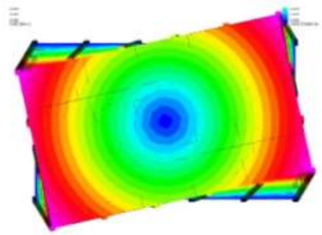
Mode 1
T = 3.3 sec

Mode 2
T = 3.3 sec



Mode 3
T = 2.0 sec





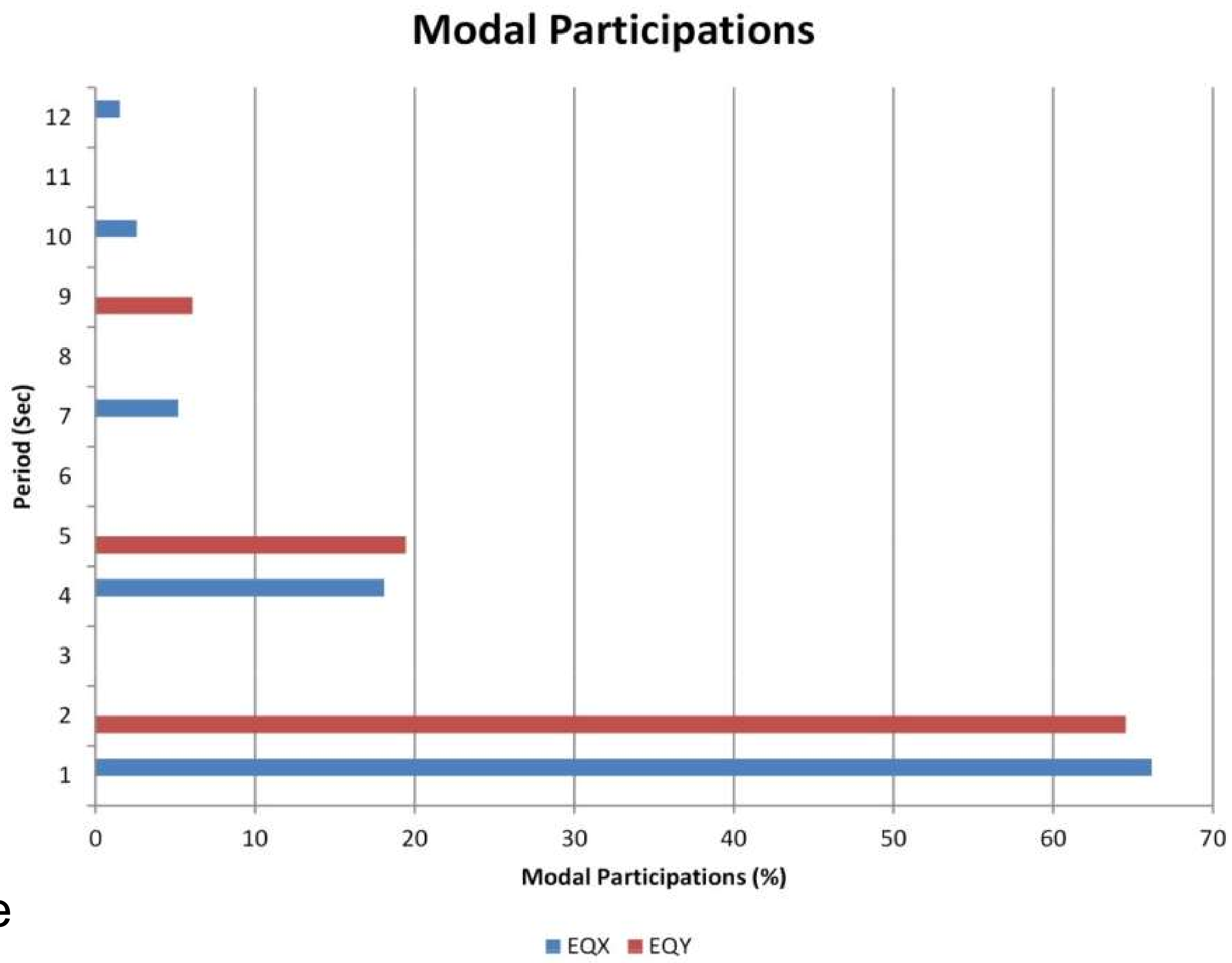
7. Modal Response Spectrum Analysis Method (Para* 4.5)

Modal Response Spectrum Analysis

A response spectrum analysis is then run in two orthogonal directions with a scale factor of 1.

Sufficient building modes should be used to ensure sufficient modal mass is activated. 90% mass participation is assumed as sufficient per SS EN 1998-1 Clause 4.3.3.3.1-3.

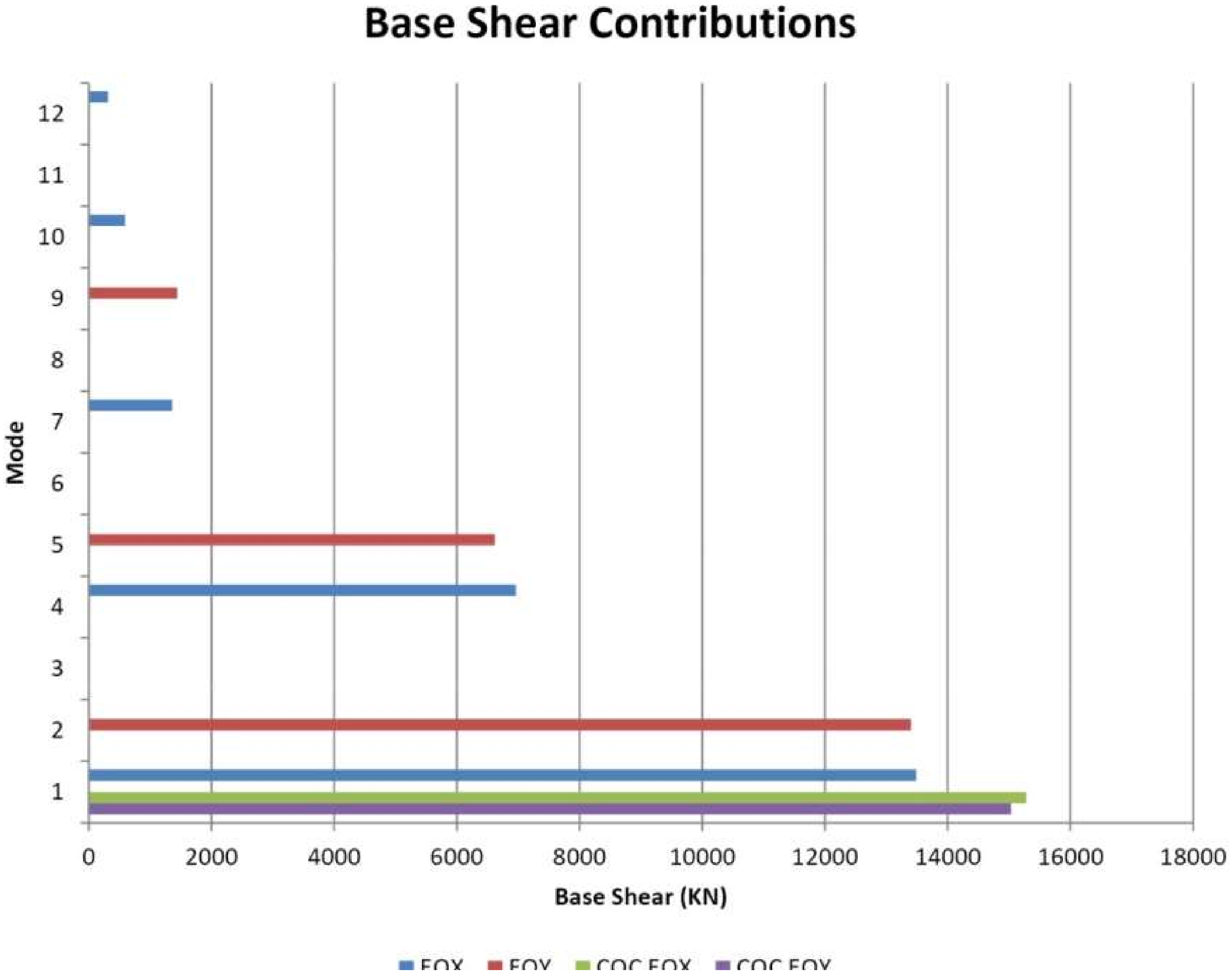
The modal participations for the example building is shown



7. Modal Response Spectrum Analysis Method (Para* 4.5)

The base shear contributions
for the selected modes
is shown in this Figure

The cumulative mass participations for the first 12 modes are 92% and 90% respectively.



Modal Base Shears

4.5 Modal Response Spectrum Analysis Method

4.5.1 Derivation of Seismic Action using Modal Response Spectrum Analysis Method. The design spectrum, $S_d(T)$, derived from paragraph 3.2 shall be used as input directly into any conventional structural analysis software as the lateral Seismic Action when carrying out dynamic analysis using the modal response spectrum method.

4.5.2 Refer to Clause 4.3.3.3.1 2(P) & 3 [M] of SS EN 1998-1 for specific requirements to be satisfied when using the modal response spectrum method.

[M] The response of all modes of vibration contributing significantly to the global response shall be taken into account. This requirement is deemed to be satisfied if either of the following can be demonstrated:

- the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;
- all modes with effective modal masses greater than 5% of the total mass are taken into account.

7. Modal Response Spectrum Analysis Method (Para* 4.5)

After confirming that sufficient modes have been utilized in the analysis, the modal base shears can be extracted and combined. Modal combinations for each seismic direction can be performed by any accepted method such as Complete Quadratic Combination (CQC) or Square Root Sum of Squares (SRSS). For this example building, the modal responses are combined using the CQC method assuming a modal damping of 5%.

For directional effects, per *Para* 5.2*, it is permitted to combine the two directions using the 100% - 30% rule or a SRSS combination of the two analyzed directions (X and Y). For this design example, the former approach has been adopted.

Similar to the Lateral Force Analysis Method, consideration for accidental torsion effects, per *Para* 5.3,* need to be considered in the Modal Response Spectrum Analysis Method. This is analytically addressed by entering a mass offset in any commercial structural analysis software which then calculates and adds the torsional response to the response spectrum output to account for the additional design forces caused by accidental eccentricity.

8. Required Combinations of Actions (Load Combinations) (Para* 5.2)

Load Combinations per *Paras* 5.1 and 5.2,* using the 100% - 30% combination rule, are listed as shown.

A list of +/- combination of permutations is shown.

Load Combinations Considering Geometric Imperfection in X Direction

Load Combination Number	Dead	SDL	LL	Sesimic Action X	Sesimic Action Torque X	Sesimic Action Y	Sesimic Action Torque Y	Geometric Imperfection Effects (X direction)	Geometric Imperfection Effects (Y direction)
1	1.00	1.00	0.24	1.00	1.00	0.30	0.30	1.00	0.00
2	1.00	1.00	0.24	1.00	-1.00	0.30	0.30	1.00	0.00
3	1.00	1.00	0.24	-1.00	1.00	0.30	0.30	1.00	0.00
4	1.00	1.00	0.24	-1.00	-1.00	0.30	0.30	1.00	0.00
5	1.00	1.00	0.24	1.00	1.00	-0.30	0.30	1.00	0.00
6	1.00	1.00	0.24	1.00	-1.00	0.30	-0.30	1.00	0.00
7	1.00	1.00	0.24	-1.00	1.00	-0.30	0.30	1.00	0.00
8	1.00	1.00	0.24	-1.00	-1.00	0.30	-0.30	1.00	0.00
9	1.00	1.00	0.24	0.30	0.30	1.00	1.00	1.00	0.00
10	1.00	1.00	0.24	0.30	0.30	1.00	-1.00	1.00	0.00
11	1.00	1.00	0.24	0.30	0.30	-1.00	1.00	1.00	0.00
12	1.00	1.00	0.24	0.30	0.30	-1.00	-1.00	1.00	0.00
13	1.00	1.00	0.24	-0.30	0.30	1.00	1.00	1.00	0.00
14	1.00	1.00	0.24	0.30	-0.30	1.00	-1.00	1.00	0.00
15	1.00	1.00	0.24	-0.30	0.30	-1.00	1.00	1.00	0.00
16	1.00	1.00	0.24	0.30	-0.30	-1.00	-1.00	1.00	0.00
17	1.00	1.00	0.24	1.00	1.00	0.30	0.30	-1.00	0.00
18	1.00	1.00	0.24	1.00	-1.00	0.30	0.30	-1.00	0.00
19	1.00	1.00	0.24	-1.00	1.00	0.30	0.30	-1.00	0.00
20	1.00	1.00	0.24	-1.00	-1.00	0.30	0.30	-1.00	0.00
21	1.00	1.00	0.24	1.00	1.00	-0.30	0.30	-1.00	0.00
22	1.00	1.00	0.24	1.00	-1.00	0.30	-0.30	-1.00	0.00
23	1.00	1.00	0.24	-1.00	1.00	-0.30	0.30	-1.00	0.00
24	1.00	1.00	0.24	-1.00	-1.00	0.30	-0.30	-1.00	0.00
25	1.00	1.00	0.24	0.30	0.30	1.00	1.00	-1.00	0.00
26	1.00	1.00	0.24	0.30	0.30	1.00	-1.00	-1.00	0.00
27	1.00	1.00	0.24	0.30	0.30	-1.00	1.00	-1.00	0.00
28	1.00	1.00	0.24	0.30	0.30	-1.00	-1.00	-1.00	0.00
29	1.00	1.00	0.24	-0.30	0.30	1.00	1.00	-1.00	0.00
30	1.00	1.00	0.24	0.30	-0.30	1.00	-1.00	-1.00	0.00
31	1.00	1.00	0.24	-0.30	0.30	-1.00	1.00	-1.00	0.00
32	1.00	1.00	0.24	0.30	-0.30	-1.00	-1.00	-1.00	0.00

8. Required Combinations of Actions (Load Combinations) (Para* 5.2)

Load Combinations Considering Geometric Imperfection in Y Direction

Paras 5.1 and 5.2* gives rise to a total of 64 combinations.

However, engineering judgment can be applied to reduce the total of required combinations.

Load Combination Number	Dead	SDL	LL	Sesimic Action X	Sesimic Action Torque X	Sesimic Action Y	Sesimic Action Torque Y	Geometric Imperfection Effects (X direction)	Geometric Imperfection Effects (Y direction)
33	1.00	1.00	0.24	1.00	1.00	0.30	0.30	0.00	1.00
34	1.00	1.00	0.24	1.00	-1.00	0.30	0.30	0.00	1.00
35	1.00	1.00	0.24	-1.00	1.00	0.30	0.30	0.00	1.00
36	1.00	1.00	0.24	-1.00	-1.00	0.30	0.30	0.00	1.00
37	1.00	1.00	0.24	1.00	1.00	-0.30	0.30	0.00	1.00
38	1.00	1.00	0.24	1.00	-1.00	0.30	-0.30	0.00	1.00
39	1.00	1.00	0.24	-1.00	1.00	-0.30	0.30	0.00	1.00
40	1.00	1.00	0.24	-1.00	-1.00	0.30	-0.30	0.00	1.00
41	1.00	1.00	0.24	0.30	0.30	1.00	1.00	0.00	1.00
42	1.00	1.00	0.24	0.30	0.30	1.00	-1.00	0.00	1.00
43	1.00	1.00	0.24	0.30	0.30	-1.00	1.00	0.00	1.00
44	1.00	1.00	0.24	0.30	0.30	-1.00	-1.00	0.00	1.00
45	1.00	1.00	0.24	-0.30	0.30	1.00	1.00	0.00	1.00
46	1.00	1.00	0.24	0.30	-0.30	1.00	-1.00	0.00	1.00
47	1.00	1.00	0.24	-0.30	0.30	-1.00	1.00	0.00	1.00
48	1.00	1.00	0.24	0.30	-0.30	-1.00	-1.00	0.00	1.00
49	1.00	1.00	0.24	1.00	1.00	0.30	0.30	0.00	-1.00
50	1.00	1.00	0.24	1.00	-1.00	0.30	0.30	0.00	-1.00
51	1.00	1.00	0.24	-1.00	1.00	0.30	0.30	0.00	-1.00
52	1.00	1.00	0.24	-1.00	-1.00	0.30	0.30	0.00	-1.00
53	1.00	1.00	0.24	1.00	1.00	-0.30	0.30	0.00	-1.00
54	1.00	1.00	0.24	1.00	-1.00	0.30	-0.30	0.00	-1.00
55	1.00	1.00	0.24	-1.00	1.00	-0.30	0.30	0.00	-1.00
56	1.00	1.00	0.24	-1.00	-1.00	0.30	-0.30	0.00	-1.00
57	1.00	1.00	0.24	0.30	0.30	1.00	1.00	0.00	-1.00
58	1.00	1.00	0.24	0.30	0.30	1.00	-1.00	0.00	-1.00
59	1.00	1.00	0.24	0.30	0.30	-1.00	1.00	0.00	-1.00
60	1.00	1.00	0.24	0.30	0.30	-1.00	-1.00	0.00	-1.00
61	1.00	1.00	0.24	-0.30	0.30	1.00	1.00	0.00	-1.00
62	1.00	1.00	0.24	0.30	-0.30	1.00	-1.00	0.00	-1.00
63	1.00	1.00	0.24	-0.30	0.30	-1.00	1.00	0.00	-1.00
64	1.00	1.00	0.24	0.30	-0.30	-1.00	-1.00	0.00	-1.00

5.1 The Seismic Action (determined in paragraph 4 using either the lateral force method or the modal response spectrum method) shall be applied at the centre of mass of each floor and the building shall be evaluated for the following Combination of Actions at **Ultimate Limit State (ULS)**:

Combination of Actions at ULS = 1.0 x Storey Weight[N] ± 1.0 x Seismic Action[O] + 1.0 x Geometric Imperfection Effects[P]

5.2 Two Load Cases are to be considered for the Combination of Actions at ULS with 100% of the prescribed Seismic Action applied in one direction and 30% of the Seismic Action applied in the perpendicular direction as shown in Figure 7. Alternatively, the Square Root of the Sum of the Squared Values (SRSS) method, as per Clause 4.3.3.5.1(2)b)[Q] of SS EN 1998-1, may be used.

Figure 7 – Two load cases at Combination of Actions at ULS

[Q] The maximum value of each action effect on the structure due to the two horizontal components of the seismic action may be estimated by the square root of the sum of the squared values of the action effect due to each horizontal component (SRSS method).

9. Interstorey Drift Limitation – Modal Response Analysis Method (Para* 7.1)

Para 7.1* requires that the design inter-story drift, d_r, shall not exceed 0.005 / $v.q$ of the story height.

- v = 0.5 (Ordinary buildings)
- q = 1.5 (Reinforced Concrete structures, DCL)

where d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements, d_e [S] at the top and bottom of the storey under consideration;

v is the reduction factor which takes into account the lower return period of the Seismic Action associated with the damage limitation requirement (refer to Clause 4.4.3.2(2) in National Annex to SS EN 1998-1 on the value of factor "v" to be adopted, namely v = 0.5 for "ordinary buildings" and v = 0.4 for "special buildings");

h is the storey height; and

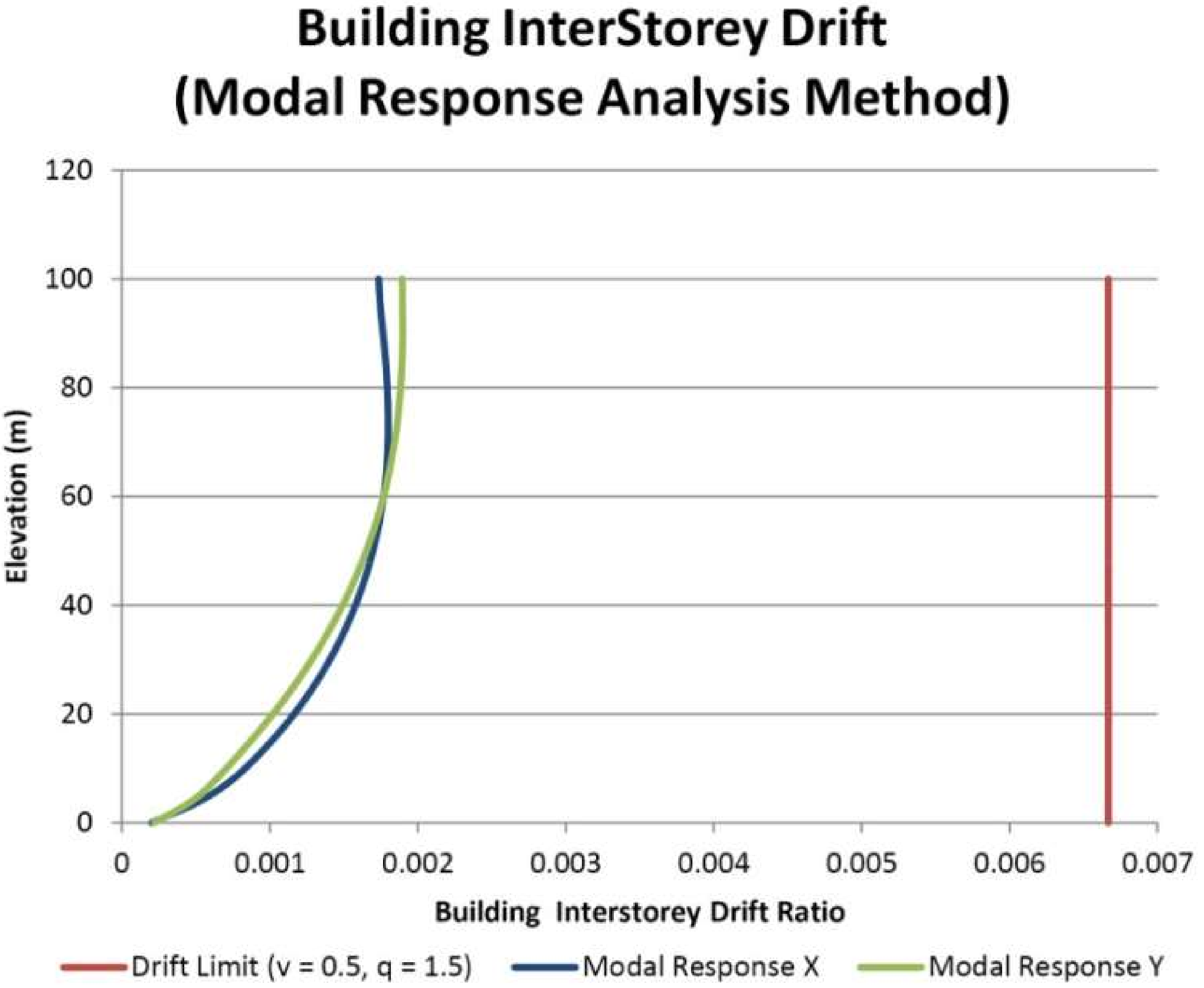
q is the behaviour factor (refer to paragraph 3.3).

[S] d_e is the displacement of the same point of the structural system, as determined by the analysis based on the design response spectrum (using the combination of actions in paragraph 5). of BC3 (2013)

9. Interstorey Drift Limitation – Modal Response Analysis Method (Para* 7.1)

The inter-storey drifts from the Modal Response Spectrum Analysis are plotted with the prescribed drift limit. It is seen that the structure drift is well within the stipulated drift limit.

For cases where the drift exceeds the stipulated limit, additional lateral load resisting elements may have to be introduced to the structural system or existing structural elements may have to be enlarged.



10. Separation from Property Line -Modal Response Analysis Method (Para*

BC3: 2013

8.1 Buildings shall have a minimum structural separation [T] as outlined in paragraphs 8.2 and 8.3 below.

8.2 The minimum structural separation for a new building A from the property boundary line (see Figure 8) <u>at each floor level</u> should be $\Delta_{A,}$ where Δ_A is the deflection of the building at that floor level determined from the structural analysis using the combination of actions in paragraph 5 multiplied by the behaviour factor q adopted based on paragraph 3.3. This minimum structural separation at each floor level should not be less than 0.1% of the height of that floor level measured from the foundation or the top of a rigid basement as defined in paragraph 2.1

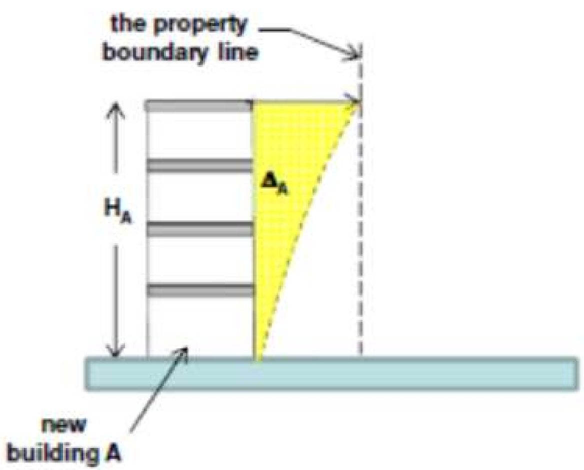


Figure 8 - Minimum structural separation from property boundary line

10. Separation from Property Line -Modal Response Analysis Method (Para* 8.1)

The required separation in both the X and Y Direction is evaluated separately. In this example, the maximum drifts at the top of the building from the Modal Response Spectrum Analysis are as follows:

EQ_X drift = 0.150 m

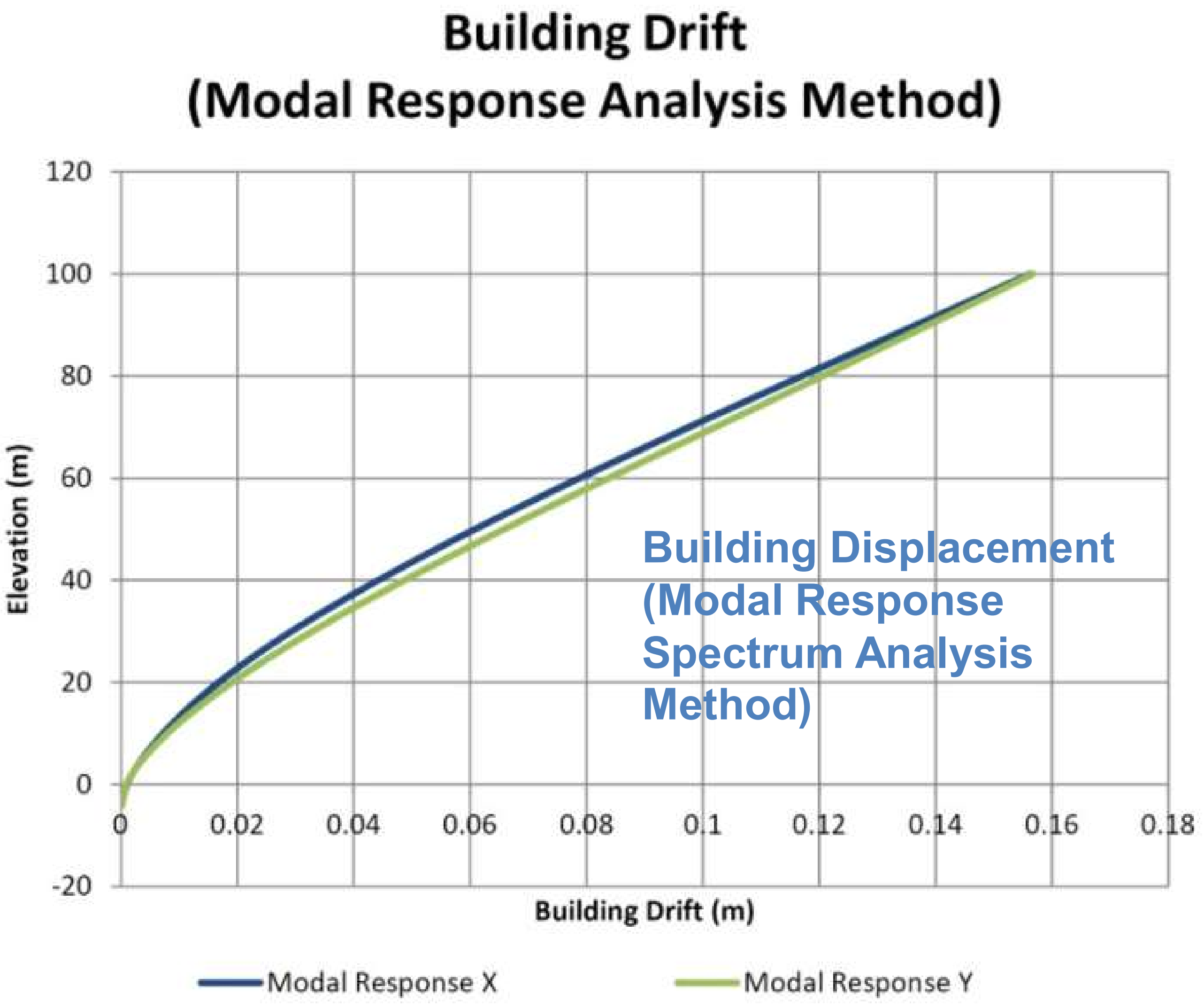
EQ_Y drift = 0.155 m

Required separations at the top of the building in both X and Y Directions are as follows:

Dx separation: EQx drift x *q* = 0.150 x 1.5 = 0.225 m

Dy separation: EQy drift x *q* = 0.155 x 1.5 = 0.233 m

Both these values are greater than the minimum limit at the top of the building which is 100mm (0.1% of the 100m building height above the basement).



11. Foundation Design (Para* 6)

As the action effects for the foundation for this design example have been determined for Ductility Class Low (DCL) with *q* = 1.5, the structure is categorized as a low-dissipative structure.

For low-dissipative structures, the reaction forces derived directly from the structural analyses can be used in the design of foundation elements, without the need for capacity design considerations accounting for the development of possible overstrength per SS EN 1998-1, Clause 4.4.2.6 (3).

The design of the foundation elements must ensure that the ultimate reaction forces from the structural analyses are less than the ultimate resistance of the foundation elements.

For example, for pile foundations resisting compression loads, if a column has an ULS load of 54,000 kN, and the ultimate geotechnical limit state design resistance of one pile, determined in accordance with SS EN 1997-1, Clause 7.6.3, is 9000 kN, then the total number of piles required under the column would be 6 numbers.

It is to be noted that foundation elements of structures designed for Ductility Classes other than Low, would require capacity design considerations in accordance with the requirements of SS EN 1998-1, Clause 4.4.2.6.

EC8 detailing of the longitudinal bars in primary beams (in secondary ones as in DC L)

	DC H	DC M	DC L
'critical region' length at member end	$1.5h$	h	
$\rho_{min} = A_{s,min}/bd$ at the tension side	$0.5 f_{ctm}/f_{yk}$ [a]		$0.26 f_{ctm}/f_{yk}$ [a], 0.13% [b]
$\rho_{max} = A_{s,max}/bd$ in critical regions [b]	$\rho' + 0.0018 f_{cd}/(\mu_{\varphi}\varepsilon_{yd}f_{yd})$ [c]		0.04
$A_{s,min}$, top and bottom bars	$2\Phi14$ (308 mm²)	–	
$A_{s,min}$, top bars in the span	$0.25A_{s,top\text{-}supports}$	–	
$A_{s,min}$, bottom bars in critical regions	$0.5A_{s,top}$ [d]		–
$A_{s,min}$, bottom bars at supports	$0.25A_{s,bottom\text{-}span}$ [b]		
Anchorage length for diameter d_{bL} [e]	$l_{bd} = a_{tr}[1-0.15(c_d/d_{bL}-1)](d_{bL}/4)f_{yd}/(2.25 f_{ctd} a_{poor})$ [f,g,h,i]		

[a] f_{ctm} (MPa) $= 0.3(f_{ck}(\mathrm{MPa}))^{2/3}$: 28-day, mean tensile strength of concrete; f_{yk} (MPa): nominal yield stress of longitudinal steel.
[b] NDP (nationally determined parameter) per EC2; the value recommended in EC2 is given here.
[c] ρ': Steel ratio at the opposite side of the section; μ_{φ}: curvature ductility factor corresponding via Equations 5.64 to the basic value of the behaviour factor, q_o, applicable to the design; $\varepsilon_{yd} = f_{yd}/E_s$.
[d] This $A_{s,min}$ is additional to the compression steel from the ULS verification of the end section in flexure under the extreme hogging moment from the analysis for the seismic design situation.
[e] Anchorage length in tension is reduced by 30% if the bar end extends by $\geq 5d_{bL}$ beyond a bend $\geq 90°$.
[f] c_d: Concrete cover of anchored bar, or one-half the clear spacing to the nearest parallel anchored bar, whichever is smaller.
[g] $a_{tr} = 1 - k(n_w A_{sw} - A_{s,t,min})/A_s \geq 0.7$, with A_{sw}: Cross-sectional area of tie leg within the cover of the anchored bar; n_w: number of such tie legs over the length l_{bd}; $k = 0.1$ if the bar is at a corner of a hoop or tie, $k = 0.05$ otherwise; $A_s = \pi d_{bL}^2/4$ and $A_{s,t,min}$ is specified in EC2 as equal to $0.25A_s$.
[h] $f_{ctd} = f_{ctk,0.05}/\gamma_c = 0.7 f_{ctm}/\gamma_c = 0.21 f_{ck}^{2/3}/\gamma_c$: Design value of 5%-fractile tensile strength of concrete.
[i] $a_{poor} = 1.0$ if the bar is in the bottom 0.25 m of the beam depth, or (in beams deeper than 0.6 m) ≥ 0.3 m from the beam top; otherwise, $a_{poor} = 0.7$.

EC2 DCL for Primary Beams

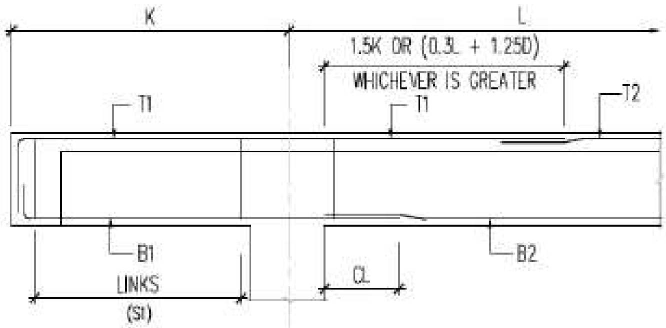
Ref. Fardis et al (2015)

EC2 DCL for Primary Beams

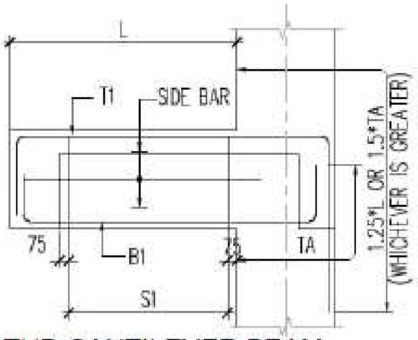
EC8 detailing rules for the transverse reinforcement of primary beams

	DC H	*DC M*	*DC L*
		Outside critical regions	
Spacing, $s_h \leq$		$0.75d$	
$\rho_w = A_{sh}/b_w s_h \geq$		$(0.08\sqrt{f_{ck}(\mathrm{MPa})})/f_{yk}(\mathrm{MPa})$[a]	
		In critical regions	
Diameter, $d_{bw} \geq$		6 mm	
Spacing, $s_h \leq$	$6d_{bL}$[b], $h/4$, $24d_{bw}$, 175 mm	$8d_{bL}$[b], $h/4$, $24d_{bw}$, 225 mm	–

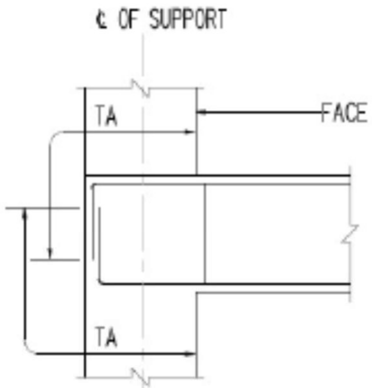
[a] NDP (nationally determined parameter) per EC2; the value recommended in EC2 is given here.
[b] d_{bL}: minimum diameter of all top and bottom longitudinal bars within the critical region.



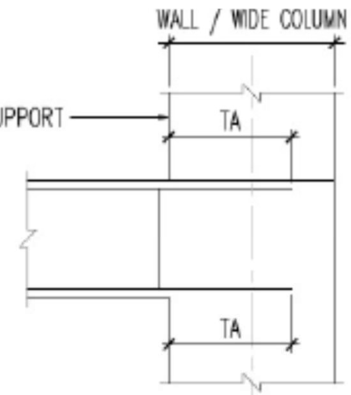
CONTINUOUS CANTILEVER BEAM



STUD CANTILEVER BEAM

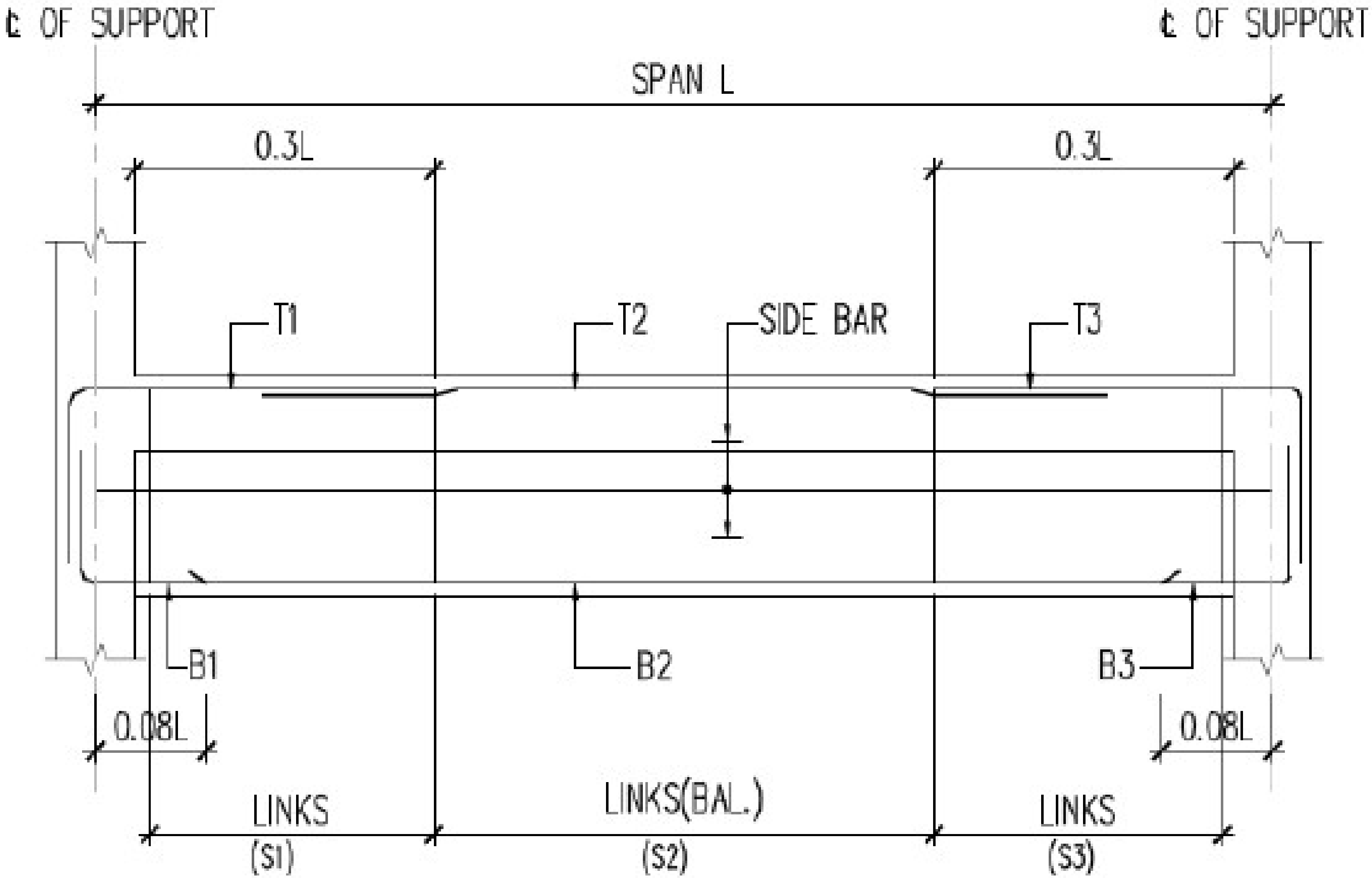


BEAM TO COLUMN / WALL



BEAM TO WIDE COLUMN / WALL

EC2 DCL for Primary Beams



EC8 detailing rules for vertical bars in primary columns (in secondary ones: as in DC L)

	DC H	*DC M*	*DC L*
$\rho_{min} = A_{s,min}/A_c$	1%		$0.1N_d/A_c f_{yd}$, 0.2%[a]
$\rho_{max} = A_{s,max}/A_c$	4%		4%[a]
Diameter, d_{bL}		≥8 mm	
Number of bars per side	≥3		≥2
Spacing along the perimeter of bars restrained by a tie corner or hook	≤150 mm	≤200 mm	–
Distance along perimeter of unrestrained bar to nearest restrained one		≤150 mm	
Lap splice length[b]		$l_0 = 1.5[1–0.15(c_d/d_{bL} - 1)]a_{tr}(d_{bL}/4)f_{yd}/(2.25f_{ctd})$[c,d,e]	

[a] NDP (nationally determined parameter) per EC2; the value recommended in EC2 is given here.

[b] Anchorage length in tension is reduced by 30% if the bar end extends by $\geq 5d_{bL}$ beyond a bend $\geq 90°$.

[c] c_d: Minimum of: concrete cover of lapped bar and 50% of clear spacing to adjacent lap splice.

[d] $a_{tr} = 1 - k(2n_w A_{sw} - A_{s,t,min})/A_s$, with $k = 0.1$ if the bar is at a corner of a hoop or tie, $k = 0.05$ otherwise; A_{sw}: cross-sectional area of a column tie; n_w: number of ties in the cover of the lapped bar over the outer third of the length l_0; $A_s = \pi d_{bL}^2/4$ and $A_{s,t,min}$ is specified in EC2 as equal to A_s.

[e] $f_{ctd} = f_{ctk,0.05}/\gamma_c = 0.7f_{ctm}/\gamma_c = 0.21f_{ck}^{2/3}/\gamma_c$: design value of 5%-fractile tensile strength of concrete.

EC2 DCL
for Columns

EC2 DCL for Columns

EC8 detailing rules for transverse reinforcement in primary columns

	DC H	DC M	DC L
Critical region length[a] $\geq$	$1.5h_c$, $1.5b_c$, 0.6 m, $H_{cl}/5$	h_c, b_c, 0.45 m, $H_{cl}/6$	h_c, b_c,
	Outside the critical regions		
Diameter, $d_{bw} \geq$	6 mm, $d_{bL}/4$		
Spacing, $s_w \leq$	$20d_{bL}$, h_c, b_c, 400 mm		
At lap splices of bars with $d_{bL} > 14$ mm, $s_w \leq$	$12d_{bL}$, $0.6h_c$, $0.6b_c$, 240 mm		
	In critical regions[b]		
Diameter, $d_{bw} \geq$[c]	6 mm, $0.4\sqrt{(f_{yd}/f_{ywd})}d_{bL}$	6 mm, $d_{bL}/4$	
Spacing, $s_w \leq$[c,d]	$6d_{bL}$, $b_o/3$, 125 mm	$8d_{bL}$, $b_o/2$, 175 mm	As outside critical regions
Mechanical ratio $\omega_{wd} \geq$[e]	0.08	–	
Effective mechanical ratio $a\omega_{wd} \geq$[d,e,f,g]	$30\,\mu_\varphi{}^*\nu_d\varepsilon_{yd}\,b_c/b_o - 0.035$	–	
	In the critical region at the base of the column (at the connection to the foundation)		
Mechanical ratio $\omega_{wd} \geq$	0.12	0.08	–
Effective mechanical ratio $a\omega_{wd} \geq$[d,e,f,h,i]	$30\,\mu_\varphi\nu_d\varepsilon_{yd}b_c/b_o - 0.035$		–

[a] h_c, b_c, H_{cl}: column sides and clear length.

[b] For DC M: If a value of $q \leq 2$ is used for the design, the transverse reinforcement in critical regions of columns with an axial load ratio $\nu_d \leq 0.2$ may follow only the rules for DC L columns.

[c] For DC H: In the two lower storeys of the building, the requirements on d_{bw}, s_w apply over a distance from the end section not less than 1.5 times the critical region length.

[d] Index *c* denotes the full concrete section; index *o* the confined core to the centreline of the perimeter hoop; b_o is the smaller side of this core.

[e] ω_{wd}: volume ratio of confining hoops to confined core (to centreline of perimeter hoop) times f_{ywd}/f_{cd}.

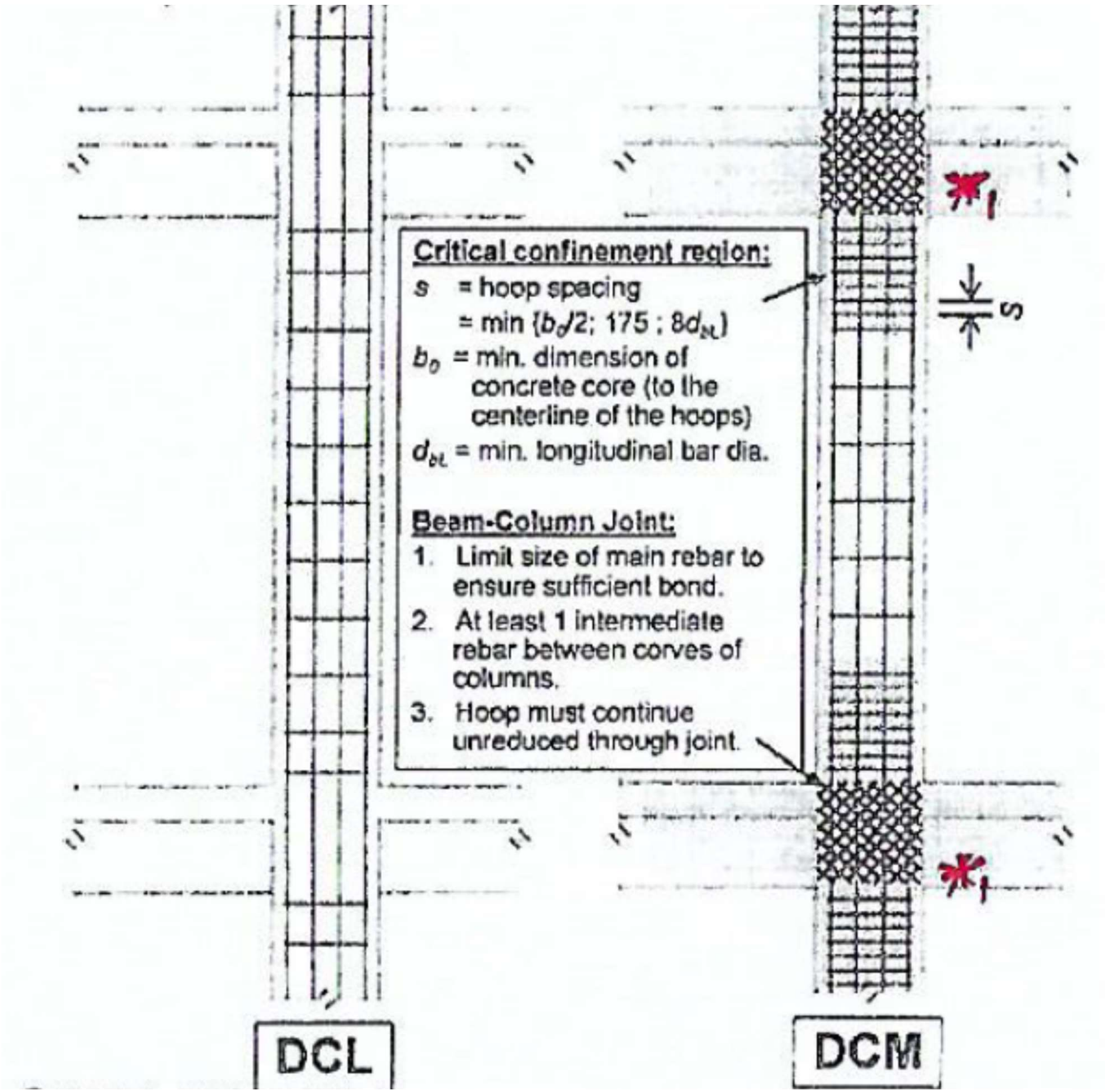
[f] $a = (1 - s/2b_o)(1 - s/2h_o)(1 - \{b_o/[(n_h - 1)h_o] + h_o/[(n_b - 1)b_o]\}/3)$: confinement effectiveness factor of rectangular hoops at spacing *s*, with n_b legs parallel to the side of the core with length b_o and n_h legs parallel to the side of length h_o.

[g] For DC H: at column ends protected from plastic hinging through the capacity design check at beam–column joints, $\mu_\varphi{}^*$ is the value of the curvature ductility factor that corresponds per Equations 5.64 to 2/3 of the basic value, q_o, of the behaviour factor applicable to the design; at the ends of columns where plastic hinging is not prevented, because of the exemptions from the application of Equation 5.31, $\mu_\varphi{}^*$ is taken equal to μ_φ defined in footnote h (see also footnote i); $\varepsilon_{yd} = f_{yd}/E_s$.

[h] μ_φ: curvature ductility factor corresponding per Equations 5.64 to the basic value, q_o, of the behaviour factor applicable to the design.

[i] For DC H: The requirement applies also in the critical regions at the ends of columns where plastic hinging is not prevented, because of the exemptions from the application of Equation 5.31.

EC2 DCL
for Columns



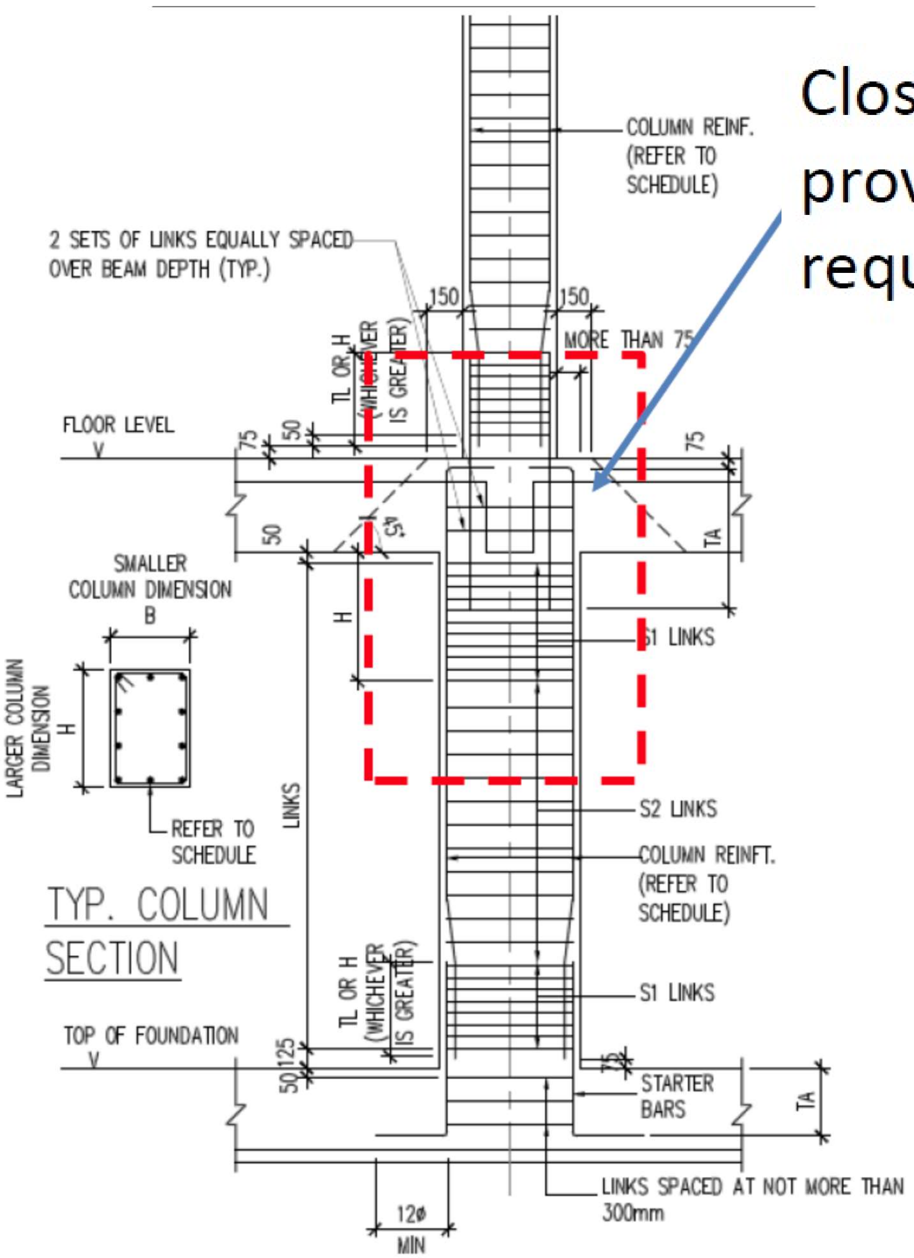
Critical confinement region – no splice allow'd (Detailing to **EC8**)

Reduced confinement region – splice allow'd (Detailing to **EC2**)

Special confinement for beam-column joint (Detailing to **EC8**)

EC2 DCL for Columns

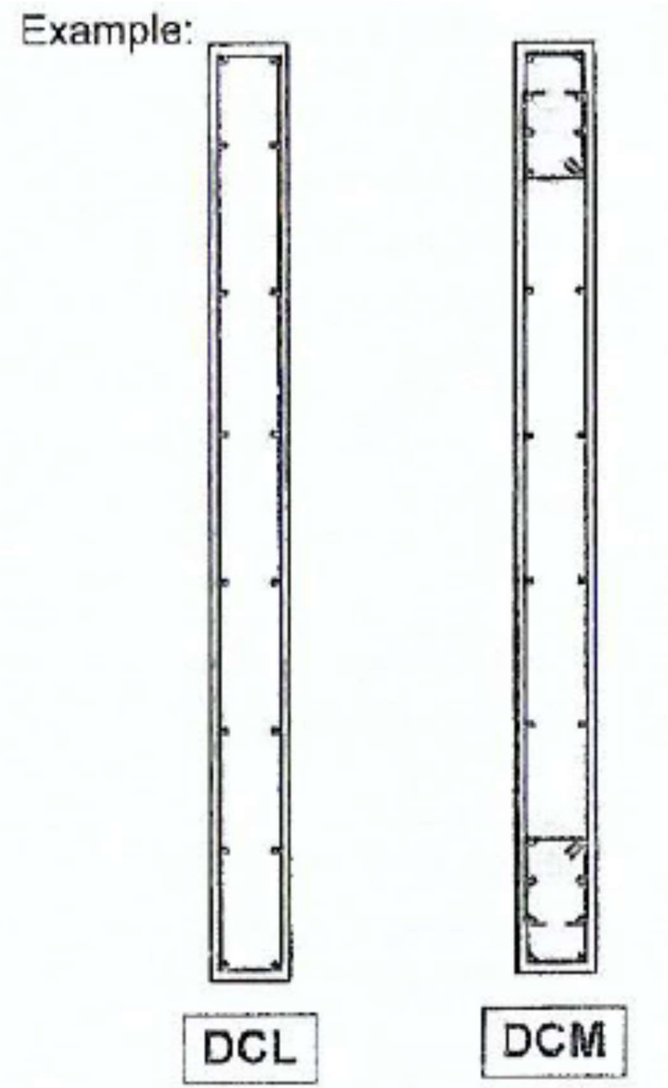
Closer link at joint area is provided to comply to EC2 requirement



EC2 DCL for Walls

EC8 detailing rules for ductile walls

	DC H	*DC M*	*DC L*
Critical region height, h_{cr}	$\geq \max(l_w, H_w/6)^b$ $\leq \min(2l_w, h_{storey})$ if wall ≤ 6 storeys $\leq \min(2l_w, 2h_{storey})$ if wall > 6 storeys		–
	Boundary elements		
a) In critical height region:			
- Length l_c from wall edge $\geq$	$0.15l_w$, $1.5b_w$, part of the section where $\varepsilon_c > 0.0035$		–
- Thickness b_w over $l_c \geq$	0.2 m; $h_{st}/15$ if $l_c \leq \max(2b_w, l_w/5)$, $h_{st}/10$ otherwise		–
- Vertical reinforcement:			
ρ_{min} over $A_c = l_c b_w$	0.5%		$0.2\%^a$
ρ_{max} over A_c		$4\%^a$	
Spacing along perimeter of bars restrained by tie corner or cross-tie hook	≤ 150 mm	≤ 200 mm	–
- Confining hoops (index w)^c:			
Diameter, $d_{bw} \geq$	6 mm, $0.4\sqrt{(f_{yd}/f_{ywd})}d_{bL}$	6 mm,	wherever $\rho_L > 2\%$ in section: as over rest of the wall (see case b below)
Spacing, $s_w \leq^d$	$6d_{bL}$, $b_o/3$, 125 mm	$8d_{bL}$, $b_o/2$, 175 mm	
$\omega_{wd} \geq^c$	0.12	0.08	–
$a\omega_{wd} \geq^{d,e}$	$30\,\mu_\varphi(\nu_d + \omega_\nu)\varepsilon_{yd}b_w/b_o - 0.035$		–
b) Over the rest of the wall height:	Wherever in the section $\varepsilon_c > 0.2\%$: $\rho_{v,min} = 0.5\%$; elsewhere: 0.2% In parts of the section where $\rho_L > 2\%$: distance of unrestrained bar in compression zone to nearest restrained bar ≤ 150 mm; hoops with $d_{bw} \geq \max(6\text{ mm}, d_{bL}/4)$, spacing $s_w \leq \min(12d_{bL}, 0.6b_{wo}, 240\text{ mm})^a$ till distance $4b_w$ above or below floor slab/beam; $s_w \leq \min(20d_{bL}, b_{wo}, 400\text{ mm})^a$ beyond that distance		

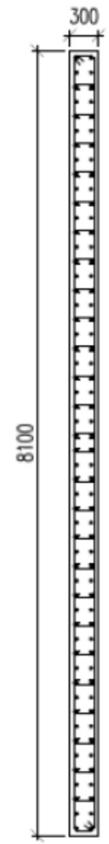


EC2 DCL for Walls

EC8 detailing rules for ductile walls

	DC H	DC M	DC L
Web			
Thickness, $b_{wo} \geq$	max(150 mm, $h_{storey}/20$)		–
Vertical bars (index: v):			
$\rho_v = A_{sv}/b_{wo}s_v \geq$	0.2%, but 0.5% wherever in the section $\varepsilon_c > 0.002$		0.2%[a]
$\rho_v = A_{sv}/b_{wo}s_v \leq$	4%		
$d_{bv} \geq$	8 mm		–
$d_{bv} \leq$	$b_{wo}/8$		–
Spacing, $s_v \leq$	min($25d_{bv}$, 250 mm)		min($3b_{wo}$, 400 mm)
Horizontal bars (index: h):			
$\rho_{h,min}$	0.2%		max(0.1%, $0.25\rho_v$)[a]
$d_{bh} \geq$	8 mm		–
$d_{bh} \leq$	$b_{wo}/8$		–
Spacing, $s_h \leq$	min($25d_{bh}$, 250 mm)		400 mm
$\rho_{v,min}$ at construction joints[f]	$\max\left(0.25\%;\ \dfrac{1.3f_{ctd} - N_{Ed}/A_c}{f_{yd} + 1.5\sqrt{f_{cd}f_{yd}}}\right)$		–

[a] NDP (Nationally Determined Parameter) per EC2; the value recommended in EC2 is given here.
[b] l_w: long side of rectangular wall section or rectangular part thereof; H_w: total height of wall; h_{storey}: storey height.
[c] (In DC M only) The DC L rules apply to the confining reinforcement of boundary elements, if: under the maximum axial force in the wall from the analysis for the seismic design situation, the wall axial load ratio $\nu_d = N_{Ed}/A_c f_{cd}$ is ≤ 0.15; or, if $\nu_d \leq 0.2$ but the q-value used in the design is $\leq 85\%$ of the q-value allowed when the DC M confining reinforcement is used in boundary elements.
[d] Footnotes d, e, f of Table 5.4 apply for the confined core of boundary elements.
[e] μ_φ: value of the curvature ductility factor corresponding through Equations 5.64 to the product of the basic value q_o of the behaviour factor times the ratio $M_{Ed,o}/M_{Rd,o}$ of the moment at the wall base from the analysis for the seismic design situation to the design value of moment resistance at the wall base for the axial force from the same analysis; $\varepsilon_{yd} = f_{yd}/E_s$; ω_{vd}: mechanical ratio of vertical web reinforcement.
[f] N_{Ed}: minimum axial load from the analysis for the seismic design situation (positive for compression); $f_{ctd} = f_{ctk,0.05}/\gamma_c = 0.7f_{ctm}/\gamma_c = 0.21f_{ck}^{2/3}/\gamma_c$: design value of 5%-fractile tensile strength of concrete.



VERTICAL BAR: H32-125 (B/F)
HORIZONTAL BAR: H13-150 (B/F)
HOOKS: R6-300

Main References

1. SS EN 1998-1 (2016) "Eurocode 8 : Design of structures for earthquake resistance. Part 1, General rules, seismic actions and rules for buildings" (Singapore Productivity and Standards Board)
2. NA to SS EN 1998-1 (2016) "Singapore National Annex to Eurocode 8 : Design of structures for earthquake resistance. Part 1, General rules, seismic actions and rules for buildings" (Singapore Productivity and Standards Board)
3. PD 6698:2009 "Recommendations for the design of structures for earthquake resistance to BS EN 1998" (British Standard Institution)
4. BC3:2016 "Guidebook for Design of Buildings in Singapore to Requirements in SS EN 1998-1" (Building and Construction Authority)
5. "Example Calculations to the Requirements of BC3:2016" (Building and Construction Authority)

Supplementary References

Balendra et al. (1990), " An Analytical Model For Far-Field Response Spectra With Soil Amplification Effects". Engineering Structures Vol. 12, No.4, pp. 263-268

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