19 India

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19.1 INTRODUCTION

India has been subjected to some of the most severe earthquakes in the world. The strongest earthquakes that occurred in the country in the last one hundred years were the Assam earthquake of 1897 ($M = 8.7$),[1] the Kangra earthquake of 1905 ($M = 8.6$), the Bihar-Nepal earthquake of 1934 ($M = 8.4$), and the Assam-Tibet earthquake of 1950 ($M = 8.7$).

The Military Engineer Service (MES) made the first attempt at earthquake-resistant construction in India after the 1935 Quetta earthquake (Quetta is now in Pakistan). The MES required strengthening of brick or stone masonry buildings by providing reinforced concrete bands at plinth, lintel, and roof levels.[2]

The first seismic design code in India was published in 1962 [IS:1893 (1962)]; the code has since been revised in 1966, 1970, 1975, and 1984. A code that specifies the design and the required detailing for seismic construction of buildings was published in 1967 [IS:4326 (1967)]; that code was revised in 1976. As of 1992, current design seismic forces for buildings, elevated liquid storage tanks, stacks, concrete and masonry dams, embankments, bridges, and retaining walls are specified by IS:1893 (1984) (hereinafter referred to as "the Code"), while detailing and other construction aspects for seismic resistance are covered in IS:4326 (1976). The Code IS:4326 of 1984 covers seismic design and detailing requirements for concrete, steel, masonry, and timber buildings. In addition, the Bureau of Indian Standards has published an explanatory handbook on the two codes [SP: 22

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[1] M is the magnitude of an earthquake as measured on the Richter scale (see Appendix on Magnitude and Intensity of Earthquakes).

[2] Many of the major structures built in the seismic regions of the country immediately after independence in 1947 were designed for an empirically fixed horizontal force of 10% of the weight of the structure (Krishna 1985).

(1982)]; besides offering explanations of the two codes, the handbook also provides some examples with their solutions (Rajasankar and Jain 1988).

In this chapter, the seismic design regulations for buildings provided by the Code are presented. To provide an overall view of the seismic safety provisions in the Code, the safety factors and load factors specified in other Indian codes are also presented. Neither the design and detailing criteria for structures nor the seismic design provisions for structures other than buildings are presented in this chapter.

19.2 PERMISSIBLE STRESSES, SAFETY FACTORS, AND LOAD FACTORS

Most multistory building construction in India is done in reinforced concrete. Steel is usually used only for industrial structures because of the high cost. Construction that involves the use of reinforced concrete, prestressed concrete, and steel is governed by IS:456 (1978), IS:1343 (1980), and IS:800 (1984), respectively. IS:875 (1987) contains design load specifications (except seismic loads) for buildings.

The limit state design method is commonly used for design of buildings, although IS:456 (1978) allows the use of either the limit state or the working stress design methods. IS:1343 (1980) prescribes the limit state design procedure for prestressed concrete structures; in addition, a check on stresses caused by service loads is required. In the limit state design method, for both the reinforced concrete and prestressed concrete, the material strength partial safety factor is prescribed at 1.5 on concrete strength and at 1.15 on the yield stress of steel. The partial safety load factors for limit design are

(a) $1.5(DL + LL)$
(b) $1.2(DL + LL + EQ/WL)$
(c) $1.5(DL + EQ/WL)$
(d) $0.9DL + 1.5EQ/WL$

where DL, LL, EQ, and WL stand for dead, live, earthquake, and wind loads, respectively.

In working stress design, factors of safety assigned to concrete in direct compression and bending compression are 4.0 and 3.0, respectively, on 150-mm cube-crushing strength; the factor of safety is 1.80 on yield stress for reinforcement bars in tension.

In the design of steel structures, IS:800 (1984) allows the use of the working stress or the plastic methods of design; however, in practice the working stress method is usually followed. The factor of safety for the working stress method is 1.50 for direct stress and 1.67 for bending stress. Load factors for plastic design are

(a) $1.7(DL)$
(b) $1.7(DL + LL)$
(c) $1.7(DL + EQ/WL)$
(d) $1.3(DL + LL + EQ/WL)$.

In combinations with seismic loads, the Code allows an increase of 33.33 percent for allowable stresses in the elastic method of design, subject to the conditions that (i) for steel with a definite yield point, the stress is to be limited to its yield stress, (ii) for steel without a definite yield point, the stress is to be limited to the 0.2% proof stress or 80% of the ultimate strength, whichever is less, and (iii) for prestressed concrete members, the tensile stress in the concrete must not exceed two-thirds of the modulus of rupture of concrete.

The Code also allows an increase in the allowable bearing pressure for design of foundations when considering the seismic loads (Table 19.1). This increase depends on the type of foundation and the soil conditions. Because the allowable bearing pressure could be governed by either settlement limitations or shear failure considerations, the Code allows a lower increase (or no increase) in allowable bearing pressure for those footings and soil types that are more vulnerable to differential settlement.

19.3 OVERVIEW OF THE CODE

The Code provides both static (seismic coefficient method) and dynamic (response spectrum method) procedures for the determination of seismic design forces for buildings. Depending upon the height of the building, the Code recommends the use of the seismic coefficient method, the response spectrum method, or even a time-history analysis (Table 19.2). The Code requires that modal analysis be used for buildings that have unusual configurations, or irregular shapes and/

Table 19.1. Permissible Increase in Allowable Bearing Pressure or Resistance of Soils (%)

Serial No.	Type of Foundation	Rock/ Hard soils	Medium soils	Soft soils
1	Piles passing through any soil but resting on rock or hard soil	50	50	50
2	Piles not covered above	–	25	25
3	Raft foundations	50	50	50
4	Combined or isolated R.C. footings with tie beams	50	25	25
5	Isolated R.C. footings without tie beams or unreinforced strip foundations	50	25	–
6	Caisson foundations	50	25	25

Table 19.2. Recommended Method for Seismic Design of Buildings

Serial No.	Building Height	Seismic Zone*	Recommended Method
1	Greater than 40 m	III, IV, and V	Detailed dynamic analysis (either modal analysis or time-history analysis based on expected ground motion for which special studies are required). For preliminary design, modal analysis using response spectrum method may be employed.
2	Greater than 90 m	I and II	Modal analysis using response spectrum method.
3	Greater than 40 m and up to 90 m	All zones	Modal analysis using response spectrum method. Use of seismic coefficient method permitted for Zones I, II, and III.
4	Less than 40 m	All zones	Modal analysis using response spectrum method. Use of seismic coefficient method permitted in all zones.

*See Section 19.4.

or irregular distribution of mass or stiffness, as well as for industrial buildings and frame structures with large spans or height. For the seismic coefficient method to be used, the story heights should be approximately uniform, ranging between 2.7 m and 3.6 m, except that one or two stories may be up to 5 m high.

The Code generally requires that the design for horizontal seismic forces be considered only in any one direction at a time. However, if the stability of the building is a criterion for design, vertical seismic forces must be considered simultaneously with horizontal forces in any one direction. The Code also states that the design earthquake forces are assumed not to occur simultaneously with maximum flood, wind, or wave loads.

Wherever the floors of the building are capable of providing rigid diaphragm action, the lateral load is to be distributed to the various lateral-load-resisting elements, assuming the floors to be absolutely rigid in their horizontal planes. Otherwise, the Code states that frames are to be designed to behave independently, with the seismic force on each frame assessed in accordance with tributary mass. The latter requirement seems to have been an oversight because numerous situations occur where the floor is not completely rigid in its own plane but still has considerable in-plane stiffness, and for such situations, assumption of zero in-plane stiffness of floors is not appropriate (e.g., Jain and Mandal, 1992). The Code also prescribes that when a combination of shear walls and moment-resisting frames is used for lateral load resistance, the frames are to be designed for at least 25% of the seismic design force.

In both the seismic coefficient and the response spectrum methods, due consideration is given to the seismic zone where the structure is located, importance of the structure, soil-foundation system, ductility of construction, flexibility of the structure, and weight of the building.

19.4 ZONING MAP AND BASIC COEFFICIENT

India may be divided into three subregions based on geological considerations: (1) the Alpine Himalayan belt, (2) the southern peninsula, and (3) the intervening Indo-Gangetic plains. While peninsular India is an ancient stable area, the Alpine Himalayan belt is one of the most earthquake-prone regions in the world. Crustal instability in this belt is ascribed to the movement of the Indian Plate towards the Eurasian Plate, which occurs at a rate of about 50 mm per year.

The seismic zone map for the country was developed based on the epicentral distribution of significant past earthquakes and on the isoseismal configurations of such events. The original map demarcated areas that had potential for ground shaking of intensities of less than V, V, VI, VII, VIII, IX, X, and more than X in the Modified Mercalli Intensity (MMI) scale. The map was revised in the 1966 and 1970 editions of the Code based on the geological and geophysical data obtained from tectonic mapping and aeromagnetic and gravity surveys. [Krishna (1992) provides a brief historical view of zoning in India.] The current Indian zone map (Fig. 19.1) divides the country into five seismic zones (I to V) with the associated MMI of V (or less), VI, VII, VIII, and IX (and above), respectively. This zoning map is based on expected maximum seismic intensity in a region and does not consider frequency of occurrence; in this sense, the map does not divide the country into areas of equal seismic risk. But, this particular limitation of the zoning map can be an advantage in that it provides for a direct comparison between the maximum seismic intensity of an earthquake and the intensity assigned to the region.

The basic seismic coefficient (α_0) is equal to 0.01, 0.02, 0.04, 0.05, and 0.08, respectively, for the five zones (Table 19.3). The seismic zone factor F_0 used in the response spectrum method is simply five times the factor α_0. While observations on building performance in severe shaking during past earthquakes formed the basis of assigning $\alpha_0 = 0.08$ to Zone V, the value of α_0 for other zones was fixed more or less arbitrarily. Although much observational, experimental, and analytical information is available on the required

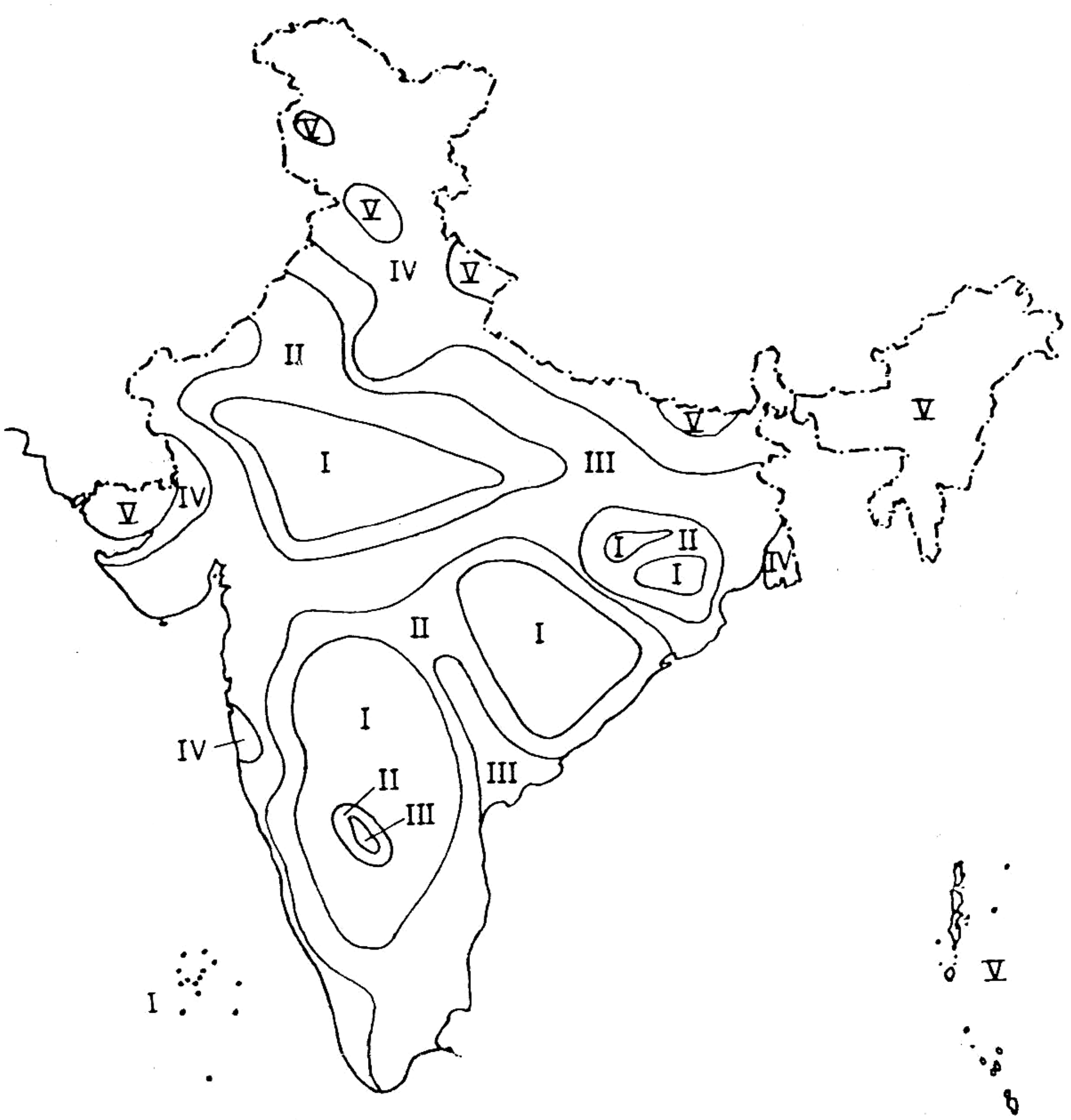

Fig. 19.1. Seismic zone map of India

seismic forces for zones of severe shaking, such data for areas of low or medium shaking is still lacking.

For underground structures and foundations at depths of 30 m or greater, the basic seismic coefficient may be taken as one-half of that in Table 19.3. Linear interpolation is allowed for depths less than 30 m. This specification of the Code recognizes the fact that seismic waves are amplified as they are reflected from a free boundary, i.e., ground surface. For situations where consideration of vertical acceleration is required, the vertical coefficient may be taken as one-half of that given in Table 19.3.

Table 19.3. Values of Basic Seismic Coefficient and Seismic Zone Factor

Serial No.	Zone No.	Basic Horizontal Seismic Coefficient* α_0	Seismic Zone Factor† F_0
1	V	0.08	0.40
2	IV	0.05	0.25
3	III	0.04	0.20
4	II	0.02	0.10
5	I	0.01	0.05

*For seismic coefficient method.
†For response spectrum method.

19.5 SOIL-FOUNDATION SYSTEM

The soil-foundation system has several important effects on the seismic behavior of a structure. First, the expected ground motion varies for different soil profiles. This is explicitly accommodated for in many codes by specifying a somewhat different design spectrum for different soil profiles (e.g., NEHRP 1991). Second, the flexibility due to soil and foundation deformation leads to a higher natural period and increased damping, and thus, in most cases a reduced seismic force. This is accommodated for in some codes by considering soil-structure interaction effects (e.g., Appendix to Chapter 6 of NEHRP 1991). The Indian Code IS:1893 does not account for these two effects, although it is also well recognized that those buildings for which the foundation system behaves as an entity, with minimal differential settlement, behave better in earthquakes. The Indian Code emphasizes the need to have a foundation system that will show minimal differential settlement by prescribing a factor β (Table 19.4) with a higher value for those soil and foundation systems that are liable to show more differential settlement. Therefore, a building on soft soil with isolated-untied footings is to be designed for 50% higher seismic loads than if the same building is supported on a raft foundation.

19.6 IMPORTANCE FACTOR

The Code prescribes an importance factor I of 1.0 for ordinary buildings and 1.5 for important service and community buildings. Table 19.5 gives the values of the importance factor for different structures. The Code indicates that these values are meant only for guidance and that the designer can choose a suitable value depending upon the importance of the structure based on economy, design strategy, and other considerations.

Table 19.4. Values of β for Different Soil-Foundation Systems

Serial No.	Type of Foundation	Rock/ Hard Soils	Medium Soils	Soft Soils
1	Piles passing through any soil but resting on rock or hard soil	1.0	1.0	1.0
2	Piles not covered above	–	1.0	1.2
3	Raft foundations	1.0	1.0	1.0
4	Combined or isolated R.C. footings with tie beams	1.0	1.0	1.2
5	Isolated R.C. footings without tie beams or unreinforced strip foundations	1.0	1.2	1.5
6	Caisson foundation	1.0	1.2	1.5

Table 19.5. Values of Importance Factor I

Serial No.	Structure	I
1	Dams (all types)	3.0
2	Containers of inflammable or poisonous gases or liquids	2.0
3	Important service and community structures, such as hospitals, water towers and tanks; schools, important bridges, important power houses, monumental structures; emergency buildings like telephone exchanges and fire bridges; large assembly structures like cinemas, assembly halls, and subway stations	1.5
4	All others	1.0

19.7 PERFORMANCE FACTOR

Prior to 1984, there was no explicit Code consideration given to ductility in the determination of design forces. The only requirement was for ductile detailing as per IS:4326 whenever the factor ($\beta I \alpha_0$) exceeded 0.05, which always happened in Seismic Zones IV and V. Since then, the Code explicitly recognizes the advantages of ductile construction and specifies a performance factor K that depends on the ductility of the structure. Values of this factor for different types of building construction are given in Table 19.6. Performance factor values for systems corresponding to Serial Nos. 1b, 2a, and 2b in Table 19.6 are applicable only if (a) the steel bracing members and the infill panels are considered in stiffness, as well as lateral strength calculations, and (b) the frame acting alone will be able to resist at least 25% of the design seismic forces. However, even in the 1984 Code the performance factor is not included for structures other than buildings.

Table 19.6. Values of Performance Factor K

Serial No.		Structure	K
1	a.	Moment-resistant frame with appropriate ductility details as given in IS:4326-1976 in reinforced concrete or steel	1.0
	b.	Frame as above with R.C. shear walls or steel bracing members designed for ductility	1.0
2	a.	Frame as in 1a with either steel bracing members or plain or nominally reinforced concrete infill panels	1.3
	b.	Frame as in 1a in combination with masonry infills	1.6
3		R.C. framed buildings not covered by 1 or 2 above	1.6

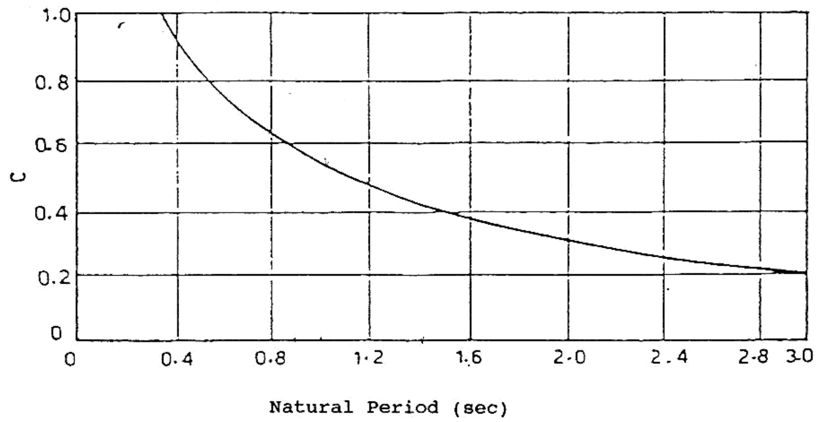

Fig. 19.2. Seismic coefficient C versus T

19.8 STRUCTURAL FLEXIBILITY

19.8.1 Seismic Coefficient Method

In the seismic coefficient method, the Code specifies a seismic factor C, which through its fundamental period T depends on the flexibility of the structure (see Fig. 19.2). Except in the low period range, the shape of the curve for C is the same as the average response spectrum curve for 5% damping proposed by Housner using four earthquake time histories (e.g., Housner and Jennings 1982).

The Code allows for the estimation of the fundamental period T by (a) experimental observations on similar buildings (this almost never happens in practice), or (b) any rational method of analysis (dynamic analysis). Alternately, for regular buildings, the Code suggests the following empirical relationships for the estimation of the fundamental period:

(i) For moment-resisting frames without shear walls or bracings

$$T = 0.1n \tag{19.1}$$

(ii) For other buildings

$$T = \frac{0.09H}{\sqrt{d}} \tag{19.2}$$

where n = number of stories including basements, H = total building height in meters, and d = maximum base dimension of building in meters in the direction parallel to the applied seismic force.

Although dynamic analysis is the preferred method for the estimation of the fundamental period, no provision is built into the Code to prevent the design for a natural period that, although obtained by dynamic analysis, may have a value unrealistically large. A designer may perform a dynamic analysis ignoring the filler walls (which usually are of unreinforced brick masonry) and obtain a rather large value for the natural period, and thus consequently low values for the seismic design forces. Future editions of the Code should incorporate a minimum design force based on empirical estimation of the fundamental period, as it is found in the codes of several other countries (e.g., UBC 1991).

19.8.2 Response Spectrum Method

In this method, a set of design spectrum curves (the Code terms them average acceleration spectra, denoted by S_a/g) are provided that account for flexibility of the structure (Fig. 19.3). These spectra are based on the average spectrum curves obtained by Housner using four earthquake time histories. A comparison of $\alpha_0 C$ and $F_0 S_a/g$ (for 5% damping) curves (Figs. 19.2 and 19.3) shows that the two match rather well except in the very low period range of 0–0.1 sec.; this range rarely governs design of a building. Thus, if the same fundamental period is used, both methods will give about the same overall design seismic force, provided 5% damping is considered.

The Code does not provide an explicit specification of damping for buildings. However, Appendix F of the Code recommends the use of the following percentages of critical damping for different types of structures:

(a)	Steel structures	2 to 5%
(b)	Concrete structures	5 to 10%
(c)	Brick structures in cement mortar	5 to 10%
(d)	Timber structures	2 to 5%
(e)	Earthen structures	10 to 30%.

Also, the Code provides definite values of damping

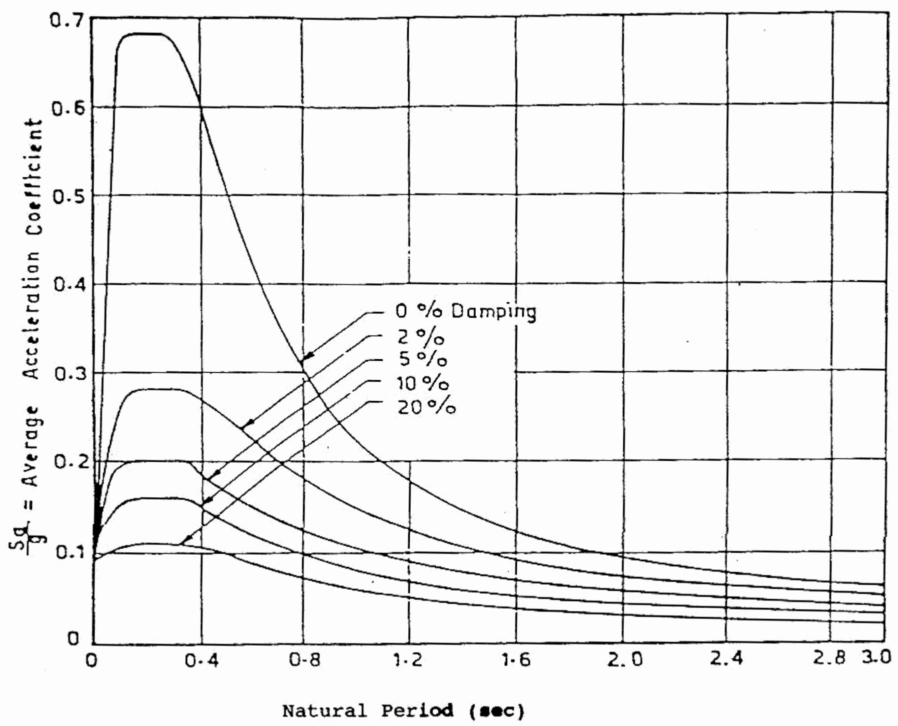

Fig. 19.3. Average acceleration spectra S_a/g versus T

for some special structures such as elevated tanks, gravity dams, and embankments.

19.9 WEIGHT OF THE STRUCTURE

The Code considers the fact that the live loads in the building do not contribute fully to the development of seismic forces, because generally such loads are not caused by masses firmly attached to the structure. Also, at a time of severe shaking, a building may not be loaded by its full design live load. Consequently, the Code specifies that only a fraction of the design live load need be considered in earthquake-resistant design (Table 19.7). The fractions of the live load indicated in Table 19.7 are to be used for the calculation of (a) lumped weights for determining the seismic forces, and (b) stresses caused by combined effects of gravitational and seismic forces. The Code also specifies that no live load for the roof of the

Table 19.7. Percentage of Design Live Load to be Considered for Seismic Load Calculation

Live Load Class (kg/m^2)	Design Live Load Percentage
200, 250, and 300	25
400, 500, 750, and 1,000	50

building be included in the calculations. The Code states that when the live load is reduced for earthquake effects, no further reduction in live load can be made, such as specified by IS:875-1987 (the usual reduction in live load allowed for the design of foundation and columns of lower stories).

19.10 BASE SHEAR FORCE AND ITS DISTRIBUTION WITH HEIGHT

19.10.1 Seismic Coefficient Method

In the seismic coefficient method, the design base shear V is obtained from the following formula:

$$V = KC\beta I\alpha_0 W \tag{19.3}$$

where W is the total dead load plus the appropriate live load (Section 19.9). The Code specifies a parabolic distribution of seismic force with respect to height, given by

$$F_i = V\frac{W_i h_i^2}{\sum_{j=1}^{n} w_j h_j^2} \tag{19.4}$$

where

F_i = lateral force at the ith floor (or roof)
W_i = gravity load (dead load plus appropriate amount of live load) at the ith floor
h_i = height measured from the base of the building to the ith floor
n = number of stories.

The gravity load W_i at any floor is to be obtained by equally distributing the weight of walls and columns, in any story, to the floor above and the floor below.

The Code provides that when the basement walls are not connected with the ground floor deck or the basement walls are not fitted between building columns, the number of stories (n) in eq.(19.4) is to include the basement stories. Otherwise n excludes the basement stories. This amounts to assuming that the building is to be fixed at the ground floor, if the basement walls are connected to the ground floor or between the columns.

19.10.2 Response Spectrum Procedure

In this method, natural frequencies and mode shapes are to be obtained by a free vibration analysis. For each significant natural mode, the average acceleration coefficient S_a/g is obtained from Fig. 19.3. The seismic design lateral load F_{ir} applied at the ith floor level corresponding to the rth mode of vibration is given by the following equation:

$$F_{ir} = K\beta I F_0 \phi_{ir} C_r \frac{S_{ar}}{g} W_i \tag{19.5}$$

where

ϕ_{ir} = mode shape coefficient at ith floor in rth mode of vibration
C_r = modal participation factor for the rth mode given by

$$C_r = \frac{\sum_{j=1}^{n} W_j \phi_{jr}}{\sum_{j=1}^{n} W_j [\phi_{jr}]^2} \tag{19.6}$$

If the absolute values of the forces F_{ir} were combined for the different modes, the resultant force would be too conservative because in such a combination the sign of opposing forces in the higher modes is lost. Instead, the Code specified that modal story *shears* be combined first, and that the final lateral forces be obtained from this combination. The Code provides for combination of different modes by using the Square Root of Sum of Squares (SRSS) for buildings taller than 90 m, and a modified version of the SRSS for buildings under 90 m in height. The shear force V_i for the ith story is to be obtained by superposition of the first three modes as

Table 19.8. Values of the Coefficient γ

Height, H (m)	γ
Up to 20	0.40
40	0.60
60	0.80
90	1.00

$$V_i = (1-\gamma)\sum_{r=1}^{3} |V_{ir}| + \gamma \sqrt{\sum_{r=1}^{3} [V_{ir}]^2} \tag{19.7}$$

where V_{ir} = maximum shear at the ith story corresponding to the rth mode; and where the value for the coefficient γ is given in Table 19.8. For buildings of intermediate height, values of γ may be obtained by linear interpolation.

The total lateral loads F_n acting at roof level n and F_i acting at the ith floor level, are back-calculated from the story shear using the following equations:

$$F_n = V_n \tag{19.8a}$$

$$F_i = V_i - V_{i+1} \tag{19.8b}$$

19.11 TORSION

The Code does not provide for a minimum design eccentricity due to accidental torsion. In case of an eccentricity between the center of stiffness at a story and the center of the above mass, the Code stipulates that torsional moments shall be calculated with a design eccentricity equal to 1.5 times the actual eccentricity. However, reduction in seismic shear in a frame due to torsion is to be ignored. The requirement of torsion analysis is particularly emphasized for buildings more than 40 m high.

19.12 STORY DRIFT

The Code requires that the maximum relative displacement (story drift) between two successive floors due to design seismic forces must not exceed 0.004 times the story height. The Code emphasizes that this check is particularly necessary for buildings more than 40 m high.

19.13 CANTILEVERS AND PROJECTIONS

The vertical and horizontal projections of a building, e.g., towers, parapets, stacks, and balconies, are

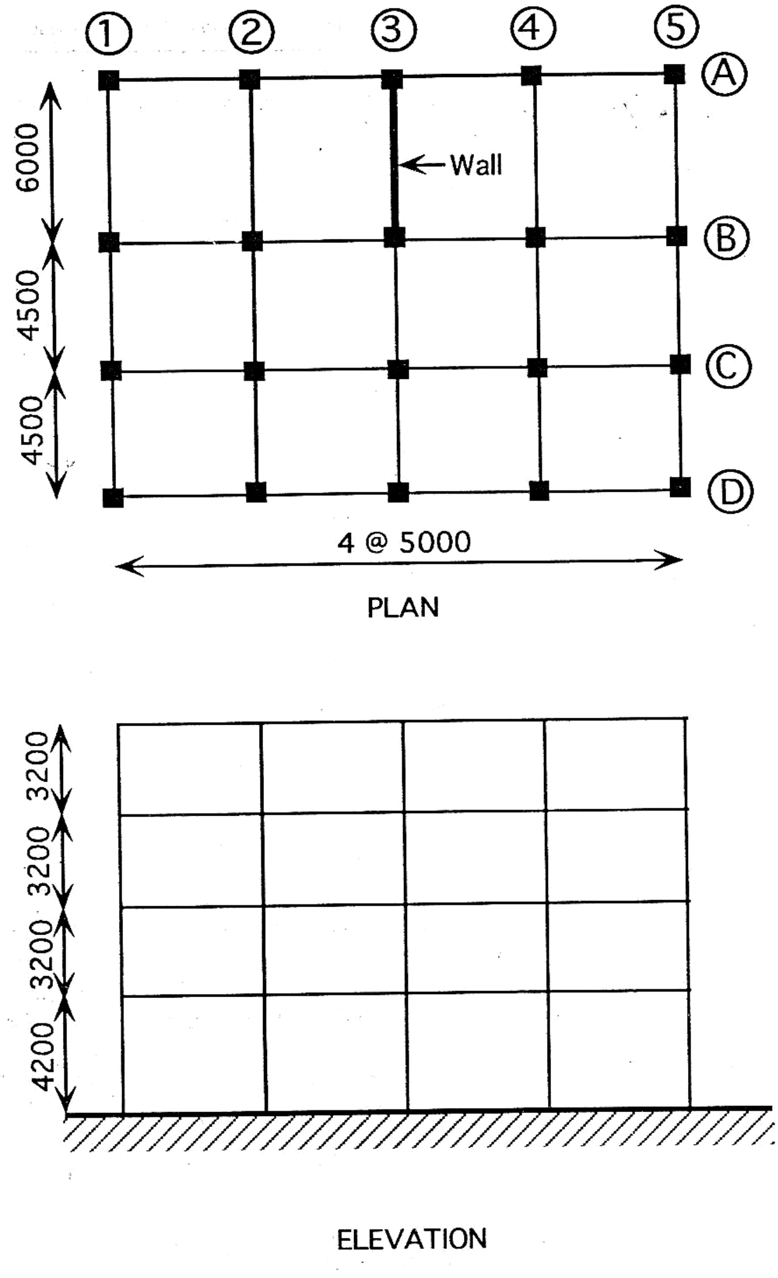

Fig. 19.4(a). Configuration of the building for Example 19.1

particularly vulnerable to damage during an earthquake. Therefore, the Code requires that the vertical projections and their connections with the structure (not the supporting building) be designed for five times the basic seismic coefficient (α_0). Similarly, horizontal projections and their connections are to be designed for five times the vertical seismic coefficient (e.g., $5 \times 0.5\alpha_0$).

19.14 EXAMPLE 19.1

Consider a four-story reinforced concrete office building as shown in Fig. 19.4(a). The building is located in Shillong (Seismic Zone V). The soils are medium stiff and the entire building is supported on a raft foundation. The column size is $40\,\text{cm} \times 40\,\text{cm}$, all outer beams are $25\,\text{cm} \times 60\,\text{cm}$, and all inner beams are

30 cm × 45 cm. The shear wall in the Y direction is 20 cm thick. All construction is in M20 concrete (cube-crushing strength = 20 MPa). The lumped weight due to dead loads is 12 kN/m^2 on floors and 10 kN/m^2 on the roof, and the live loads are 4 kN/m^2 on floors and 1.5 kN/m^2 on the roof. The floors are of cast-in-place reinforced concrete and provide rigid diaphragm action.

Determine the design seismic forces on the structure by (a) the seismic coefficient method (and distribute these forces to different frames and the wall), and (b) the response spectrum method. Also, compare the results obtained with these two methods.

Solution

Design parameters. For Seismic Zone V, the basic horizontal seismic coefficient α_0 for the seismic coefficient method is 0.08, and the seismic zone factor F_0 for the response spectrum method is 0.40 (Table 19.3). Because the building is supported on a raft foundation, factor β for the soil-foundation system is 1.0 (Table 19.4). For an office building, the importance factor I is 1.0 (Table 19.5). It is assumed that the building will be designed for ductility according to the provisions of IS:4326, hence the performance factor K is 1.0 (Table 19.6).

Lumped weights. The floor area is $15 \times 20 = 300\,\text{m}^2$. The live load class is [3]400 kg/m^2, thus only 50% of the live load is lumped at the floors (Table 19.7). At the roof, no live load is to be lumped. Hence, the total lumped weight on the floors and the roof is:

Floors: $W_1 = W_2 = W_3 = 300 \times (12 + 0.5 \times 4)$
$= 4{,}200\,\text{kN}$

Roof: $W_4 = 300 \times (10) = 3{,}000\,\text{kN}$

Total weight of the structure:

$$W = \Sigma W_i = 3 \times 4{,}200 + 3{,}000 = 15{,}600\,\text{kN}.$$

(a) Seismic coefficient method

- **Fundamental period.** For earthquake motion in the X direction, lateral load resistance is provided by moment-resisting frames without bracing or shear walls, hence eq.(19.1) is applicable. In the Y direction, the building has a shear wall, hence eq.(19.2) has to be applied. Thus,

In X direction: $T = 0.1n = 0.1 \times 4 = 0.40\,\text{sec}$

$$\text{In } Y \text{ direction:}\quad T = \frac{0.09H}{\sqrt{d}} = \frac{0.09 \times 13.8}{\sqrt{15}} = 0.32\,\text{sec}$$

- **Design base shear.** From Figure 19.2:

In X direction: $C = 0.9$ (for $T = 0.40$ sec)
In Y direction: $C = 1.0$ (for $T = 0.32$ sec)

Also, $\alpha_0 = 0.08$, $I = 1.0$, $K = 1.0$, and $\beta = 1.0$. Hence, from eq.(19.3), the design base shear is

In X direction:
$V = 1.0 \times 0.9 \times 1.0 \times 1.0 \times 0.08 \times 15{,}600$
$= 1{,}123\,\text{kN}$

In Y direction:
$V = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 0.08 \times 15{,}600$
$= 1{,}248\,\text{kN}$

- **Force distribution with height.** The design base shear is to be distributed with height as per eq.(19.4). Table 19.9 gives the calculations and the resulting lateral forces in the X and Y directions.
- **Force distribution in frames (Y direction).** In the Y direction, the building is symmetrical, therefore no torsion takes place. The monolithic reinforced concrete slab provides a rigid diaphragm action. Therefore, the lateral forces calculated in Table 19.9 should be distributed to different frames and the wall such that the lateral displacements in the wall and in the frames, at each level, will be the same. The mode of deformation in the wall and the in frames differs; the wall tends to deform like a flexural cantilever beam, while the frames have a tendency to deform like a

Table 19.9. Lateral Load Distribution with Height by the Seismic Coefficient Method

Story Level i	W_i (kN)	h_i (m)	$W_i h_i^2$ ($\times 10^3$)	$\frac{W_i h_i^2}{\Sigma W_i h_i^2}$	Lateral Force for direction (kN)	
					X	Y
4	3,000	13.8	571.3	0.424	476	529
3	4,200	10.6	471.9	0.350	393	437
2	4,200	7.4	230.0	0.171	192	213
1	4,200	4.2	74.1	0.055	62	69
Σ			1,347.3	1.000	1,123	1,248

[3]One kilogram weight = 9.8 Newtons. However, in engineering practice, the conversion factor is rounded to 10.

shear beam. An accurate computer analysis ensures force distribution among the frames/walls such that displacement compatibility is enforced at all levels. However, when using an approximate manual calculation, it is common practice to ensure only equal displacement at the roof level. This procedure then consists of (Macleod 1971) (i) lumping the walls into one equivalent wall and the frames into one equivalent frame, (ii) applying the vertically distributed seismic forces to the equivalent wall, which is assumed to interact with the equivalent frame only at the roof level, (iii) calculating the interaction force at roof level by ensuring equal displacement at the roof, and (iv) distributing the resulting forces on the equivalent wall and the equivalent frame to the different real walls and frames, respectively, in proportion to their lateral stiffness. Such a calculation may underestimate the shear force for the middle stories of the frames by as much as 30%. For this reason it is the usual practice to increase the calculated force in the frame by 30%.

The lateral stiffness of frames can be determined either by using a computer program analysis or by using approximate manual calculations. Herein the stiffness has been calculated by an approximate procedure (Macleod 1971). The modeling assumptions include (i) columns rigidly fixed at the base, (ii) gross-section moment of inertia (rectangular section for beams), (iii) finite flexibility of beams, and (iv) finite size of beam-column joints, which provide rigid zones at either end of the beams and the columns. The wall stiffness has been obtained considering (i) flexural as well as shear deformations, (ii) gross area of cross-section of the wall including the two columns at both ends of the wall, and (iii) the wall rigidly fixed at the base because of the raft foundation. The modulus of elasticity of concrete has been assumed equal to 25,500 MPa. Under these conditions, the following values of lateral displacement at roof level that are due to lateral forces, and the resulting lateral stiffnesses in individual resisting elements, are obtained [see Fig. 19.4(b)]:

1,000 kN at roof	Deflection (m)	Stiffness (kN/m)
Frames 1 and 5	0.0822	12,200
Frames 2 and 4	0.1320	7,580
Frame 3 excluding wall	0.220	4,540
Wall	0.00704	
Wall with 1,248 kN distributed (Fig.19.4b)	0.00645	

Net deflection in wall (Δ_w) and in the equivalent frame (Δ_f) are [(Fig. 19.4(b)]:

$$\Delta_w = 0.00645 - \frac{P \times 0.00704}{1{,}000}$$

$$\Delta_f = \frac{P}{44{,}100}$$

$$\Delta_w = \Delta_f \quad \text{gives} \quad P = 217\,\text{kN}$$

As mentioned earlier, the shear in frames is increased by 30% to account for approximations. Hence, the frames would be designed for a seismic shear of 282 kN, which is 22.6% of the total base shear. Because the Code requires that frames be designed for a minimum of 25% of the total base shear, the design force for frames will be 312 kN (= 0.25 × 1,248 kN). This force is distributed further to the five frames in proportion to the lateral stiffness. Thus, design forces for different frames and the wall are [(Fig. 19.4(c)]:

Frames 1 and 5:

$$\frac{312 \times 12{,}200}{44{,}100} = 86.3\,\text{kN (force applied at roof)}$$

Frames 2 and 4:

$$\frac{312 \times 7{,}580}{44{,}100} = 53.6\,\text{kN (force applied at roof)}$$

Frame 3 (excluding wall):

$$\frac{312 \times 4{,}540}{44{,}100} = 32.1\,\text{kN (force applied at roof)}$$

Wall: At roof = (529 − 217) = 312 kN
At third floor = 437 kN
At second floor = 213 kN
At first floor = 69 kN

- **Force distribution in frames (*X* direction).** The earthquake force in the X direction is resisted by four moment resisting frames that are not symmetrically placed; this placement causes torsion. The lateral stiffness of these frames calculated by the approximate method of Macleod is shown in Fig. 19.4(d).

$$\text{Total stiffness in } X \text{ direction} = 2 \times 15{,}850 + 2 \times 9{,}940 = 51{,}580\,\text{kN/m}$$

The distance d of the center of stiffness from frame D is

$$d = \frac{15{,}850 \times 15 + 9{,}940 \times 9 + 9{,}940 \times 4.5}{51{,}580} = 7.21\,\text{m}$$

Calculated eccentricity between the center of mass and the center of stiffness is $e = 7.5 - 7.21 = 0.29$ m. The building will be designed for an eccentricity of $1.5e$ ($= 1.5 \times 0.29 = 0.435$ m). Thus, the lateral force V at the center of mass C_m can be represented as a lateral force V at the center of stiffness C_s, and with a torsional moment of magnitude $M_t = 1.5eV$ [Fig. 19.4(e)].

The lateral load at the center of stiffness is to be distributed in proportion to the frame stiffness. Hence,

$$\text{Force in frames A and D} = \frac{15{,}850}{51{,}580} V = 0.307V$$

$$\text{Force in frames B and C} = \frac{9{,}940}{51{,}580} V = 0.193V$$

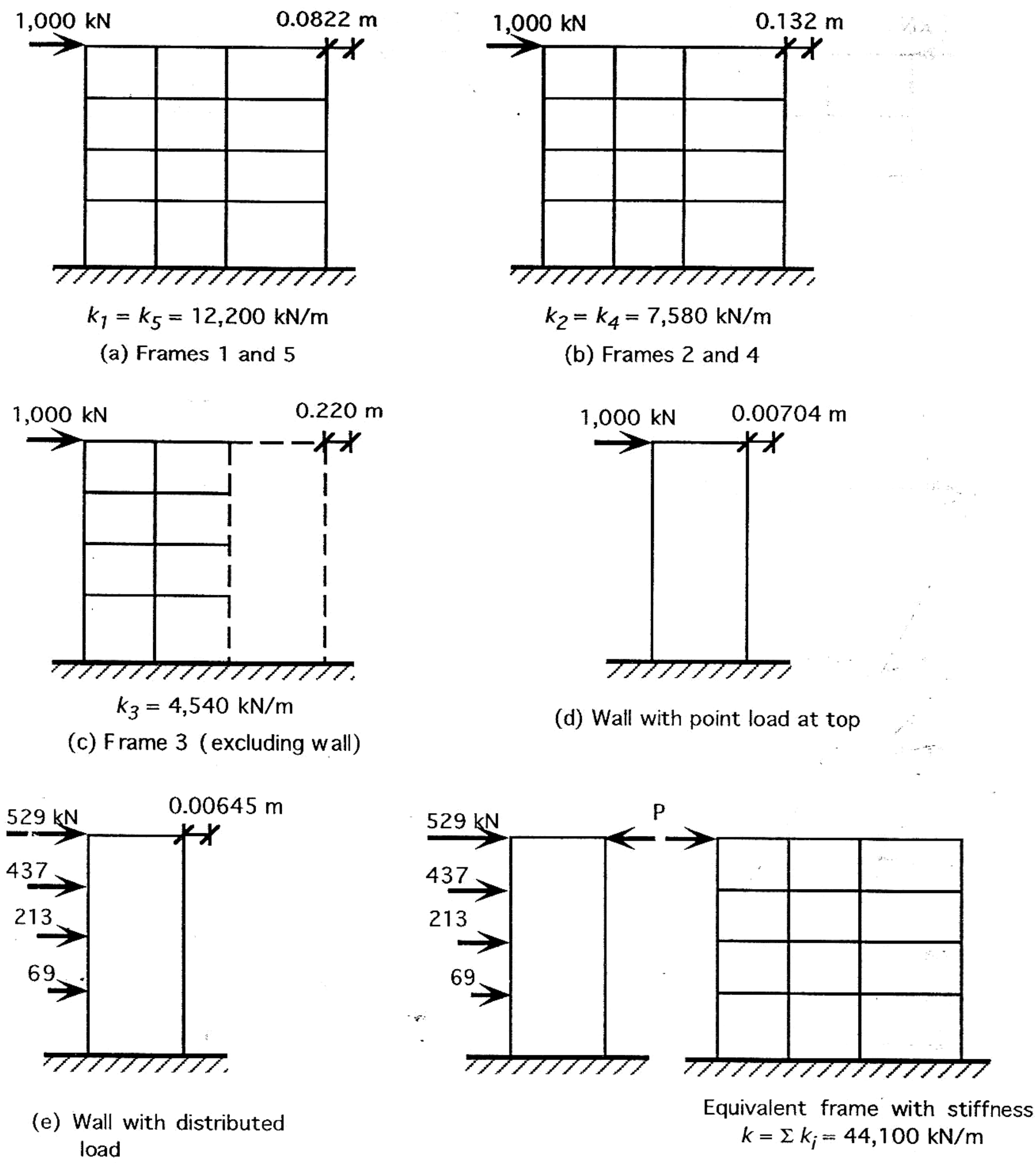

Fig. 19.4(b). Lateral stiffness of frames and wall in *Y* direction

The torsional moment applied at the center of stiffness will be resisted by frames A, B, C, D, 1, 2, 4, and 5. Frame 3 and the shear wall are at zero distance from the center of stiffness and do not provide resistance to torsion. Force in the *i*th frame is given by $M_t(k_i r_i)/(\Sigma k_i r_i^2)$; where k_i = stiffness of the *i*th frame; r_i = distance of the *i*th frame from the center of stiffness; and M_t = torsional moment (= $1.5eV$). Table 19.10 shows the calculations necessary to determine the forces developed by torsion on the various resisting elements of the building. The negative sign for force in frames C and D indicates that the force due to torsion is in a direction opposite to that due to direct lateral force. Such a reduction in frame force due to torsion is to be ignored as stipulated by the code. The forces in frames 1, 2, 4, and 5 caused by torsion as a result of earthquake motion in the *X* direction are small as compared to the forces in these frames resulting from

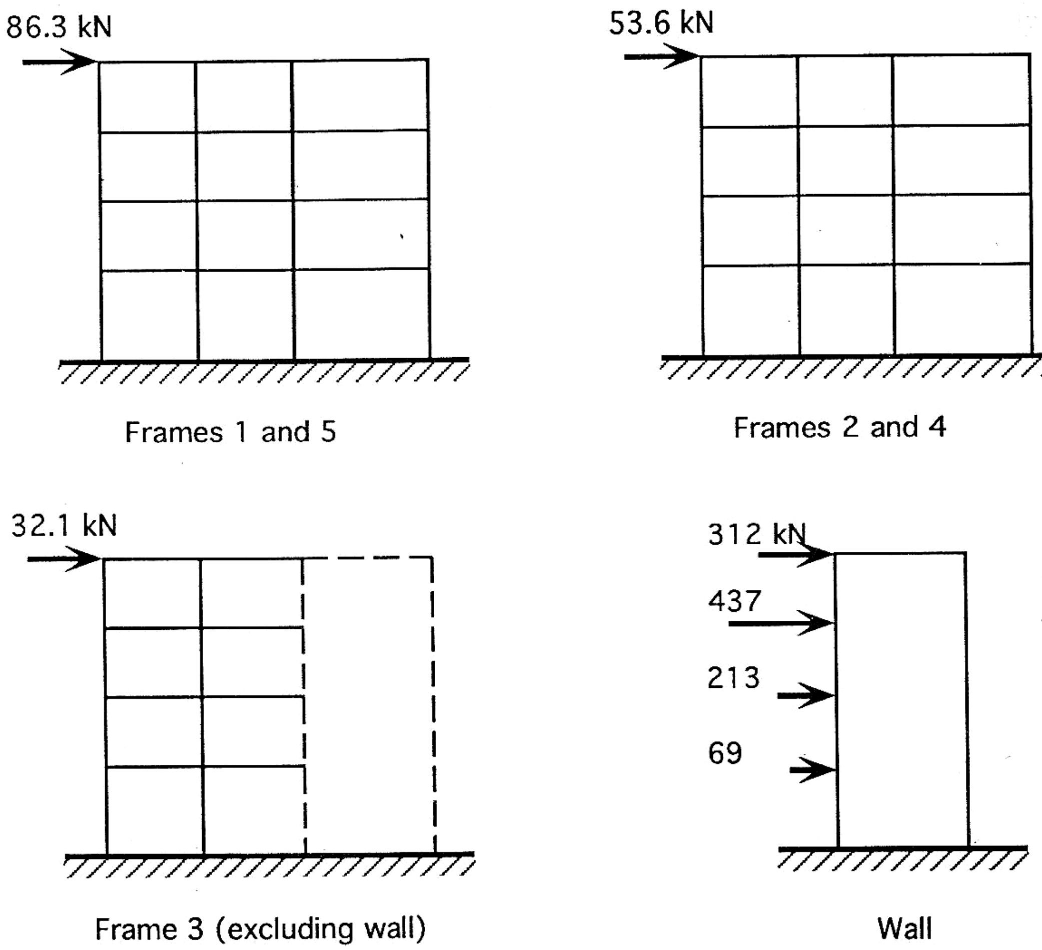

Fig. 19.4(c). Design force in frames and wall for earthquake in *Y* direction

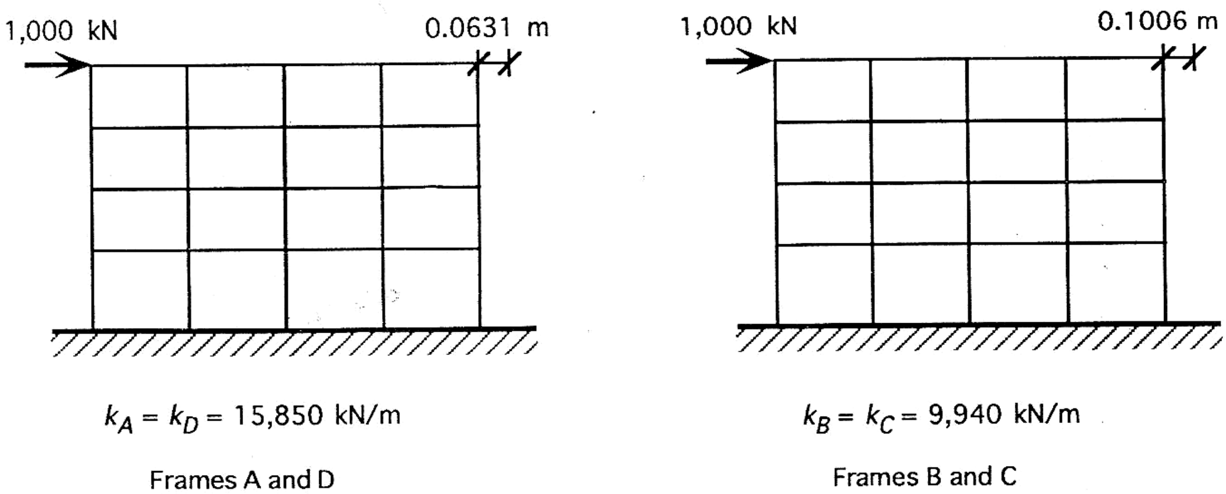

Fig. 19.4(d). Lateral stiffness of frames in *X* direction

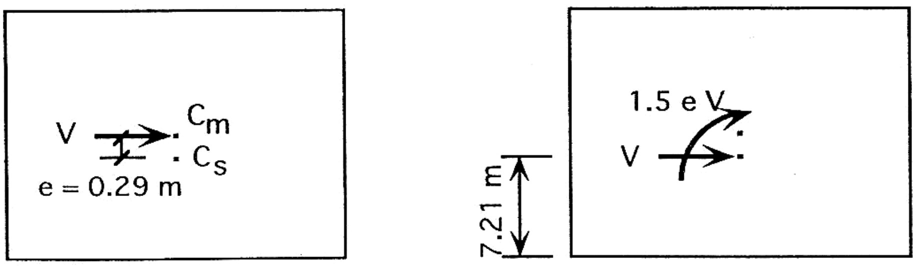

Fig. 19.4(e). Calculated eccentricity and design torsional moment

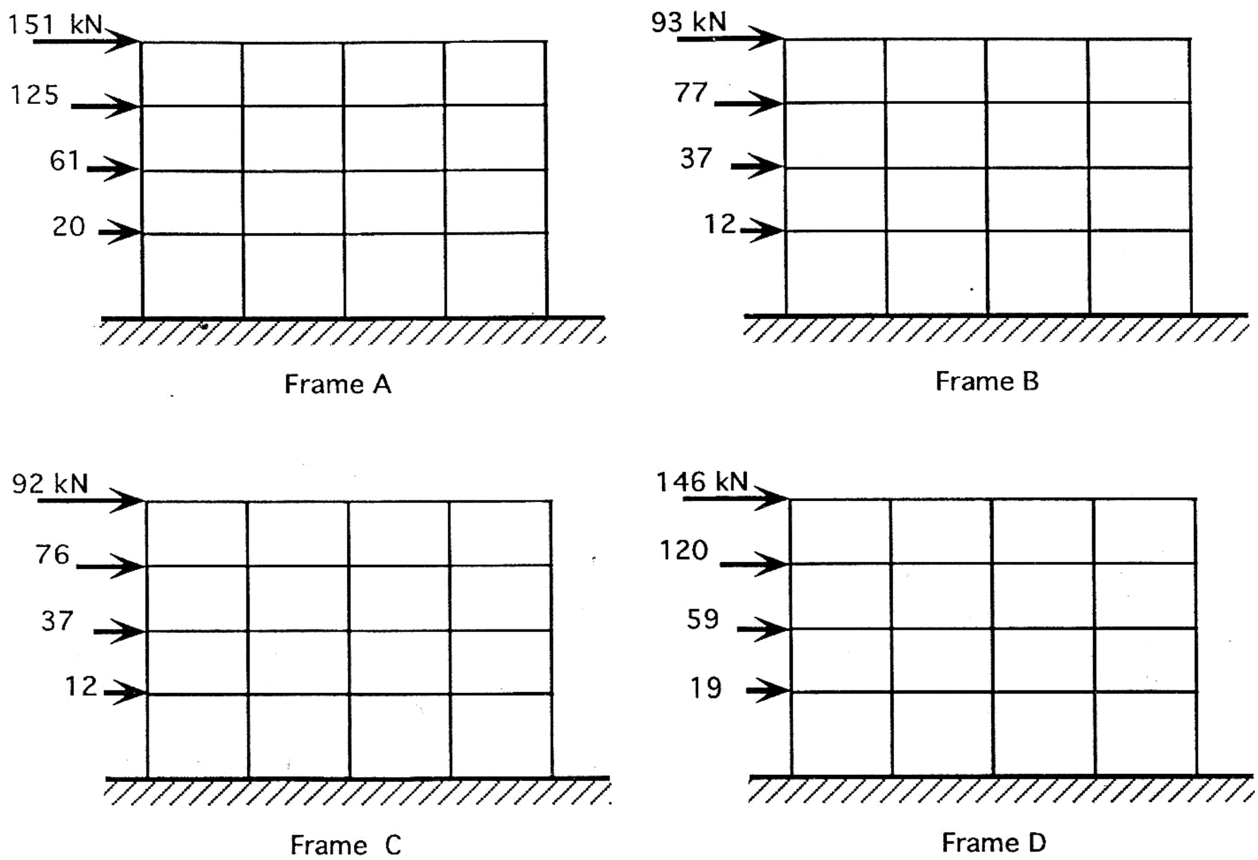

Fig. 19.4(f). Design forces in frames for earthquake in X direction

earthquake motion in the Y direction. Therefore, forces caused by torsion do not affect the design of these frames. Fig.19.4(f) shows the design forces in frames and in the wall for earthquake motion in the X direction.

(b) Response spectrum method. This method requires a free-vibration analysis to determine the natural periods and corresponding modal shapes of the building. The Code requires three modes to be superposed; therefore, the free-vibration analysis must yield at least three modes in each of the two main directions of the building. A free-vibration analysis of the building was performed using a computer program; the building properties and the modeling assumptions were the same as those used in the solution of part (a). The natural periods, the mode shapes, and the modal participation factor thus obtained are shown in Table 19.11. The Code does not specify the value of

Table 19.10. Calculation of Force in Different Frames due to Torsional Moment

Frame	Stiffness ($\times 10^3$ kN/m)	r_i (m)	$k_i r_i$ ($\times 10^3$ kN)	$k_i r_i^2$ ($\times 10^3$)	$\frac{k_i r_i}{\Sigma k_i r_i^2}$	Force in Frame due to Torsion	Force in Frame due to Direct Force	Total Design Force
A	15.85	7.79	123.5	961.8	0.0206	$0.0114V$	$0.307V$	$0.318V$
B	9.94	1.79	17.8	31.8	0.0038	$0.0017V$	$0.193V$	$0.195V$
C	9.94	−2.71	−26.9	73.0	−0.0057	$-0.0025V$	$0.193V$	$0.193V$
D	15.85	−7.21	−114.3	823.9	−0.0241	$-0.0105V$	$0.307V$	$0.307V$
1	12.20	−10.0	−122.0	1,220.0	−0.0258	$0.0112V$	—	—
2	7.58	−5.0	−37.9	189.5	−0.0080	$0.0035V$	—	—
4	7.58	5.0	37.9	189.5	0.0080	$0.0035V$	—	—
5	12.20	10.0	122.0	1,220.0	0.0258	$0.0112V$	—	—
Σ				4,709.6				

Table 19.11. Free-Vibration Properties of the Building of Example 19.1

	X direction			*Y* direction		
	Mode 1	Mode 2	Mode 3	Mode 1	Mode 2	Mode 3
Natural Period (sec)	0.860	0.265	0.145	0.303	0.057	0.021
			Mode Shape			
Roof	1.000	1.000	1.000	1.000	1.000	1.000
3rd Floor	0.904	0.216	−0.831	0.690	−0.327	−1.407
2nd Floor	0.716	−0.701	−0.574	0.393	−0.986	0.040
1st Floor	0.441	−0.921	1.016	0.147	−0.694	1.636
Modal Participation Factor	1.240	−0.329	0.118	1.423	−0.568	0.183

damping to be adopted; 5% damping has been considered appropriate.

The lateral force F_{ir} acting at the ith floor in the rth mode is

$$F_{ir} = K\beta I F_0 \phi_{ir} C_r \frac{S_{ar}}{g} W_i$$

$$= 1.0 \times 1.0 \times 1.0 \times 0.40 \times C_r \frac{S_{ar}}{g} W_i \phi_{ir} \qquad \text{[eq.(19.5)]}$$

$$= 0.40 C_r \frac{S_{ar}}{g} W_i \phi_{ir}$$

The modal participation factor C_r is given by eq.(19.6) while the acceleration coefficient (S_a/g) for different modes is obtained from Fig. 19.3. Calculated lateral forces at different levels in each mode are as follows:

Earthquake in X direction.

Mode 1 ($T = 0.860$): $S_a/g = 0.12$; $F_{i1} = 0.05952 W_i \phi_{i1}$
Mode 2 ($T = 0.265$): $S_a/g = 0.20$; $F_{i2} = 0.02632 W_i \phi_{i2}$
Mode 3 ($T = 0.145$): $S_a/g = 0.20$; $F_{i3} = 0.00944 W_i \phi_{i3}$

Earthquake in Y direction

Mode 1 ($T = 0.303$): $S_a/g = 0.20$; $F_{i1} = 0.11384 W_i \phi_{i1}$
Mode 2 ($T = 0.057$): $S_a/g = 0.18$; $F_{i2} = 0.04090 W_i \phi_{i2}$
Mode 3 ($T = 0.021$): $S_a/g = 0.12$; $F_{i3} = 0.00878 W_i \phi_{i3}$

Table 19.12 summarizes the calculation of lateral forces at different floors in each mode for earthquake motion in the X direction and the resulting story shear for corresponding modes. The contributions of different modes are combined by eq.(19.7), with building height = 13.8 m the value of α is 0.4. Thus, for earthquakes in the X direction,

$$V_1 = (1 - 0.4)(693.8 + 76.4 + 12.9) + 0.4[693.8^2 + 76.4^2 + 12.9^2]^{1/2} = 749.1 \text{ kN}$$

$$V_2 = 615.7 \text{ kN}$$

$$V_3 = 474.2 \text{ kN}$$

$$V_4 = 250.5 \text{ kN}$$

Table 19.12. Lateral Force Calculation by Response Spectrum Method (Earthquake in X Direction)

Floor	Weight W_i (kN)	Mode 1 ϕ_{i1}	Mode 1 F_{i1} (kN)	Mode 1 V_{i1} (kN)	Mode 2 ϕ_{i2}	Mode 2 F_{i2} (kN)	Mode 2 V_{i2} (kN)	Mode 3 ϕ_{i3}	Mode 3 F_{i3} (kN)	Mode 3 V_{i3} (kN)
4	3,000	1.000	178.6	178.6	1.000	79.0	79.0	1.000	28.3	28.3
3	4,200	0.904	226.0	404.5	0.216	23.9	102.9	−0.831	−32.9	4.6
2	4,200	0.716	179.0	583.5	−0.701	−77.5	25.4	−0.574	−22.8	27.4
1	4,200	0.441	110.2	693.8	−0.921	−101.8	76.4	1.016	40.3	12.9

Table 19.13. Lateral Force Calculation by Response Spectrum Method (Earthquake in Y Direction)

Floor	Weight W_i (kN)	Mode 1 ϕ_{i1}	Mode 1 F_{i1} (kN)	Mode 1 V_{i1} (kN)	Mode 2 ϕ_{i2}	Mode 2 F_{i2} (kN)	Mode 2 V_{i2} (kN)	Mode 3 ϕ_{i3}	Mode 3 F_{i3} (kN)	Mode 3 V_{i3} (kN)
4	3,000	1.000	341.5	341.5	1.000	122.7	122.7	1.000	26.4	26.4
3	4,200	0.690	329.9	671.4	−0.327	−56.2	66.5	−1.407	−51.9	25.5
2	4,200	0.393	187.9	859.3	−0.986	−169.4	102.8	−0.040	1.5	24.0
1	4,200	0.147	70.3	929.6	−0.694	−119.2	222.0	1.636	60.4	36.4

Table 19.14. Comparison of Design Seismic Forces

Floor	Seismic Coefficient Method X direction (kN)	Seismic Coefficient Method Y direction (kN)	Response Spectrum Method X direction (kN)	Response Spectrum Method Y direction (kN)
4	476	529	251	440
3	393	437	224	288
2	192	213	142	210
1	62	69	133	157
Base Shear (kN)	1,123	1,248	750	1,095
Base Moment (kN-m)	12,416	13,798	7,448	11,338

The externally applied design forces are obtained from eq.(19.8) as

$$F_4 = V_4 = 250.5\,\text{kN}$$

$$F_3 = V_3 - V_4 = 474.2 - 250.5 = 223.7\,\text{kN}$$

$$F_2 = V_2 - V_3 = 615.7 - 474.2 = 141.5\,\text{kN}$$

$$F_1 = V_1 - V_2 = 749.1 - 615.7 = 133.4\,\text{kN}$$

In a similar manner, the lateral force and story shear forces are calculated in the Y direction for different modes (Table 19.13). The combined story shears are

$$V_1 = 1{,}095\,\text{kN},\ V_2 = 938\,\text{kN},\ V_3 = 728.1\,\text{kN},\ V_4 = 440.0\,\text{kN}$$

The externally applied design loads are obtained from eq.(19.8) as

$$F_4 = 440.0\,\text{kN},\ F_3 = 288.1\,\text{kN},\ F_2 = 210.0\,\text{kN},\ F_1 = 157.0\,\text{kN}$$

Table 19.14 compares the design seismic forces obtained by the two procedures: the seismic coefficient method and the response spectrum method. It should be apparent that the response spectrum method underestimates the design force by 33% and 12%, respectively, for the X and the Y directions, as compared to the design values obtained by the seismic coefficient method. The discrepancy is large in the X direction because the free-vibration analysis on a bare frame (ignoring effects of filler walls) gives a high value of the fundamental period (0.86 sec) as compared to 0.40 sec used in the seismic coefficient method. In the Y direction, the reinforced concrete wall provides significant stiffness, hence the bare-frame fundamental period by free-vibration analysis (0.303 sec) is close to the value of 0.32 sec used in the static method. As a result, in the Y direction, the difference in design shear using the two procedures is not so large.

19.15 COMPUTER PROGRAM AND EXAMPLES

A computer program has been developed for the evaluation of the design seismic force on a multistory building according to the provisions of IS:1893 (1984). The program can be used in an interactive mode or through an input data file. In the seismic coefficient method, the program allows users to calculate the fundamental period by either of the expressions prescribed in the Code, or by the use of a value that the user obtains through dynamic analysis or experimentation, which are options provided by the Code. In the response spectrum method, the natural frequencies and mode shapes of the building are input as data. The program calculates the design base shear, design base moment, and design seismic forces at the different levels of the building. The user may opt for an approximate distribution (Macleod 1971) of design seismic forces to different frames and/or walls, based on the rigid floor diaphragm assumption. In this case, the user must provide properties such as dimensions and concrete grade for each frame and wall. The procedure is valid for uniform frames and walls; if there is some nonuniformity in span, story height, or member sizes, average properties should be given.

Example 19.2

Solve the problem of Example 19.1 using the computer program.

Solution

The results obtained by the program differ slightly from those obtained by manual calculations because the values of C (in the seismic coefficient method) and S_a/g (in the response spectrum method) interpolated by the program differ slightly from those interpolated from the plot for S_a/g. The following is the output file for this example, which also echoes the input data.

```
(a) Seismic Coefficient Method

Input file name   : EX2A.IN
Output file name  : EX2A.OUT
Title of problem  : EXAMPLE 19.2A

              Method selected : Seismic Coefficient Method

Performance factor                               K =  1
Coefficient for soil foundation system           ß =  1
Importance factor                                I =  1
Seismic zone (1,2,3,4,5)                         Z =  5
Number of stories                                N =  4
Maximum base length in X-direction              Dx =  20 m
Maximum base length in Y direction              Dy =  15 m

  Level          Story height (m)        Weight (kN)
    4                 3.2                  3000.0
    3                 3.2                  4200.0
    2                 3.2                  4200.0
    1                 4.2                  4200.0

              Fundamental period (sec)        Value of C

 X-direction          0.400                     0.920
 Y-direction          0.321                     1.000

                      Base shear (kN)         Base moment (kNm)
   X-direction           1148.16                  12697.22
 Y-direction             1248.00                  13801.33

    Level              Design seismic force (kN)
                       X-direction          Y-direction
   4                      487                   529
   3                      402                   437
   2                      196                   213
   1                       63                    69

  Design Force Distribution to Different Frames and Walls
  (with Rigid Floor Diaphragms)
  (MacLeod, 1971)
---------------------------------------------------------
FRAMES IN X-DIRECTION:
---------------------
     Number of frames =  4   :  Types of frame =  2

Frames Type 1

     Total  width of frame in X-direction = 20.000
     Distance between column center lines =  5.000
     ΣI of columns at top of frame        = 0.1067E-01
     ΣI of columns at bottom of frame     = 0.1067E-01
     ΣI of beams at top of frame          = 0.1800E-01
     ΣI of beams at bottom of frame       = 0.1800E-01
     Area of ext. cols at top of frame    =  0.160
     Area of ext. cols at bottom of frame =  0.160
     Width of column                      =  0.400
     Depth of beam                        =  0.600
     Grade of concrete used in columns    = M20
     Grade of concrete used in  beams     = M20

Frames Type 2 :

     Total  width of frame in X-direction = 20.000
     Distance between column center lines =  5.000
     ΣI of columns at top of frame        = 0.1067E-01
     ΣI of columns at bottom of frame     = 0.1067E-01
     ΣI of beams at top of frame          = 0.9112E-02
     ΣI of beams at bottom of frame       = 0.9112E-02
     Area of ext. cols at top of frame    =  0.160
     Area of ext. cols at bottom of frame =  0.160
     Width of column                      =  0.400
     Depth of beam                        =  0.450
     Grade of concrete used in columns    = M20
     Grade of concrete used in  beams     = M20

 Frame      Type      Y location
   A          1         15.000
   B          2          9.000
   C          2          4.500
   D          1          0.000

FRAMES IN Y-DIRECTION:
---------------------
     Number of frames =  5   :  Types of frame =  3

Frames Type 1 :

     Total  width of frame in Y-direction = 15.000
     Distance between column center lines =  5.000
     ΣI of columns at top of frame        = 0.8533E-02
     ΣI of columns at bottom of frame     = 0.8533E-02
     ΣI of beams at top of frame          = 0.1350E-01
     ΣI of beams at bottom of frame       = 0.1350E-01
     Area of ext. cols at top of frame    =  0.160
     Area of ext. cols at bottom of frame =  0.160
     Width of column                      =  0.400
     Depth of beam                        =  0.600
     Grade of concrete used in columns    = M20
     Grade of concrete used in beams      = M20
```

Frames Type 2 :

Total width of frame in Y-direction	=	15.000
Distance between column center lines	=	5.000
ΣI of columns at top of frame	=	0.8533E-02
ΣI of columns at bottom of frame	=	0.8533E-02
ΣI of beams at top of frame	=	0.6834E-02
ΣI of beams at bottom of frame	=	0.6834E-02
Area of ext. cols at top of frame	=	0.160
Area of ext. cols at bottom of frame	=	0.160
Width of column	=	0.400
Depth of beam	=	0.450
Grade of concrete used in columns	=	M20
Grade of concrete used in beams	=	M20

Frames Type 3 :

Total width of frame in Y-direction	=	4.500
Distance between column center lines	=	4.500
ΣI of columns at top of frame	=	0.4267E-02
ΣI of columns at bottom of frame	=	0.4267E-02
ΣI of beams at top of frame	=	0.4556E-02
ΣI of beams at bottom of frame	=	0.4556E-02
Area of ext. cols at top of frame	=	0.160
Area of ext. cols at bottom of frame	=	0.160
Width of column	=	0.400
Depth of beam	=	0.450
Grade of concrete used in columns	=	M20
Grade of concrete used in beams	=	M20

Frame	Type	X location
1	1	0.000
2	2	5.000
3	3	10.000
4	2	15.000
5	1	20.000

WALLS IN Y-DIRECTION:

Number of walls = 1 : Types of wall = 1

Walls Type 1 :

Moment of inertia	=	5.811
Shear area	=	1.200

Wall	Type	X location
1	1	10.000

ECCENTRICITY CALCULATION:

Center of mass	: (10.000, 7.500)	: given
Center of stiffness	: (10.000, 7.211)	: calculated
Eccentricity (e)	: (0.000, 0.289)	
Design eccentricity (1.5e)	: (0.000, 0.434)	

FRAME AND WALL STIFFNESSES:

Frames in X-direction

Frame	Stiffness (kN/m) (Point load at top)
A	15843.44
B	9933.17
C	9933.17
D	15843.44

Frames in Y-direction

Frame	Stiffness (kN/m) (Point load at top)
1	12154.84
2	7578.51
3	4540.91
4	7578.51
5	12154.84

Walls in Y-direction

Wall	Stiffness (kN/m) (Point load at top)	Stiffness (kN/m) (Actual seismic load distribution)
1	142010.00	193288.80

FORCE DISTRIBUTION AMONG FRAMES IN X-DIRECTION

1	Frame A	Frame B	Frame C	Frame D
4	155.17	94.61	93.81	149.63
3	128.17	78.15	77.49	123.59
2	62.46	38.09	37.76	60.23
1	20.12	12.27	12.17	19.40

FORCE DISTRIBUTION AMONG FRAMES IN Y-DIRECTION

Level	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
4	59.91	37.36	22.38	37.36	59.91
3	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00
1	0.00	0.00	0.00	0.00	0.00

For wall-frame system, the frame force thus obtained are usually increased by 30%. These are:

Level	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
4	77.89	48.56	29.10	48.56	77.89
3	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00
1	0.00	0.00	0.00	0.00	0.00

The seismic force in frames as above is less than 25% of the total design seismic force. Hence, as per code requirement, the design force for frames is to be increased to:

Level	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
4	86.17	53.73	32.19	53.73	86.17
3	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00
1	0.00	0.00	0.00	0.00	0.00

FORCE DISTRIBUTION AMONG SHEAR WALLS IN Y-DIRECTION

Level	Wall 1
4	312.29
3	437.13
2	213.04
1	68.63

(b) Response Spectrum Method

Input file name : EX2B.IN
Output file name : EX2B.OUT
Title of problem : EXAMPLE 19.2B

Method selected : Response Spectrum Method

Performance factor	K = 1
Coefficient for soil foundation system	β = 1
Importance factor	I = 1
Seismic zone (1,2,3,4,5)	Z = 5
Number of stories	N = 4
Percentage of damping (0,2,5,10,20)	= 5

Story level	Story height (m)	Weight (kN)
4	3.2	3000.0
3	3.2	4200.0
2	3.2	4200.0
1	4.2	4200.0

Mode Shapes in X-direction (given)

Level	MODE 1	MODE 2	MODE 3
4	1.000	1.000	1.000
3	0.904	0.216	-0.831
2	0.716	-0.701	-0.574
1	0.441	-0.921	1.016

Mode Shapes in Y-direction (given)

Level	MODE 1	MODE 2	MODE 3
4	1.000	1.000	1.000
3	0.690	-0.327	-1.407
2	0.393	-0.986	0.040
1	0.147	-0.694	1.636

	Time period (sec)	Value of Sa/g	Participation factor
X-direction:			
Mode 1	0.860	0.121	1.240
Mode 2	0.265	0.200	-0.329
Mode 3	0.145	0.200	0.118
Y-direction			
Mode 1	0.303	0.200	1.423
Mode 2	0.057	0.182	-0.568
Mode 3	0.021	0.132	0.183

LATERAL LOAD CALCULATION (Fi = Force, Vi = Story Shear)

(X-direction)

Level	Mode 1 Fi(1)	Mode 1 Vi(1)	Mode 2 Fi(2)	Mode 2 Vi(2)	Mode 3 Fi(3)	Mode 3 Vi(3)	SRSS value Vi	SRSS value Fi
4	180.3	180.3	-79.0	79.0	28.1	28.1	252.0	252.0
3	228.2	408.5	-23.9	102.9	-32.7	4.6	478.1	226.1
2	180.7	589.2	77.6	25.4	-22.6	27.2	621.3	143.1
1	111.3	700.6	101.9	76.5	40.0	12.8	755.9	134.6

LATERAL LOAD CALCULATION (Fi = Force, Vi = Story Shear)

(Y-direction)

Level	Mode 1 Fi(1)	Mode 1 Vi(1)	Mode 2 Fi(2)	Mode 2 Vi(2)	Mode 3 Fi(3)	Mode 3 Vi(3)	SRSS value Fi	SRSS value Vi
4	341.5	341.5	-124.2	124.2	29.0	29.0	442.6	442.6
3	329.9	671.4	56.8	67.3	-57.1	28.1	730.2	287.6
2	187.9	859.3	171.4	104.1	1.6	26.5	940.3	210.1
1	70.3	929.5	120.6	224.7	66.3	39.9	1099.3	159.1

	Base shear (kN)	Base moment (kNm)
X-direction	755.88	7499.28
Y-direction	1099.33	11378.92

Level	Design seismic force (kN) X-direction	Y-direction
4	252	443
3	226	288
2	143	210
1	135	159

Design Force Distribution to Different Frames and Walls
(with Rigid Floor Diaphragms)
MacLeod (1971)

FRAMES IN X-DIRECTION:

Number of frames = 4 : Types of frame = 2

Type 1 :

```
Total width of frame in X-direction      = 20.000
Distance between column center lines     = 5.000
7S1I of columns at top of frame          = 0.1067E-01
7S1I of columns at bottom of frame       = 0.1067E-01
7S1I of beams at top of frame            = 0.1800E-01
7S1I of beams at bottom of frame         = 0.1800E-01
Area of ext. cols at top of frame        = 0.160
Area of ext. cols at bottom of frame     = 0.160
Width of column                          = 0.400
Depth of beam                            = 0.600
Grade of concrete used in columns        = M20
Grade of concrete used in beams          = M20
```

Type 2 :

```
Total width of frame in X-direction      = 20.000
Distance between column center lines     = 5.000
7S1I of columns at top of frame          = 0.1067E-01
7S1I of columns at bottom of frame       = 0.1067E-01
7S1I of beams at top of frame            = 0.9112E-02
7S1I of beams at bottom of frame         = 0.9112E-02
Area of ext. cols at top of frame        = 0.160
Area of ext. cols at bottom of frame     = 0.160
Width of column                          = 0.400
Depth of beam                            = 0.450
Grade of concrete used in columns        = M20
Grade of concrete used in beams          = M20
```

Frame	Type	Y location
A	1	0.000
B	2	4.500
C	2	9.000
D	1	15.000

FRAMES IN Y-DIRECTION:

Number of frames = 5 : Types of frame = 3

Type 1 :

```
Total width of frame in Y-direction      = 15.000
Distance between column center lines     = 5.000
7S1I of columns at top of frame          = 0.8533E-02
7S1I of columns at bottom of frame       = 0.8533E-02
7S1I of beams at top of frame            = 0.1350E-01
7S1I of beams at bottom of frame         = 0.1350E-01
Area of ext. cols at top of frame        = 0.160
Area of ext. cols at bottom of frame     = 0.160
Width of column                          = 0.400
Depth of beam                            = 0.600
Grade of concrete used in columns        = M20
Grade of concrete used in beams          = M20
```

Type 2 :

```
Total width of frame in Y-direction      = 15.000
Distance between column center lines     = 5.000
7S1I of columns at top of frame          = 0.8533E-02
7S1I of columns at bottom of frame       = 0.8533E-02
7S1I of beams at top of frame            = 0.6834E-02
7S1I of beams at bottom of frame         = 0.6834E-02
Area of ext. cols at top of frame        = 0.160
Area of ext. cols at bottom of frame     = 0.160
Width of column                          = 0.400
Depth of beam                            = 0.450
Grade of concrete used in columns        = M20
Grade of concrete used in beams          = M20
```

Type 3 :

```
Total width of frame in Y-direction      = 4.500
Distance between column center lines     = 4.500
7S1I of columns at top of frame          = 0.4267E-02
7S1I of beams at top of frame            = 0.4556E-02
7S1I of beams at bottom of frame         = 0.4556E-02
Area of ext. cols at top of frame        = 0.160
Area of ext. cols at bottom of frame     = 0.160
Width of column                          = 0.400
Depth of beam                            = 0.450
Grade of concrete used in columns        = M20
Grade of concrete used in beams          = M20
```

Frame	Type	X location
1	1	0.000
2	2	5.000
3	3	10.000
4	2	15.000
5	1	20.000

WALLS IN Y-DIRECTION:

Number of walls = 1 : Types of wall = 1

Type 1 :

```
Moment of inertia =   5.811
Shear area        =   1.200
```

Wall	Type	X location
1	1	10.000

ECCENTRICITY CALCULATION:

```
Center of mass               : (10.000, 7.500) : given,
Center of stiffness          : (10.000, 7.211) : calculated
Eccentricity (e)             : ( 0.000, 0.289)
Design eccentricity (1.5e)   : ( 0.000, 0.434)
```

FRAME AND WALL STIFFNESSES:

Frames in X-direction

Frame	Stiffness (kN/m) (Point load at top)
A	15843.44
B	9933.17
C	9933.17
D	15843.44

Frames in Y-direction

Frame	Stiffness (kN/m) (Point load at top)
1	12154.84
2	7578.51
3	4540.91
4	7578.51
5	12154.84

Walls in Y-direction

Wall	Stiffness (Point load at top) (kN/m)	Stiffness (Actual seismic load distribution) (kN/m)
1	142010.00	210408.30

FORCE DISTRIBUTION AMONG FRAMES IN X-DIRECTION

Level	Frame A	Frame B	Frame C	Frame D
4	77.46	48.56	48.98	80.33
3	69.48	43.56	43.93	72.06
2	43.99	27.58	27.81	45.61
1	41.37	25.94	26.16	42.91

FORCE DISTRIBUTION AMONG FRAMES IN Y-DIRECTION

Level	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
4	48.48	30.23	18.11	30.23	48.48
3	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00
1	0.00	0.00	0.00	0.00	0.00

For wall-frame system, the frame force thus obtained are usually increased by 30%. These are

Level	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
4	63.03	39.30	23.55	39.30	63.03
3	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00
1	0.00	0.00	0.00	0.00	0.00

The seismic force in frames as above is less than 25% of the total design seismic force. Hence, as per code requirement, the design force for frames is to be increased to

Level	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
4	75.91	47.33	28.36	47.33	75.91
3	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00
1	0.00	0.00	0.00	0.00	0.00

FORCE DISTRIBUTION AMONG SHEAR WALLS IN Y-DIRECTION

Story	Wall 1
4	267.05
3	287.62
2	210.06
1	159.07

Example 19.3

A 16-story office building of reinforced concrete is located in New Delhi (Seismic Zone IV). The building is founded on a raft foundation. The lumped weight is 5,400 kN on the floors and 3,600 kN on the roof. The story height is 4.4 m for the first story and 3.2 m for the remaining stories. Calculate the design seismic force on the building by the seismic coefficient method according to IS:1893 (1984). Assume $I = 1.0$ and $K = 1.0$.

Solution

```
Input file name  : ex3.in
Output file name : EX3.OUT
Title of problem : EXAMPLE 19.3
Method selected  : Seismic Coefficient Method

Performance factor (K)                        = 1
Coefficient for soil foundation system   (ß)  = 1
Importance factor (I)                         = 1
Seismic zone (1,2,3,4,5)                      = 4
Number of stories (N)                         = 16
Maximum base length in X-direction (Dx)       = 30 m
Maximum base length in Y direction (Dy)       = 15 m
```

Story level	Story height (m)	Weight (kN)
16	3.2	3600.0
15	3.2	5400.0
14	3.2	5400.0
13	3.2	5400.0
12	3.2	5400.0
11	3.2	5400.0
10	3.2	5400.0
9	3.2	5400.0
8	3.2	5400.0
7	3.2	5400.0
6	3.2	5400.0
5	3.2	5400.0
4	3.2	5400.0
3	3.2	5400.0
2	3.2	5400.0
1	4.4	5400.0

	Fundamental time period (sec)	Value of C
X-direction	1.600	0.380
Y-direction	1.600	0.380

	Base shear (kN)	Base moment (kNm)
X-direction	1607.40	63913.84
Y-direction	1607.40	63913.84

Story level	Design seismic force (kN) X-direction	Design seismic force (kN) Y-direction
16	190	190
15	251	251
14	220	220
13	190	190
12	163	163
11	138	138
10	115	115
9	94	94
8	75	75
7	58	58
6	43	43
5	31	31
4	20	20
3	12	12
2	6	6
1	2	2

[4]To save space, detailed computer output has not been reproduced for this example.

19.16 EVALUATION

The Code does not yet have any regulation to control the use of the value for the fundamental period obtained by dynamic analysis, which may be too large. This enables the designer to perform a bare-frame analysis by excluding the stiffness of nonstructural members, which will result in an unrealistically large value for the fundamental period, in turn resulting in low design forces. The seismic codes of many countries now avoid such a situation by (1) specifying the use of a fundamental period obtained from an empirical formula, (2) establishing a lower limit on design seismic force based on empirical formulas for the fundamental period, or (3) establishing an upper limit on the fundamental period based on empirical formulas.

The revision of the Code IS:1893 has begun, but it will take some time to be finalized. It is expected that the next revision will incorporate one of the aforementioned provisions. Also, it appears that the Code may be revised to specify a higher elastic force, to be reduced by the response factor, to obtain the design force, as is now prevalent in many other codes.

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