**Ninth Canadian Conference on
Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007**

ALTERNATE LATERAL LOAD PROFILE FOR ASEISMIC DESIGN OF OPEN GROUND STOREY BUILDINGS

R. Davis[1] D. Menon[2] and A. M. Prasad[2]

ABSTRACT

The lateral load distribution for equivalent static load analysis suggested by design codes is applicable only for the vertically regular building. This is generated from the shape of the fundamental mode of the building, and typically lies between parabolic and linear variation with height. But in the case of an open ground storey building, the pattern of the mode shape is different, when the stiffness of the infill walls is included. Due to the low lateral stiffness of the ground storey compared to the upper storeys, a relatively large deflection occurs in the first floor level, which affects the mode shape. An alternate realistic lateral load profile for equivalent static analysis for open ground storey buildings (OGS) and the consequent effects on the local bending moments in the columns of the ground storey are addressed in this paper.

Introduction

Seismic design codes recommend the use of equivalent lateral force (ELF) analysis for regular buildings. The load profile suggested for the ELF analysis is based on the fundamental mode shape of the building. Generally the fundamental mode shape of a regular building varies between parabolic and linear profiles along the height, depending on the stiffness distribution of various stories.

The assumptions made in the ELF procedure are applicable for regular structures, which do not have any discontinuities in mass, stiffness, and strength over the height (Valmundsson and Nau 1997). The open ground storey building does not fall in this category on account of its vertical irregularity.

In the case of irregular buildings, the code recommends response spectrum analysis, where the effects of irregularities are inherently taken care of and multiple modes are considered. This will yield accurate results provided the irregularity is modelled properly. For example, in the case of an open ground storey type of building, the irregularity can be captured only by modelling the stiffness of the infill walls. For this kind of building, the fundamental mode shape will be different from that of a regular building due to the upper stories being much stiffer, compared to the ground storey. Response spectrum analysis, by modelling the infill stiffness, can bring out accurate results, which are significantly different compared to that with the bare frame model.

In the normal design practice, infill walls are treated as non-structural elements, and hence the buildings are modelled as bare framed structures. This approach however ignores the significant stiffness

[1] Graduate Student, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai – 600 036
[2] Professor, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai – 600 036, India

contribution of the infill walls. This has several implications. Firstly, increased lateral stiffness of the building implies decrease in time period, resulting in increase in base shear. Secondly, absence of infill in the ground storey affects the mode shapes and imposes higher drift at first floor level which is not captured in bare frame analysis. Thirdly, this introduces higher displacement demand in the ground floor columns, which may lead to an undesirable storey sway mechanism.

Design codes attempt to account for this by imposing enhanced design forces in ground floor columns, which are analysed as part of bare frame. A magnification factor of 2.5 is prescribed in IS 1893. A factor of $\left(1+\frac{\Delta V_{RW}}{\sum V_{Ed}}\right)$ for the discontinuity in storey stiffness (ΔV_{RW} denotes the deficit in infill resistance in the ground storey relative to first storey, ΣV_{Ed} the total design shear force at the first floor level) and a factor λ for the shift in period of infilled building are introduced in Euro code for the computation of design moments and shears in such columns. A more rational design procedure is to incorporate the increase in base shear through proper modelling of infill stiffness and to distribute the base shear in the ELF analysis by means of an appropriate height-wise distribution of lateral loads. The present paper proposes a suitable lateral load distribution. In the case of open ground storey frames, the participation of fundamental mode is highly predominant and this is used in modelling the lateral load profile.

Distribution of Lateral Loads Specified in Design Codes

It can be proved using the principles of dynamics that the lateral load (f_{jn}) acting at any level j from base of the building in a particular mode (n) is dependent on the mode shape and can be written as follows:

$$f_{jn} = \frac{m_j \varphi_{jn}}{\sum_{j=1}^{n} m_j \varphi_{jn}} V_{bn} \tag{1}$$

Where j is any floor level from base of the building, n is the mode number, φ_{jn} is the ordinate of the n^{th} mode shape vector at level j, V_{bn} is the base shear corresponding to n^{th} mode, m_j is mass at j^{th} level. Assuming the higher mode contributions are negligible the load distribution for the equivalent static analysis can be taken as the profile of the fundamental mode.

The codal lateral force distribution represents the forces obtained from the predominant mode of vibration (Elnashai 2001). The uniform lateral load distribution has been used for the pushover analysis of soft ground storey building as it has the capability to expose the possible soft storey collapse. Another lateral load distribution often used is assumed to be proportional to the masses at floor levels.

Design codes follow different distributions of lateral loads for regular building. Table 1 shows a summary of expressions of lateral load distributions used by various codes. The Indian standard (IS1893) recommends parabolic distribution along the height. International Building Code (IBC) suggests linear shape (k = 1) when the fundamental time period (T) of the building is less than 0.5s, parabolic shape (k = 2) when T greater than 2.5s, intermediate values for k for intermediate values of time periods. Canadian Code (NBCC) gives a load profile which is a linear function of height. Mexico Federal District Code (MFDC) follows a linear profile when the fundamental time period is less than a critical time period and a combination of linear and parabolic terms when the time period greater than the critical time period. Euro code proposes the load distribution based on the fundamental mode shape. It also allows the designer to use distribution linearly varying with height.

The expression given by IBC for time period less than 0.5s and that given by NBCC for time period less than 0.7s gives linear relations. The expression given by the Mexican code comprises a summation of linear and parabolic terms. The parabolic term is to account for the effects of higher modes in the response.

It is to be noted that Euro code follows the theoretical approach based on the mode shape for the lateral load. This will yield accurate results provided the modelling takes in to account all irregularities in the real structure.

Table 1. Comparison of load profiles in different codes.

Code	Expression
IS 1893	$f_j = V_B \frac{w_j h_j^2}{\sum_{j=1}^{n} w_j h_j^2}$
IBC	$f_j = V_B \frac{w_j h_j^k}{\sum_{j=1}^{n} w_j h_j^k}, k = \begin{cases} 1 & \text{if } T < 0.5s \\ (T+1.5)/2 & \text{if } 0.5s \le T \le 2.5s \\ 2 & \text{if } T > 2.5s \end{cases}$
NBCC	$f_j = (V_B - F_t) \frac{w_j h_j}{\sum_{j=1}^{n} w_j h_j}, F_t = \begin{cases} 0 & \text{if } T < 0.7s \\ 0.07TV_B & \text{if } 0.7s < T \le 3.6s \\ 0.25V_B & \text{if } T > 3.6s \end{cases}$
Mexico Federal District	$f_j = V_B \frac{w_j h_j}{\sum_{j=1}^{n} w_j h_j} \text{ for } T < T_c \quad f_j = V_B^{(1)} \frac{w_j h_j}{\sum_{j=1}^{n} w_j h_j} + V_B^{(2)} \frac{w_j h_j^2}{\sum_{j=1}^{n} w_j h_j^2} \text{ for } T > T_c$
Euro Code	$f_j = V_B \frac{w_j \phi_j}{\sum_{j=1}^{n} w_j \phi_j}$

Measures of Lateral Stiffness of Storey

The lateral stiffness of the storey is an important parameter that determines the particular storey is soft or not. The stiffness of the storey increases when the infill is present. It can be defined for bare frame and infilled frame.

Storey Stiffness of a Bare Frame (K_{SB})

The lateral storey stiffness of the typical bare frame can be estimated using closed form expression (Shultz 1992). Eq. 2 represents the typical storey stiffness of a bare frame as a function of storey height (h), sum of stiffness of columns (ΣK_c), sum of stiffness of beams above the storey (ΣK_{ga}) and sum of stiffness of beams below the storey (ΣK_{gb}) is given below. Studies indicate that this equation is reasonably good to estimate the lateral stiffness of the storey considering the rotation of the beam also.

$$K_{SB} = \left(\frac{24}{h^2}\right)\left(\frac{1}{\frac{2}{\Sigma K_c} + \frac{1}{\Sigma K_{ga}} + \frac{1}{\Sigma K_{gb}}}\right) \qquad (2)$$

The stiffness of the ground storey (K_0) is a special case that can be developed from Eq. 2 by substituting K_{gb} as zero. The columns at the ground storey are assumed to be fixed at base.

Storey Stiffness of Infilled Frame (K_{SI})

The typical storey stiffness for an infilled frame (K_{SI}) is assumed as the sum of the stiffness due to bare frame (K_{SB}) and the infill walls (K_{IW}). The presence of infill frame in any storey increases the stiffness considerably until the failure of infill in the case of lateral load. It is to be noted that in this present study only the elastic behaviour of the infill is considered. Studies show that the equivalent strut model (Smith and Carter 1969) can be used to model the infill walls in the elastic analysis. The infill behaves as an equivalent strut diagonally connected between the frames and is active only in compression. The stiffness of the equivalent struts in a particular storey can be calculated using the following expression:

$$K_{IW} = \Sigma\left(\frac{AE}{L}\right)_{ES} \cos^2\theta \tag{3}$$

Where A, E, L are area, modulus of elasticity and length of equivalent strut (ES). θ is the angle made by the strut with horizontal direction as shown in Fig. 1.

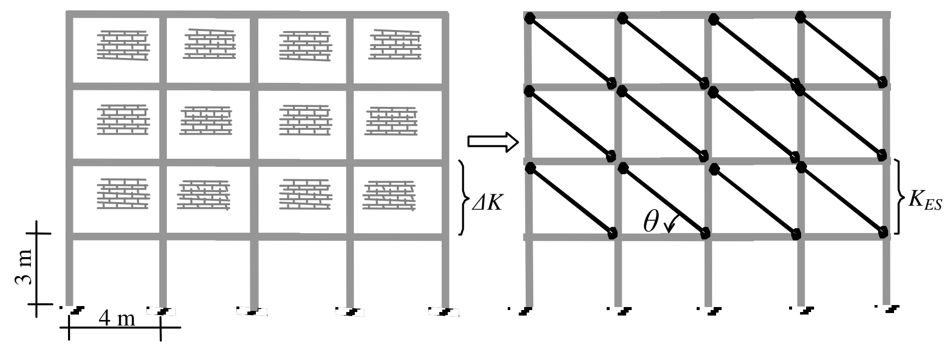


Figure 1. Infilled frame idealized as equivalent strut.

Measure of Softness of Ground Storey (K_r)

An open ground storey frame is considered which has infill walls in all storeys except the ground storey. Figure 1 shows an ideal open ground storey frame considered. K_r is defined as the ratio of additional stiffness due to infill walls alone in the storey above to the stiffness of ground storey. It provides a measure of the degree of softness of the storey. K_r takes a lower bound value of zero for the bare frame and in practical case it can take a value up to about 20. It is expressed as follows:

$$K_r = \frac{\Delta K}{K_0} \tag{4}$$

ΔK is the additional storey stiffness of the first storey due to infill walls only.

Code Criteria for Soft-storey

Criteria for soft storey given in many codes use the quantity of lateral stiffness of storeys. Lateral stiffness of any storey less than 70% of the first storey implies that the particular storey is a soft-storey. Another criterion for soft-storey is when the stiffness of any storey is less than average stiffness of three consecutive storeys. There is no clear guideline given in the codes to estimate the lateral stiffness of the storey manually (Scarlet 1997). The authors understand that the Eq. 2 can be applied to estimate the lateral storey stiffness approximately for bare frame only.

As an example the lateral storey stiffness of the frame shown in the Figure 1 is calculated. The sizes of the beams are 230 x 400 mm and that of columns are 230 x 230 mm. The concrete used is of the grade M20. The infill walls are of 230mm thick brick. In this particular case the value of K_r is found to be around 12 which implies that ground storey is 1200% softer than the first storey.

Variation of Mode Shape with K_r

The mode shape of an OGS building does not follow the same profile always. The profile varies with the degree of softness of the ground storey. As the value of K_r depends on the degree of softness the variation of mode shape with the value of K_r can be plotted. When the value of K_r increases, the profile tends to become almost uniform from the first floor onwards. The OGS frame shown in the Figure 1 is considered and modelled in SAP 2000. The infill walls are modelled as equivalent strut. The K_r value is varied from zero to an upper value of three. The masses are lumped at the floor levels. Eigen value analysis of the frame is done for all the cases to get the mode shapes. The variation of mode shape is plotted with various values of K_r as shown in Fig. 2. As the value of K_r increases the storeys above ground storey become stiffer and move as a rigid body. Since the ground storey is relatively flexible compared to other storeys, the displacement at first floor level becomes larger. The inter-storey drift at the first floor level is turned out to be larger as compared to that of other storeys.

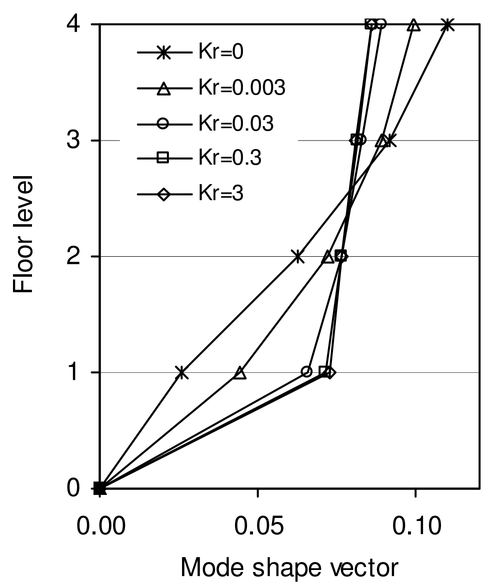
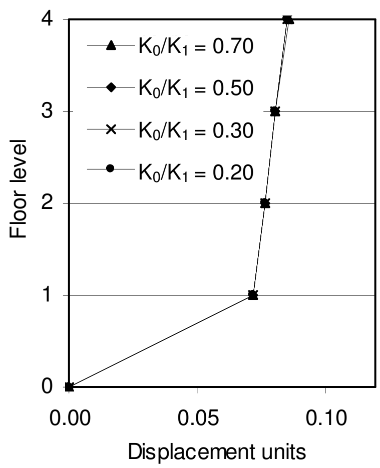


Figure 2. Variation of mode shape with K_r

Generally the existence of soft storey is checked as per the codes by using the ratio of stiffness of adjacent storeys (K_i/K_{i+1}). K_0 is stiffness of ground storey and K_1 is the stiffness of storey above (K_1 includes infill stiffness also). The variation of the mode shape can also be plotted with the ratio K_0/K_1. As the stiffness of the ground storey decreases then the form of the mode shape also alters.

The value of K_0 (ground-storey) is less than 70% of K_1 (first-storey) implies the particular storey is a soft-storey. Consider the situation where the ratio K_0/K_1 is less than 0.7. Fig. 3a shows the mode shape variation in this particular case. It can be seen that mode shape merges to a shape joining two lines, one with a steep slope up to first floor from origin and another line with a mild slope up to top storey as shown in Fig. 3b. This shape can be taken as the most likely lateral load profile for the open ground storey building. The resulting load profile can be imagined as combination of rectangular and triangular areas. The width of the rectangular area is assumed as (1-α) and base of adjoining triangle as α.

The profile of the mode shape changes with the number of storeys and bays. As the number of bays increases the building becomes stiffer and slope of the profile increases as shown in Fig. 4a.



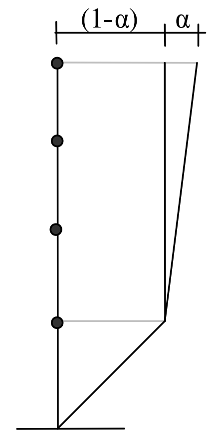
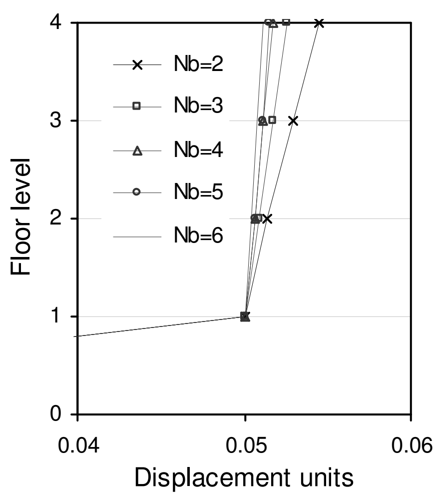


Figure 3. (a) Variation of mode shape with K_r (b) Variation of mode shape with (K_0 / K_1).

The profile of the mode shape changes with the number of storeys also. As the number of storeys of a five bay framed structure increases from two to eight, the slope of the mode shape profile also increases. The profile of the mode shapes with different number of storeys is plotted in Fig. 4b. It can be inferred that the load profile follows a pattern and is not purely uniform across the height of the building but varies with number of bays and storeys.



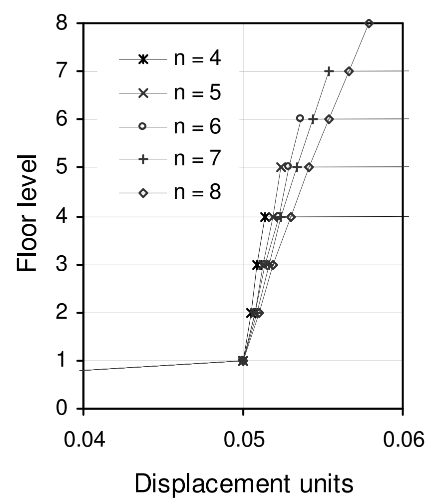


Figure 4. (a) Variation of mode shape with K_r (b) Variation of mode shape with (K_0 / K_1).

Load Profile for Open Ground Storey Building

For uniform frames where the stiffness of the infill is modelled properly, a load distribution can be proposed for ELF analysis based on the above presumptions. The proposed distribution is of the form given below:

$$f_j = \left((1-\alpha)\frac{w_j}{\sum_{j=1}^{n} w_j} + \alpha \frac{w_j h_j}{\sum_{j=1}^{n} w_j h_j} \right) V_B \tag{5}$$

Where w_j is the seismic weight at floor level j (j greater than zero), V_B is the design base shear and n is the number of storeys. α is the factor defined earlier which is a measure of the slope of load profile which can be taken as 0.15 for results on conservative side. The proposed load distribution is applicable for K_0/K_1 less than 0.70.

Comparison of Bending Moment

The proposed Eq. 5 would give realistic resultant forces in the ground storey columns. For comparison of the bending moments, the proposed load distribution is applied to a four storied four bay regular building shown in Fig. 1, with the infill modelled. The code prescribed base shear and lateral loads in both bare frame and infilled frame is given in the Table 3. The bending moments in the ground storey columns of the infilled frame analysis is compared with that from the conventional bare frame analysis. For the conventional bare frame analysis, load profile suggested by IS 1893 is used.

Table 3. Design base shear and lateral load distribution in bare frame and infilled frame.

Load	Bare frame (kN)	OGS (kN)
V_B	1396	1678
Q_1	41	395
Q_2	215	491
Q_3	484	462
Q_4	656	330

The increase in the base shear in the two frames is due to the shift in elastic time periods of the bare frame and the OGS frame. This can be calculated manually by estimating the time periods and this aspect is discussed in the following section. The comparison of bending moments in the ground storey columns (corner, exterior and interior columns) from bare frame and OGS frame analysis at top and bottom points are shown in the Figs.5a and 5b respectively.

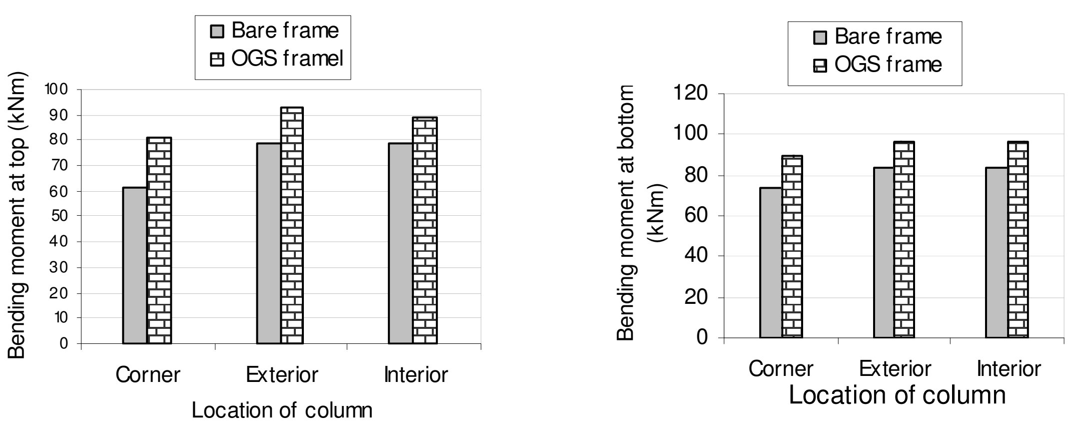


Figure 5. (a) Comparison of BM at top. (b) Comparison of BM at bottom.

The bending moments at top (Fig. 5a) and bottom (Fig. 5b) points of the ground storey columns in the OGS frame are found to exceed that from conventional bare frame analysis. The ratio of bending moments in ground floor columns from OGS frame analysis to that from bare frame analysis at the corresponding points is defined as magnification factor. The magnification factors for typical columns, at corner, exterior and interior points, are plotted in Fig. 6. In this particular case the magnification factors are found to be less than 2.5 suggested by the IS code.

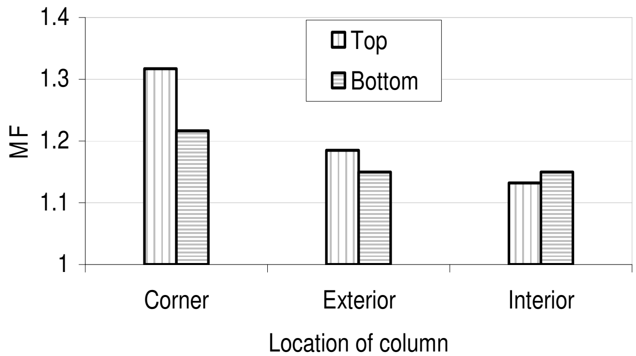


Figure 6. Magnification factors for typical ground storey columns at top and bottom points.

The maximum moment in the ground storey column can also be determined by ultimate load analysis assuming the ultimate moment to be M_c at all columns of ground storey. The support condition at the base of the columns is assumed to be fixed. The total design base shear is assumed to act at first floor level. The following approximate expression gives the maximum moment M_c for the design base shear V_B:

$$M_c = \frac{V_B h}{2.n_c} \tag{5}$$

n_c is the total number of columns. M_c is an estimation of lower bound of the bending moment to which the ground storey columns are to be designed. In the above example the M_c is nearly 100 kNm. The bending moments at top points of some of the columns in the ground floor are less than M_c which are to be magnified to 100 kN. In general for any open ground storey building M_c value can be estimated manually and the ground floor columns can be designed for this minimum moment instead of providing the magnification factor for all the columns.

Ratio of Base Shear in the Infilled Framed Building to that of Bare Framed Building (λ)

The magnification of bending moment and shear forces is due to two factors. One factor is the increases in design base shear due to shift in time period and other due to the discontinuity in storey stiffness. The ratio of design base shear in the infilled framed building to that of bare framed building due to shift in time period is defined as λ. The base shear attracted by the building is directly proportional to pseudo spectral acceleration (S_a). The time periods of OGS building (T_{OGS}), bare framed (T_B) building, along with spectral acceleration for OGS building $(S_a/g)_{OGS}$ and bare framed building $(S_a/g)_B$ are marked in the response spectrum for a typical case. The response spectrum suggested by code (for eg. IS1893-2002) is divided in to three regions. Region 1, (time period T varying from 0 to 0.1s in IS 1893) which is acceleration sensitive, Region 2 (0.1s to 0.55s for medium soil in IS 1893) which is the velocity sensitive region (S_a is proportional to C_a - Region 2), Region 3 (0.55s to 4s for medium soil in IS 1893) is the displacement sensitive region (Sa is proportional to C_v/T - Region 3) and is shown in Fig. 7. If both time periods of OGS and bare framed building are in the velocity sensitive region, (T_B and T_{OGS} less than the time period (T_c) at the boundary of transition from region 2 to region 3) then λ will be unity, where T_c is given by :

$$T_c = \frac{C_v}{C_a} \tag{6}$$

If the both the time periods T_B and T_{OGS} are in the region 3, spectral acceleration values are inversely proportional to the time period and λ can be expressed as the ratio of time period of bare frame to that of infilled frame. It can be expressed as shown in the Table 4.

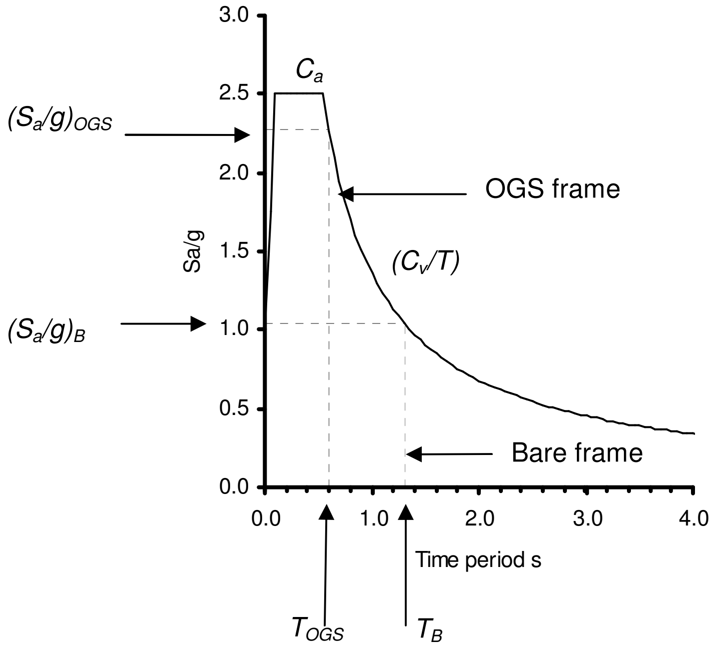


Figure 7. Response spectrum for medium soil (IS 1893).

Table 4. Estimation of λ for various values of T_B and T_{OGS}.

T_B	$T_{(OGS)}$	λ
Less than T_c	Less than T_c	1
Greater than T_c	Greater than T_c	$\frac{T_B}{T_{OGS}}$
Greater than T_c	Less than T_c	$\frac{C_a T_B}{C_v}$

The estimation of λ will give the amount of magnification due to the shift in elastic time periods. The time periods of the OGS and the bare framed building can be estimated using the modified Rayleigh formula:

$$T = 2\sqrt{\Delta} \tag{7}$$

Where Δ is the displacement in metre at top of the building due to gravity loads applied horizontally. Displacement can be evaluated at every storey level using the storey stiffness expressions. If both time periods are in region 2 then λ becomes unity. λ estimated using the computational time periods give a value 1.56 and that estimated using the closed from equation for storey stiffness yield a value of 1.14. The difference is due to the error in estimation of stiffness. For conservative results the upper bound of the stiffness in the case of OGS buildings and lower bound of stiffness in the bare frame building is preferable.

Conclusions

The proposed expression for the lateral load can be considered as an alternate load distribution for ELF analysis. This is based on the fundamental mode shape of typical open ground storey building. The proposed equation is appropriate in doing equivalent static analysis of open ground storey buildings with stiffness modelled to produce a more realistic effect on the ground storey columns. The designer can use this load distribution while incorporating the stiffness of the infill walls. The magnification factor normally used for soft storey type of buildings can be avoided as the proposed load distribution is capable of generating realistic resultant forces. A simple check of the lower bound of the bending moments in the ground storey columns can also be estimated by applying the total design base shear at the first floor level. The increase in the design base shear in the OGS building due to the shift in fundamental time period has to be incorporated separately. This could be done by estimating the lateral stiffness.

References

Elnashai, A. S., 2001. Static pushover versus dynamic collapse analysis of RC buildings, *Engineering Structures*, 23, 407-424.

IS 1893: Part 1, 2002. Criteria for earthquake resistant design of structures, Bureau of Indian Standards, New Delhi.

IS 456, 2000. Indian Standard for Plain and Reinforced Concrete - Code of Practice, Bureau of Indian Standards, New Delhi.

Scarlat, A., 1997. Design of soft stories - a simplified energy approach, *Earthquake Spectra*, 13(2), 305-315.

Shultz, A. E., 1992. Approximating lateral stiffness of storeys in elastic frames, *Journal of Structural Engineering*, 118(1), 243-263.

Smith, S. B., and Carter C., 1969. A method of analysis for infilled frames, *Proceedings of Institution of Civil Engineers*, Part 2, 44, 31-48.

Smith, S. B., 1962. Lateral stiffness of infilled frames, *Journal of Structural Engineering*, 88, 183-199.

Valmundsson, E. V. and Nau, J. M., 1997. Seismic response of building frames with vertical structural irregularities", *ASCE Journal of Structural Engineering*, 123(1), 30-41.