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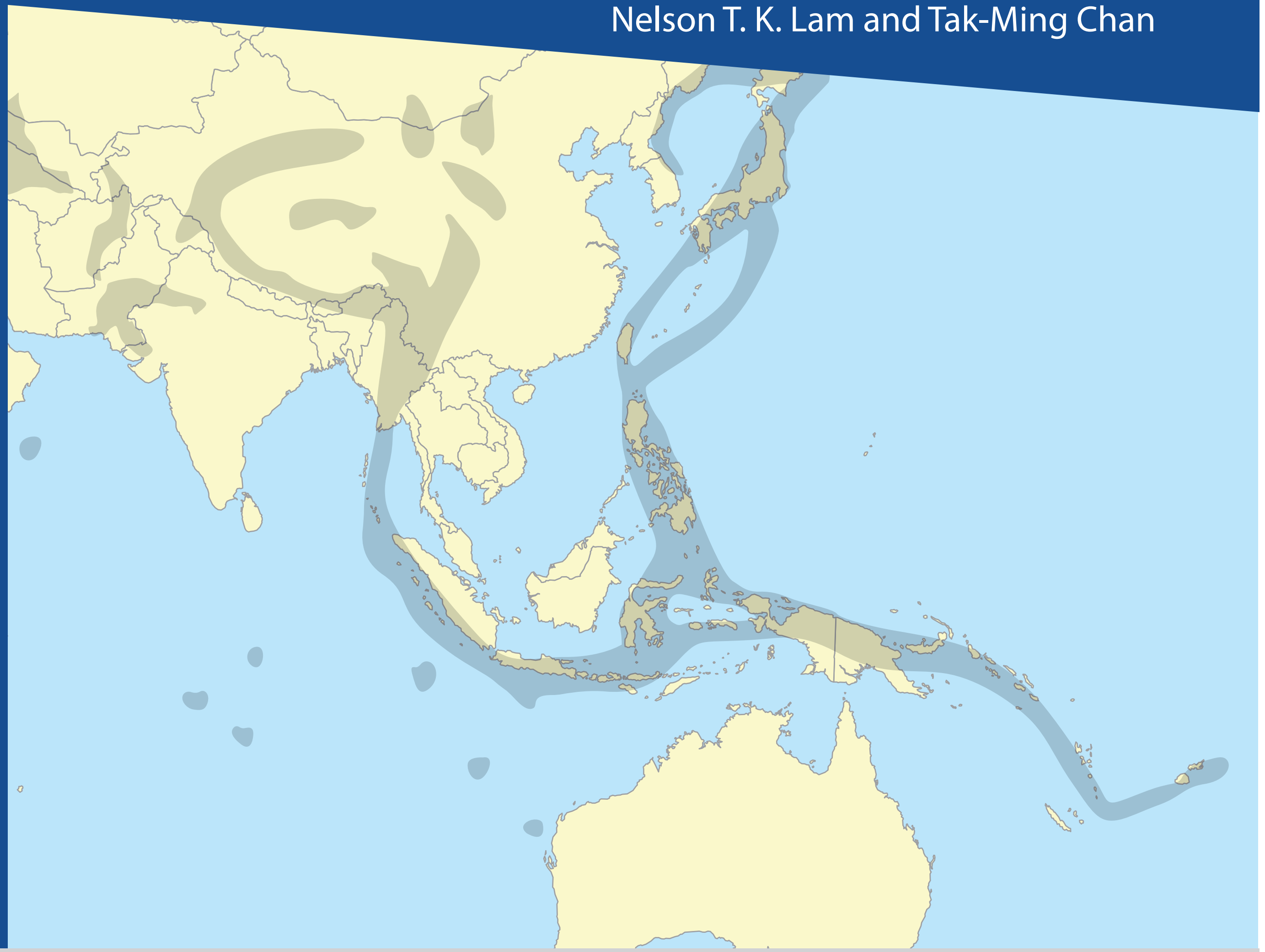
For Steel Construction (Hong Kong Branch)

Design of Buildings and Structures in Low to Moderate Seismicity Regions

Professional Guide: PG-002

Nelson T. K. Lam and Tak-Ming Chan



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This Professional Guide represents CNERC's continued contribution towards the building of a resilient world community. The underlying objective is to maintain living standards and improve safety in the face of natural disasters such as earthquakes. The scope of the undertaking in its entirety is broad, drawing expertise from many allied disciplines. Managing natural disasters entails developing adequate mitigation measures and preparedness levels. This Professional Guide is concerned specifically with mitigating the impact of future disasters by improving the robustness of newly constructed buildings in regions of low to moderate seismicity through a better understanding of hazard assessment and design methodology. The targeted readers are designers of building structures and policy makers in these regions.

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A Case Study of Code Compliant Design of Buildings in Regions of Low to Moderate Seismicity

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This chapter presents a case study example of a reinforced concrete building, for the benefit of practicing structural engineers working in regions of low to moderate seismicity, who have not had to take earthquake actions into account in the past and have little experience in earthquake code compliant design. A worked example of a 9-storey medium rise building located in Malaysia is presented, starting with site period calculations based on borehole records and proceeding to a generalised force method of analysis as opposed to the conventional code lateral force method. This approach is to circumvent issues generated by the various uncertainties involved when deriving the natural period properties of a real building structure. The underlying concept is explained with a simple 2D example in the first part of the chapter, followed by how best to make use of a structural analysis computer package when faced with the real design 3D environment challenges.

Keywords: Eurocode 8, code compliant, RC building, low to moderate seismicity, lateral force method, generalised force method

1. Introduction

With the aim of achieving a more robust level of structural safety, seismic design codes have been recently introduced and enforced in regions of low to moderate seismicity where earthquake actions have not been taken into account in the past. For example, Eurocode 8 (EC8) National Annex (NA) has been enforced in Singapore (SS EN1998-1:2013) and a completed NA which went through public comments in 2017 in Malaysia (NA for MS EN1998-1:2015). It is noted however, that structural engineers in these regions have little experience in earthquake code compliant design. This chapter, as an example, elaborates the seismic design of a typical medium rise Reinforced Concrete (RC) building. Such a building type is most vulnerable because of its short natural period ($T < 1.25$s) which may well coincide with the low period range of an earthquake response spectrum. This example, in addition to the RC hospital building example (Looi et al., 2015) aims to enhance the confidence of engineers in using a checking method, as an alternative to the typical commercial software dynamic modal analysis method, when deriving a seismic code compliant design.

2. Site natural period

A calculation of the site period based on borehole records is demonstrated below, so as to estimate the seismic loading demand, using a proposed Response Spectrum (RS) model expressed as a function of site period (Tsang et al., 2016).

2.1 Borehole records

The project site, not identified here, has an area of approximately 12 acres (48562 m^2) and is intended for the construction of one block of residential staff quarters, a main hospital block and a multi-storey car park in Peninsular Malaysia (see Figure 1). A general rule of thumb specifies that two boreholes for a low-rise building block are sufficient, and that the spacing of boreholes for multi-storey buildings should be between 15 m and 45 m. More boreholes are necessary for problematic and erratic soil formations (Sowers, 1979). For this site example, a total of 11 borehole records were selected, spread as evenly as possible over the whole site area.

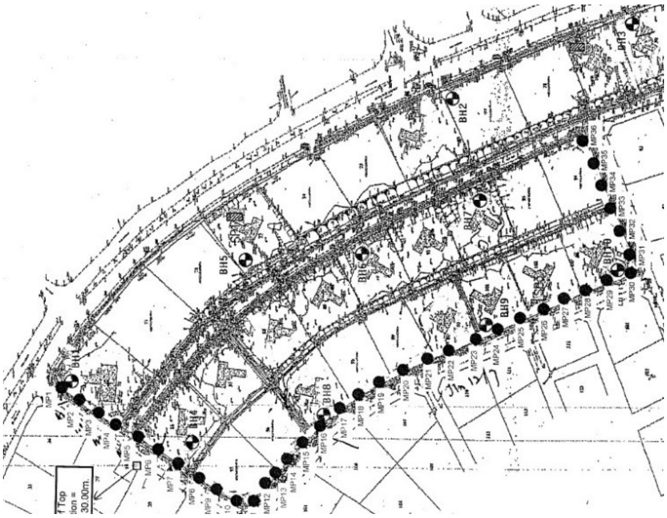


Figure 1. A site in Peninsular Malaysia with borehole layout

2.2 Computation of site natural period

The site natural period (T_S) is estimated by correlating Standard Penetration Test (SPT-N) values with shear wave velocity (SWV). Wair et al. (2012) summarised empirical formulae applicable to all types of soil. In this example, the Imai and Tonouchi (1982) SPT-N to SWV equation of SWV = $97N^{0.31}$ was adopted. For SPT-N $\geq$ 50, the equivalent SPT-N is derived by proportioning the SPT-N to a 300 mm penetration, e.g. if an SPT-N = 50 for a 270 mm penetration, the equivalent SPT-N is 50×300/270 = 55.6.

The individual soil layer thicknesses (di) divided by the respective initial SWV ($V_{s,i}$) ratio were calculated to obtain the weighted average SWV (VS) using Eq. (1).

$$V_s = \sum_{i=1}^{n} d_i \bigg/ \sum_{i=1}^{n} \frac{d_i}{V_{s,i}} \tag{1}$$

where $V_{s,i}$ = the SWV in m/s; d_i = the thickness of any layer.

$$T_S = \sum_{i=1}^{n} \frac{d_i}{V_{s,i}} \times 4 = \frac{4H_S}{V_S} \tag{2}$$

An example of site natural period computation for borehole number 1 is shown in Table 1.

Table 1. Computation of site natural period based on borehole number 1

Depth	d_i	SPT-N	$V_{s,i}$ (Imai and Tonouchi, 1982)	d_i / $V_{s,i}$		Depth	d_i	SPT-N	$V_{s,i}$ (Imai and Tonouchi, 1982)	d_i / $V_{s,i}$
0	0	0	0.0	0		22.5	1.5	29	279.2	0.005
1.5	1.5	6	170.3	0.009		24	1.5	24	263.1	0.006
3	1.5	7	178.7	0.008		25.5	1.5	29	279.2	0.005
4.5	1.5	10	199.9	0.008		27	1.5	31	285.1	0.005
6	1.5	10	199.9	0.008		28.5	1.5	34	293.5	0.005
7.5	1.5	16	231.7	0.006		30	1.5	31	285.1	0.005
9	1.5	17	236.1	0.006		31.5	1.5	33	290.8	0.005
10.5	1.5	17	236.1	0.006		33	1.5	55.6	342.6	0.004
12	1.5	21	252.3	0.006		34.5	1.5	60	350.8	0.004
13.5	1.5	19	244.5	0.006		36	1.5	88.2	396.0	0.004
15	1.5	21	252.3	0.006		37.5	1.5	107	420.7	0.004
16.5	1.5	24	263.1	0.006		39	1.5	100	411.9	0.004
18	1.5	27	273.0	0.005		40.5	1.5	150	467.8	0.003
19.5	1.5	25	266.5	0.006		42	1.5	214	523.0	0.003
21	1.5	27	273.0	0.005		Sum:	42	-	-	0.155

Hence, from Eq. (1), $V_s = \sum_{i=1}^{n} d_i \Big/ \sum_{i=1}^{n} \frac{d_i}{V_{s,i}} = 42/0.155 = 272 \text{ m/s}$

From Eq. (2), $T_S = \sum_{i=1}^{n} \frac{d_i}{V_{s,i}} \times 4 = \frac{4H_S}{V_S} = \frac{4(42)}{272} = 0.62 \text{ s}$

It is suggested that the arithmetic mean of the site natural periods (T_S) derived from all the boreholes should be adopted for site classification purposes. In this example, the mean value of T_S is computed as 0.60 s (see Table 2).

Table 2. Arithmetic mean of the site natural periods

Borehole no.	T_S (s)		Borehole no.	T_S (s)
1	0.62		7	0.65
2	0.55		8	0.75
3	0.45		9	0.50
4	0.51		10	0.65
5	0.64		11	0.61
6	0.71		Mean	0.60

2.3 The corresponding elastic RS

The soil RS model in this section is based on the draft Malaysia NA (MS EN1998-1:2015). In Table 2, T_s = 0.6 s falls within the Flexible Soil (FS) classification range where 0.5 s ≤ T_s ≤ 1.0 s. For the region in Peninsular Malaysia, the elastic RS (S_e) is shown in different formats in Figure 2 (Looi et al., 2015).

RS sheet no. 4.4

T (s)	S_e (g)	S_{De} (mm)
0.00	0.31	0.00
0.10	0.31	0.77
0.20	0.31	3.07
0.30	0.31	6.91
0.40	0.31	12.29
0.50	0.31	19.20
0.60	0.31	27.65
0.70	0.31	37.63
0.72	0.31	39.81
0.80	0.28	44.24
0.90	0.25	49.77
1.00	0.20	49.77
1.10	0.17	49.77
1.20	0.14	49.77
1.30	0.12	49.77
1.40	0.10	49.77
1.60	0.08	49.77
1.80	0.06	49.77
2.00	0.05	49.77
2.20	0.04	49.77
2.40	0.03	49.77
2.60	0.03	49.77
2.80	0.03	49.77
3.00	0.02	49.77
3.20	0.02	49.77
3.40	0.02	49.77
3.60	0.02	49.77
3.80	0.01	49.77
4.00	0.01	49.77

Location: Peninsular Malaysia

Importance class III

Site condition: FLEXIBLE SOIL

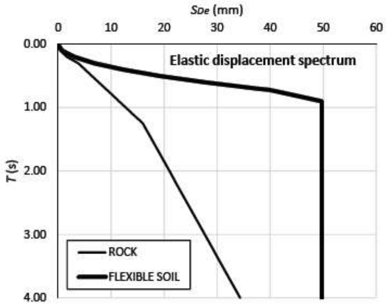
Site period: T_s = 0.6 s

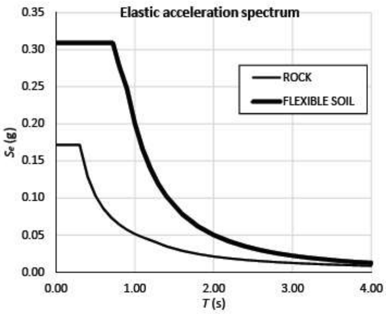
Notional pga: 0.12 g (= $S_{e\,max}$/2.5)

Equations for spectra		
T (s)	S_e (g)	S_{De} (mm)
$T \le T_C$	0.31	$76.8\,T^2$
$T_C \le T \le T_D$	$0.222/T$	$55.3\,T$
$T \ge T_D$	$0.2/T^2$	49.77

where T_C = 0.72 s

and T_D = 0.90 s





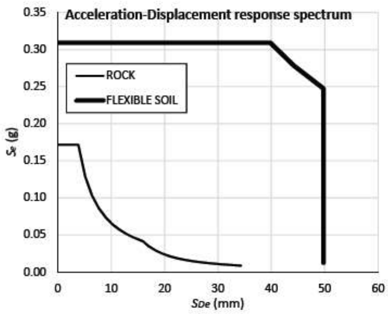


Figure 2. Elastic RS in different formats (Peninsular Malaysia, T_S = 0.6 s, Class III Importance)

3. RC building description

3.1 Briefs of the 9-storey RC building

One block of the residential staff quarters on the unidentified project site is presented. Figure 3 shows the architectural perspectives of the 9-storey RC building.

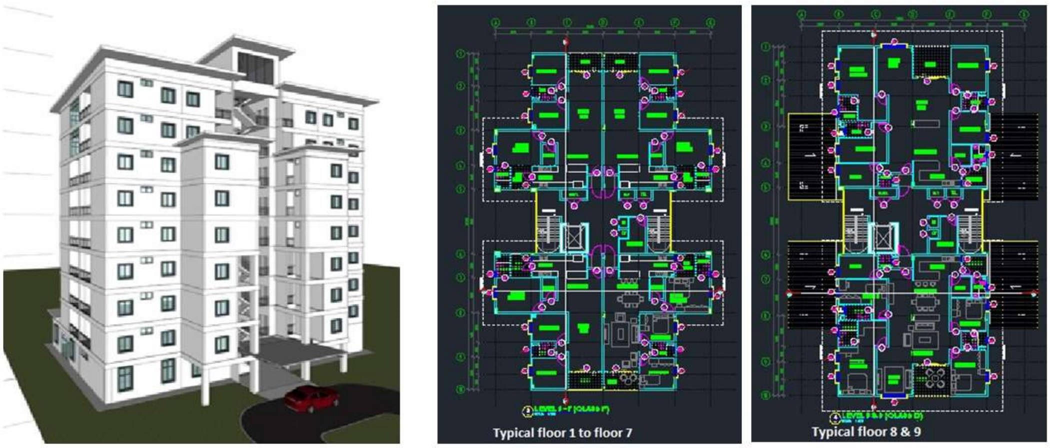


Figure 3. A 9-storey residential RC building – architectural perspective & plan

The RC building corresponds to the Class III importance level, i.e. buildings with large numbers of occupants (condominiums, shopping centres, schools and public buildings) according to Table B1 in the draft Malaysia NA (MS EN1998-1:2015), possessing an importance factor of 1.2, are assumed to experience a notional peak ground acceleration (PGA) of 0.08g. The building measures 34.2 m × 21.6 m on plan for the lower 7 floors, 34.2 m × 14.4 m on plan for the upper 2 floors and is 27 m high, above ground. The lateral force resisting system is by wall-frame interaction. The typical storey height is 3 m, typical beam size is 300 mm × 600 mm and slabs are 150 mm thick. The main columns are sized at 300 mm × 1000 mm (lower 7 floors except wings) and 300 × 750 mm (wings of lower 7 floors and all columns at upper 2 floors). The core wall thickness is 300 mm and the shear walls, 200 mm. The concrete grade is C30/37 according to Eurocode 2 (MS EN1992-1:2010). Figure 4 shows the computer model using ETABS (CSI, 2003) and its typical structural key plans. Frames are modelled as line elements, shear walls as membrane elements and typical floor slabs as shell elements. Rigid diaphragm behaviour is assumed for all floors. The supports are modelled as fixed.

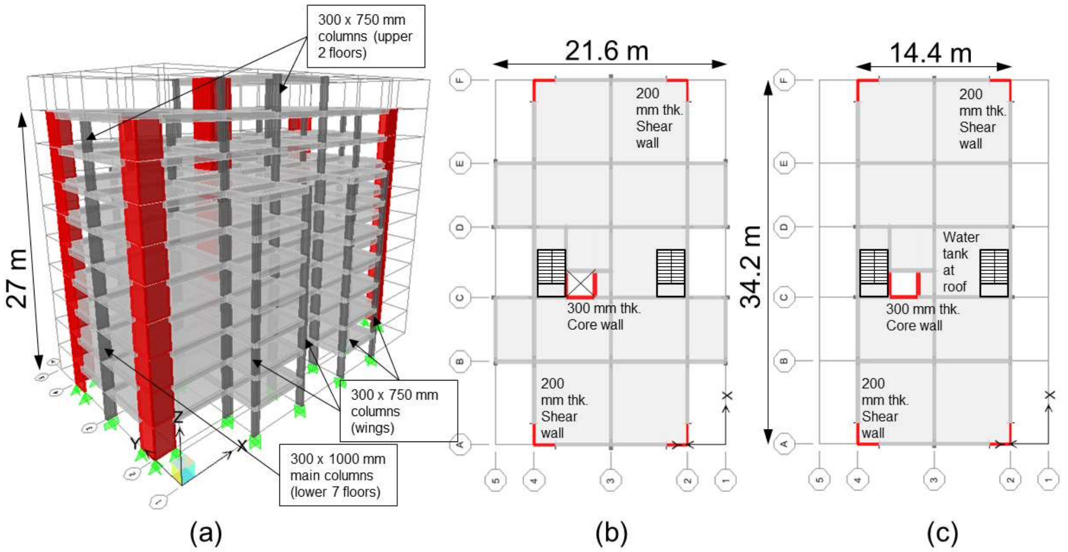


Figure 4. A 9-storey residential RC building (a) computer structural model (b) Typical structural key plan lower 7 floors (c) Typical structural key plan upper 2 floors 9-storey residential RC building – architectural perspective & plan

3.2 Actions for the building

For gravity load, the average building density, including selfweight of 4 kN/m³, was estimated, arriving at a typical floor permanent action of 10.5 kPa (0.875 × 4 kN/m³ × 3 m) plus a variable action of 1.5 kPa (0.125 × 4 kN/m³ × 3 m). At roof level, 6 kPa and 0.25 kPa were applied as permanent and variable actions respectively. The water tank permanent action was 25 kPa and variable action was 5 kPa, situated at the slab panel at grid C2-D3 at roof level. For lateral loading, wind action was calculated in accordance with the generic Eurocode 1, where the building is situated in terrain category 4 (city area), with local basic wind speed 20 m/s and wind eccentricity of 15% perpendicular to wind direction. A unique imperfection load requirement of EC2 (MS EN1992-1:2010 and simplified in Table 3.1 of IStructE Manual 2006), was applied for stability robustness purposes, by taking a maximum inclination 1/400, amounting to 0.25% of the ultimate permanent and variable actions. ie. 0.25% (1.35 × 6007 kN + 1.5 × 858 kN) = 24 kN.

For computation of the storey mass of the building, Eq. (3) in accordance with EN 1998-1 Cl 3.2.4(2)P,

$$m = \sum G_{k,i} + \sum \psi_{E,I} \quad Q_{k,i} \tag{3}$$

where $G_{k,i}$, and $Q_{k,i}$ = characteristic permanent and variable mass respectively

$\psi_{E,I} = \phi\psi_{2i}$ (EN 1998-1 Cl 4.2.4(2)P)

ϕ = 0.8 (Category A in EN 1991-1 Cl 6.3.1.1)

ψ_{2i} = 0.3, quasi-permanent value of the variable action Q_i.

Hence, it is assumed that 100% of permanent action and 24% (being 0.8 × 0.3) of variable action will 'translate' into the mass of the building during an earthquake.

In view of the minimum requirement for EC8 Ductility Class Low (DCL) and in line with the limited ductile construction detailing practice in Malaysia, the elastic spectrum is reduced using the behaviour factor (q) 1.5 into a design spectrum. Figure 5 shows the design RS acceleration (S_d).

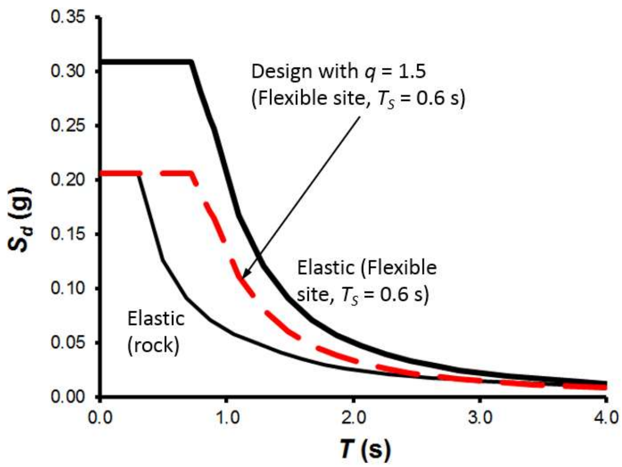


Figure 5. The design acceleration spectrum

4. Generalised force method (GFM)

The approximate fundamental period of a building is uncertain and is regionally dependent as to its structural configuration, construction materials and local construction practice (Jacobs, 2008), which explains the inconsistencies among the empirical formulae in different design codes. A GFM of analysis, as opposed to the conventional code lateral force method, is used to circumvent issues generated by the uncertainties in the natural period of real building structures.

4.1 The code lateral force method

The lateral force method of analysis as stipulated in EC8, entails the determination of the natural period of vibration, T1, using Eq. (4a) and the determination of the design base shear, F_b, using Eq. (4b).

$$T_1 = 0.05H^{0.75}\text{, where H is building height.} \quad (4a)$$

$$F_b = S_d(T_1)\,\lambda m \quad (4b)$$

where $S_d(T_1)$ is the design RS acceleration at period T_1, and λ_m is the effective mass of the building where the correction factor, λ, can be taken as 85% of the total mass for the first mode (EC8 Cl. 4.3.3.2.2(1)P).

Figure 6 shows the 2D plane frame of the 9-storey RC building in the X and Y directions along with the standard lateral force method calculation steps according to EC8.

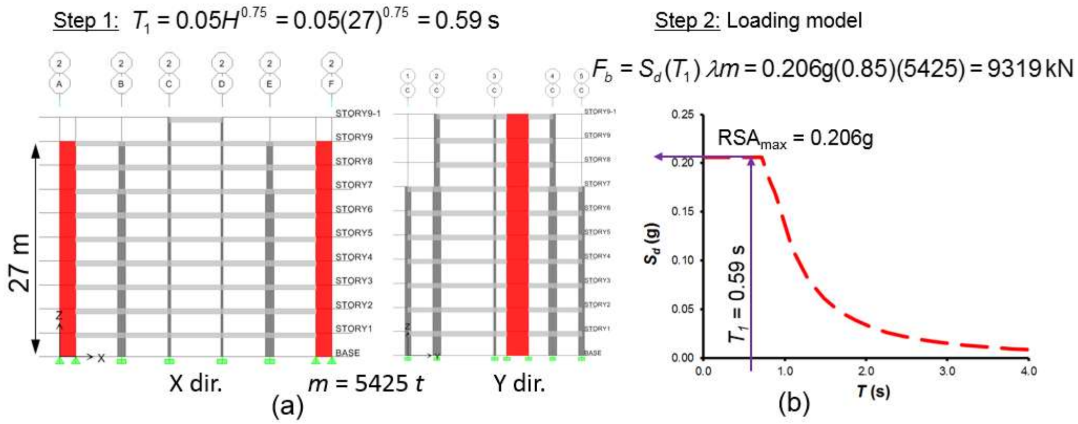


Figure 6. Code lateral force method, as EC8. (a) Step 1, estimating the fundamental period (b) Step 2 computing the base shear demand

The lateral forces, F_j, are applied to individual floor levels in the building using Eq. (4c), assuming the fundamental mode shape is approximated by a lateral displacement increasing with z (see Table 3).

$$F_j = F_b \frac{m_j z_j}{\sum_j m_j z_j} \quad (4c)$$

where z_j is the height at floor level j of the building when subject to the lateral force and m_j is the floor mass.

Table 3. Base shear distribution according to code lateral force method

Floor	mj (ton)	zj (m)	mj zj	Fj (kN)
R	54.6	30	1637	194
9F	397.7	27	10739	1275
8F	537.9	24	12910	1532
7F	633.6	21	13305	1579
6F	633.6	18	11405	1354
5F	633.6	15	9504	1128
4F	633.6	12	7603	903
3F	633.6	9	5702	677
2F	633.6	6	3802	451
1F	633.6	3	1901	226
BASE		0	0	0.00
	5425.3		78506.6	9319

The static load should be applied to two orthogonal directions on plan (see Figure 7).

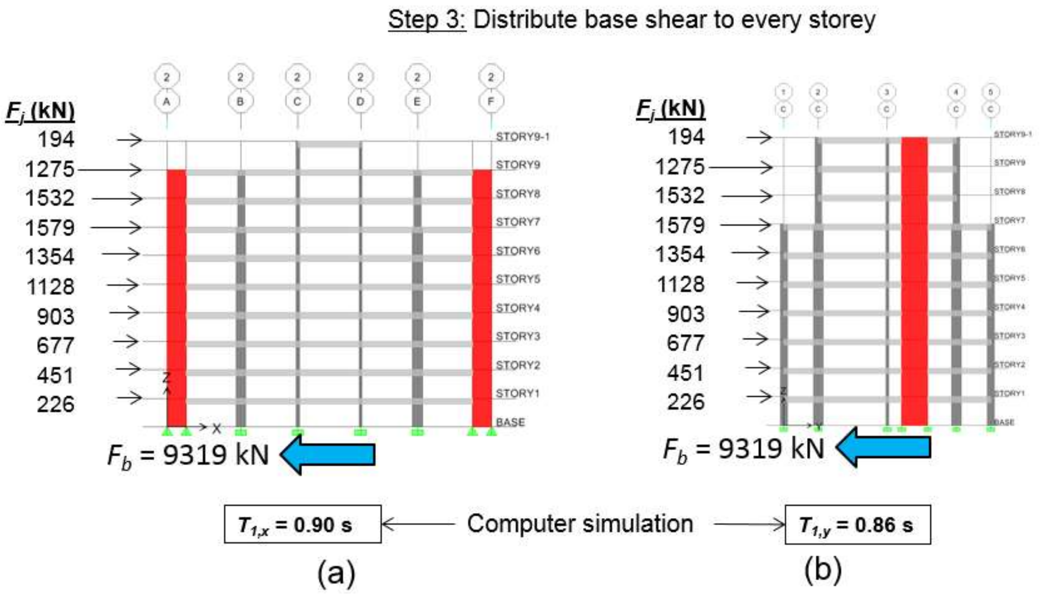


Figure 7. Step 3 of code lateral force method to distribute base shear for (a) X-direction (b) Y-direction

The code lateral force method as required by EC8 is completed at this point. It can be seen that the force distribution in the code lateral force method is identical in both X and Y directions, unrelated to changes of fundamental period obtained from computer modelling. The uncertainties stemming from the inconsistencies in the natural period value calculated by Eq. (4a) (where T_1 = 0.59 s) and that reported by the computer structural model (where $T_{1,x}$ = 0.90 s and $T_{1,y}$ = 0.86 s) can be circumvented by continuing with the GFM method to obtain improved estimates.

4.2 Operating the GFM

The force obtained at each storey using the code lateral force method is loaded into a computer structural analysis system to obtain the lateral displacement (δ_j). Table 4 shows the computation of GFM in the X direction, where simple spreadsheet computation enables $m_j\delta_j^2$ and $m_j\delta_j$, to be obtained. This is Step 4.

Table 4. Step 4 – Deflection obtained from code lateral force method and further computation of GFM in the X direction

Floor	m_j (ton)	F_j (kN)	δ_j (mm)	$m_j\ \delta_j^2$	$m_j\ \delta_j$
R	54.6	194	71.8	281372.5	3918.1
9F	397.7	1275	66.4	1753497.1	26409.3
8F	537.9	1532	59.7	1918741.1	32126.3
7F	633.6	1579	52.5	1749176.4	33290.4
6F	633.6	1354	44.6	1262447.5	28281.9
5F	633.6	1128	35.9	818662.7	22774.8
4F	633.6	903	26.8	453809.4	16956.6
3F	633.6	677	17.6	196593.5	11160.6
2F	633.6	451	9.3	54610.2	5882.2
1F	633.6	226	2.9	5237	1821.6
BASE		0.00	0.0	0.0	0.0
	5425.3	9319		8494147.4	182621.6

The objective of the GFM is to obtain the "correct" natural period estimate as a function of mass and stiffness. The effective displacement (δ_{eff}) is required, therefore, to evaluate the equivalent effective stiffness (k_{eff}) together with the effective mass (m_{eff}) using simple structural dynamic relationships.

The effective displacement is calculated using Eq. (5).

Step 5:

$$\delta_{eff} = \frac{\sum m_j\delta_j^2}{\sum m_j\delta_j} = \frac{8{,}494{,}147.4}{182{,}621.6} \approx 46.5\text{ mm} \tag{5}$$

The equivalent effective stiffness is calculated using Eq. (6).

Step 6:

$$k_{eff} = \frac{F_b}{\delta_{eff}} = \frac{9319}{46.5/1000} = 200{,}356\text{ kN/m} \tag{6}$$

The effective mass is calculated using Eq. (7).

Step 7:

$$m_{eff} = \frac{\left(\sum m_j \delta_j\right)^2}{\sum m_j \delta_j^2} = \frac{(182{,}621{,}6)^2}{8{,}494{,}147.4} \approx 3926 \text{ tons} \quad (7)$$

Step 8:

$$T_{eff} = 2\pi\sqrt{\frac{m_{eff}}{k_{eff}}} = 2\pi\sqrt{\frac{3926}{200356}} = 0.88 \text{ s} \quad (8)$$

Step 9:

Reading the loading demand from Figure 5 at 0.88 s, results in S_d = 0.168g, hence F_b = 0.168g (0.85) 5425.3 = 7600 kN.

Table 5. Step 9 - Base shear distribution of GFM in the X direction

Floor	m_j (ton)	z_j (m)	$m_j z_j$	F_j (kN) *revised as per GFM	δ_j (mm) *revised as per GFM
R	54.6	30	1637	158	58.5
9F	397.7	27	10739	1040	54
8F	537.9	24	12910	1250	48.7
7F	633.6	21	13305	1288	42.8
6F	633.6	18	11405	1104	36.4
5F	633.6	15	9504	920	29.3
4F	633.6	12	7603	736	21.8
3F	633.6	9	5702	552	14.3
2F	633.6	6	3802	368	7.6
1F	633.6	3	1901	184	2.3
BASE		0	0		0.0
	5425.3		78506.6	7600	

The revised lateral forces and the corresponding deflections obtained using the GFM Eqs. (5 – 8) can be notably lower than those estimated using the conventional code lateral force method in Eqs. (4a – 4c) (see Table 5, Figures 8 and 9). It should be noted that the newly estimated natural period is 0.88 s, which is closer to the results obtained using a computer analysis package (0.90 s in the X direction).

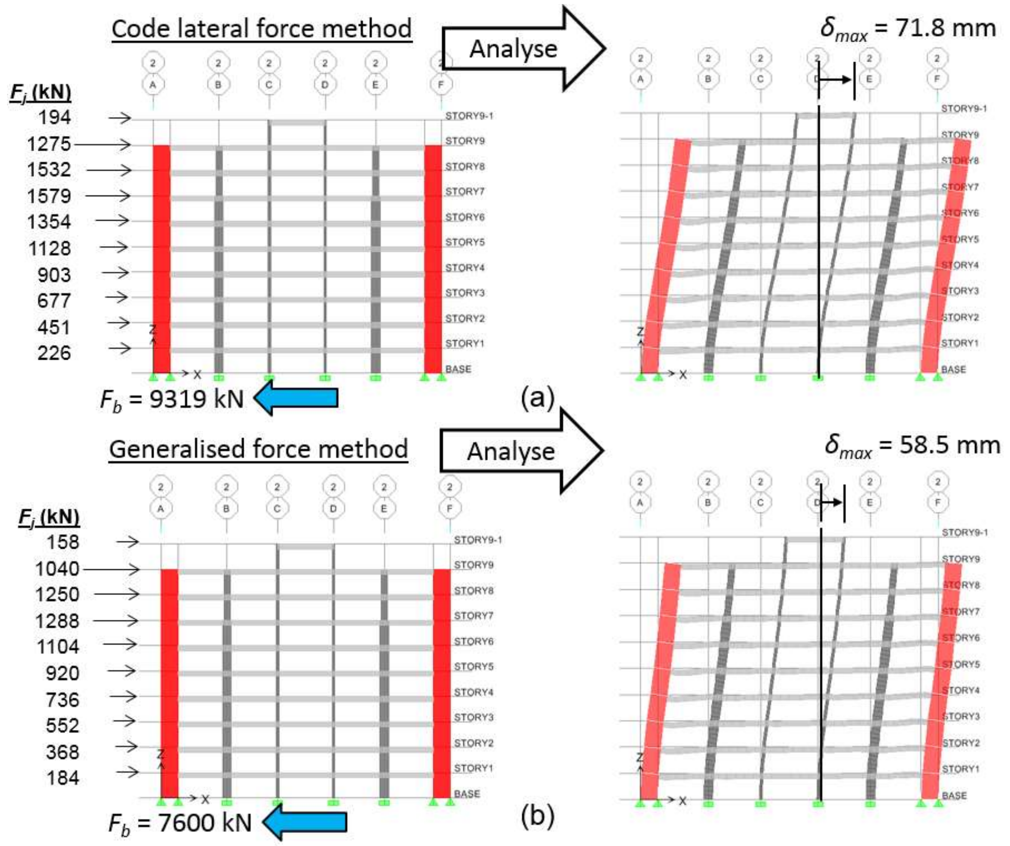


Figure 8. Comparison of base shear and deflection in the X direction (a) Code lateral force method; (b) Revised lateral force as per GFM

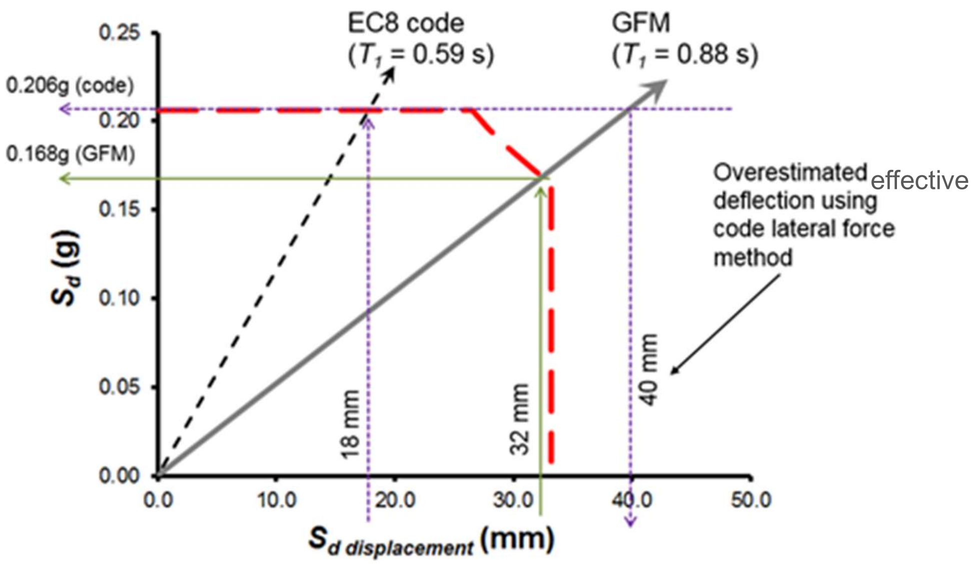


Figure 9. The comparison of results in RS formats for the different methods.

5. Dynamic modal analysis

A typical dynamic modal analysis using the response spectrum of Figure 5 was carried out, to compare with the code lateral force method and the GFM in the X direction. Table 6 shows the results of the modal analysis.

Table 6. Results of modal participation mass ratio (%) using commercial structural analysis package

Mode	Period	UX	UY	UZ	RX	RY	RZ
1	0.90	65.80	2.95	0.00	65.80	2.95	0.00
2	0.86	5.05	59.43	0.00	70.86	62.38	0.00
3	0.72	1.19	9.31	0.00	72.05	71.69	0.00
4	0.26	11.82	1.34	0.00	83.87	73.03	0.00
5	0.24	1.82	12.23	0.03	85.69	85.26	0.04
6	0.20	0.35	1.86	0.01	86.05	87.13	0.04
7	0.15	0.01	0.00	16.91	86.06	87.13	16.95
8	0.14	0.00	0.00	10.64	86.06	87.13	27.59
9	0.13	0.15	0.00	4.69	86.21	87.13	32.27
10	0.12	1.65	2.41	1.04	87.86	89.54	33.32
11	0.12	3.17	0.83	2.05	91.03	90.37	35.36
12	0.12	0.76	1.44	7.69	91.78	91.81	43.06

Since most structures have some form of irregularity, in order to fulfil architectural and functional requirements, the criterion, as stipulated in Cl. 4.2.3 in EC8, is very stringen, because it probably precludes the majority of building structures from being designed based on static analysis only. The vertical regularity prerequisite in EC8 should be relaxed in view of recent findings (in the literature) that buildings with $T_1 < 1.5$ s (which is fulfilled by most buildings with heights up to 50 m, or 16 storeys) are unlikely to experience any significant higher mode effects in their dynamic response to earthquake ground shaking. Reported analyses support this proposition including buildings possessing mass and stiffness irregularity in the building elevation (Su et al. 2011, Fardipour et al. 2011, Zhu et al. 2007). In Australia (AS 1170.4 2007, AEES 2009), dynamic analysis is only required for buildings exceeding 50 m (16 storeys) which are founded on rock, or stiff soil. In Singapore (NA to SS EC8 2013, BC3 2013) only one of the two prerequisites allowing the lateral force method listed in EC8 needs to be fulfilled. In view of the findings reported from the literature and prerequisites imposed by codes of practice in other areas of low to moderate seismicity, it is recommended that buildings up to 25 m high may be analysed using the lateral force analysis method irrespective of its degree of regularity in elevation. Table 7 summarises the various provisions applying to dynamic modal analysis and the GFM and EC8 code lateral force method, in the X direction.

Table 7. Summary of dynamic modal analysis, GFM and code lateral force method in the X direction

Approach	T_1 (s)	*Fb* (kN)	*Remarks*
Dynamic modal analysis	0.90	6338 (CQC combination)	This method is particularly encouraged in EC8 and is regarded as the 'reference method' in view of availability of commercial software possessing dynamic analysis capability. However, it is not easy to comprehend by the average structural engineer with limited experience in dynamic analysis.
GFM	0.88	7600	Buildings of up to 25 m in height may be subject to GFM irrespective of its regularity conditions in elevation. Results are comparable with dynamic modal analysis. This method is easy to comprehend and able to serve as a quick check against 3D structural dynamic analysis results
EC8 lateral force method	0.59	9319	Applicable when a) Fundamental period $T_1 \leq 4T_c$ and $T_1 \leq 2$ s. b) Building regular in elevation (EC 8 Cl.4.2.3.3.)

Subsequent rigorous design check based on acceptance criteria of ultimate strength and serviceability drift in accordance to design codes should be carried out.

6. Conclusions

One of the major drawbacks of the code lateral force method is that it prescribes building period estimation using empirical formula. This chapter discusses the generalised force method as a way of improving code lateral force method results. Calculations relating to a 9-storey RC building case study are presented above, as a demonstration for those practicing structural engineers, working in regions of low to moderate seismicity, who have not taken earthquake actions into account in the past. In the authors' opinion, since GFM involves only static analysis, it is easily adopted by those average structural engineers who possess limited experience of dynamic analysis. Through GFM, ballpark figures can be obtained to serve as a quick check against 3D structural dynamic analysis results. It is noted, that since torsional behaviour is not captured, readers are advised to refer to Lam et al. (2016), if necessary, for the enhanced version of GFM which is suitable for torsionally unbalanced buildings.

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