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Displacement-based seismic design of open ground storey buildings

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Abstract. Open ground storey (OGS) buildings are characterized by the sudden reduction of stiffness in the ground storey with respect to the upper infilled storeys. During earthquakes, this vertical irregularity may result in accumulated damage in the ground storey members of OGS buildings without much damage in the upper storeys. Hence, the structural design of OGS buildings needs special attention. The present study suggests a modification of existing displacement-based design (DBD) procedure by proposing a new lateral load distribution. The increased demands of ground storey members of OGS buildings are estimated based on non-linear time history analysis results of four sets of bare and OGS frames having four to ten storey heights. The relationship between the increased demand and the relative stiffness of ground storey (with respect to upper storeys) is taken as the criterion for developing the expression for the design lateral load. It is also observed that under far-field earthquakes, there is a decrease in the ground storey drift of OGS frames as the height of the frame increases, whereas there is no such reduction when these frames are subjected to near-field earthquakes.

Keywords: open ground storey buildings; pilotis; displacement-based design; stiffness irregularity; time history analysis; storey drift; base shear

1. Introduction

Buildings which are constructed without masonry infills in the ground storey are generally called open ground storey (OGS) buildings (or buildings with pilotis). They provide large parking space, which is a major requirement for buildings in urban areas. Several OGS buildings collapsed during recent earthquakes and the failure can be attributed to the sudden reduction in stiffness compared to the upper infilled storeys. A storey can be considered as soft, if its lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above (IS 1893 (1) 2002). If the storey lateral strength is less than 80 percent of the storey above, it is considered as a weak storey. As per IS 1893(1) (2002), for open ground storey buildings, the columns and beams of the soft storey are to be designed for 2.5 times

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the storey shears and moments calculated under seismic loads, neglecting infill walls in other storeys. But, as per Fardis *et al*. (1999), if the beams of the open-storey are also strengthened, it decreases the beam elastic and inelastic flexural demands and increases the tendency of concentration of damage to the ground storey columns. Moreover, no special measures are needed for the beams of a less infilled storey; as such beams are protected from the development of large rotations at their ends by the in-plane stiffness of the heavier-infilled adjacent storey.

Davis (2009) had proposed modifications for the procedures like equivalent static analysis (ESA), response spectrum analysis (RSA) and non-linear dynamic analysis (NDA) such that they can be used for the design of OGS buildings. The multiplication factor for base shear for use in ESA approach is given by

$$MF = 0.656 + 0.044 n_s^{0.979} \beta^{-0.279} \geq 1.0 \tag{1}$$

where n_s is the number of storeys and β is the storey stiffness index which is given by

$$\beta = \frac{\eta}{\eta_B} \tag{2}$$

The stiffness ratio parameter for OGS frames (η) and bare frames (η_B) can be calculated as k_o/k_{av} of the respective frames where k_o is the stiffness of ground storey and k_{av} is the average stiffness of upper storeys. The multiplication factor varies roughly from 1.0 to 1.45 for 4- to 10-storeyed buildings.

Davis (2011) compared the performances of four-storeyed OGS framed buildings, designed as per Indian seismic code for three cases, viz., (i) for gravity load alone, (ii) for gravity load combined with seismic load without magnification factor as suggested in IS 1893 (2002) and (iii) considering magnification factor of 2.5. Pushover analyses of designed buildings were carried out including effects of infill walls. The relative performance of three cases is evaluated as per the Capacity spectrum method and the building designed with the magnification factor 2.5 was found to achieve the performance criteria, compared to the other two buildings.

Surendran and Kaushik (2012) performed a comparative study on the in-plane lateral load behaviour of masonry infilled RC frames with and without openings as specified in seismic codes of different countries. Selection of suitable analytical models and the estimation of strength, stiffness, failure modes, and other properties of infill RC frames with openings were thoroughly discussed. A parametric study on the response of infilled RC framed buildings under lateral loading was carried out by Mahmud *et al*. (2010).The effect of number of bays and the number of storeys was investigated. They concluded that the infill modelling is essential in predicting the realistic behaviour especially in the case of soft storey buildings.

Asteris *et al*. (2011) discussed the advantages and disadvantages of various macro-models for infilled frames and recommended different models for practical purposes. Their studies had shown that the numerical simulation of infilled frames is difficult and generally unreliable because of the uncertainties associated with the various modelling parameters.

Mondal and Jain (2008) had proposed reduction factor for the effective width of diagonal strut over that of the solid infilled frame to calculate its initial stiffness when a central window opening is present. Their study was based on initial lateral stiffness corresponding to 10% of the lateral strength of the infilled frames. The presence of central opening can be considered by reducing the effective width through a reduction factor, ρ_w=1−2.6α_{co}, where α_{co} is the ratio of the area of opening to the area of the infill.

The contribution of masonry infill in enhancing the strength and stiffness of frames is not generally considered in structural design. Seismic codes contain provisions for calculating the stiffness of infill by modelling it as a 'diagonal strut'. However, there is no recommendation for incorporating the effect of openings in infill panels. Asteris (2003), Asteris *et al*. (2012) present the details of numerical investigations conducted on infilled frames with openings. A reduction factor is proposed for finding out the effect of opening and is given as

$$\lambda = 1 - 2\alpha_w^{0.54} + \alpha_w^{1.14} \tag{3}$$

where λ is the infill panel stiffness reduction factor and α_w is the infill wall opening percentage. They concluded that column shear decreases due to the presence of infill walls; however in the case of infilled frames with a soft ground storey, there is considerable increase in the column shear.

A double-strut nonlinear cyclic model for unreinforced masonry panels was implemented within a fibre-based Finite Element program by Smyrou *et al*. (2011). The adequacy of the model in predicting the cyclic/seismic response of multi-storey infilled reinforced concrete frames was verified through comparisons against experimental results. They suggested that a model that would account for all types of masonry panel failures would be unpractical, due to the appreciable level of complexity and uncertainty involved.

The traditional force-based design (FBD) recommended in most of the seismic design codes, estimates the design base shear based on the spectral acceleration corresponding to the fundamental period of the building and the building seismic mass, appropriately reduced to allow inelastic response under design-basis earthquake. It is assumed that the designed structure would have the intended ductility capacity and hence, this results in designing structures with unknown reliability. However, displacement-based design (DBD) methods focus on designing the structure to have a target displacement capacity, corresponding to a critical drift and can be performed for several limit states, thus ensuring predictable response under various seismic hazard intensities.

Several DBD methods are developed recently, but their application is basically intended for regular buildings. Priestley and Kowalsky (2000) developed direct displacement based design (DDBD) which can be used for regular frames, structural wall buildings and dual systems. The substitute structure approach is used to get the properties of equivalent structure and the elastic displacement spectra for equivalent damping are used for specifying the demand.

In partially infilled RC buildings with the ground storey open, inelastic demand tends to concentrate in the columns of the open storey. A capacity design based approach was proposed by Fardis *et al*. (1997) aiming at shifting plastic hinge formation from the open-storey columns to the beams and columns of the adjacent infilled storey.

One of the DBD methods which can be extended to the design of OGS buildings is the Deformation-controlled design (Panagiotakos and Fardis 1999). It aims to integrate a DBD approach within the entire structural design process, including the effects of gravity loads. First of all, longitudinal reinforcement in plastic hinge regions is proportioned for non-seismic loads and service level seismic loads. Using the results from the design of plastic hinge, the flexural design of columns above the first storey is conducted. The design moments for the capacity protected members follows from the moments of the plastic hinge regions. All members are designed for shear in accordance with capacity design principles. Peak inelastic member deformations for life safety level earthquake are estimated and transverse reinforcement is proportioned to resist inelastic deformation demand. Change in longitudinal reinforcement, if required, is also made to limit the rotations.

The present study attempts to derive suitable load distribution pattern for OGS buildings such that DBD (Priestley and Kowalsky 2000) can be applied to such buildings.

2. Details of the present work

4, 6, 8 and 10-storeyed frames having 5 equal bays of 3.0m span each and storey height equal to 3.2m, are selected. They are designed for a PGA of 0.6*g* assuming that that building is located in hard soil. Three sets of frames are identified and are named as (i) bare frames (without infill, which is commonly used for modelling fully infilled frames), (ii) fully infilled (FI) frames (infills in all the storeys) and (iii) OGS frames (infills in all the storeys except in the ground storey). The weight of infills is considered in the design of all the frames, but the stiffness and damping contributions of the infills are used only in the second and third sets.

Frames are designed considering them as bare frames and hence, the cross-sectional details of beams and columns are the same for the three groups. To see the effect of multiplication factor on the performance of OGS frames, another set of frames are designed with ground storey columns having a strength equal to 1.25 times that obtained from the bare frame analysis (Davis 2009). OGS frames with increased ground storey column strength are designated as OGS (MF). Thus, non-linear time history analyses are carried out on four sets of frames with four models in each set.

3. Non-linear material modelling

A realistic material modelling is highly essential for an accurate non-linear analysis. There are several ways for incorporating non-linearity viz., using lumped plasticity models, distributed plasticity models or by fiber elements. Though fiber element modelling results in accurate outputs, modelling the entire structure using fiber elements is time consuming and hence for the present study, only the designated yielding members (beams) are modelled using fiber elements and the columns are modelled using lumped plasticity P-M2-M3 hinges. Concrete is modelled using nonlinear 'constant confinement' model, following the constitutive relation proposed by Mander *et al*. (1988) and modified by Panagiotakos and Fardis (2001). Reinforcing steel is modelled as per IS 456 (2000) stress-strain relation. Tangent stiffness-proportional Rayleigh damping is adopted with 5% damping specified for 0.2 and 0.9 times the fundamental period.

The masonry infills are modelled as equivalent compression struts. The diagonal strut model consists of two struts. The strength of the equivalent strut is calculated considering corner crushing at the compression corners, shear cracking failure along the bedding joints of the brickwork and diagonal compression failure of the slender infill wall. For doing non-linear analysis, in addition to strut width, elastic modulus and strength, the axial load versus deformation curve is also required. Asokan (2006) recommends a simplified piece-wise linear hinge property after studying the effects of parameters such as wall panel dimensions, grade of concrete and yield moments of adjacent beams and columns, size of adjoining columns, wall thickness, compressive strength and shear strength of masonry, coefficient of friction between brick and mortar and inter-face friction between frame and infill wall. For the present work, the modulus of elasticity of masonry and the strain in the strut corresponding to yield and ultimate are taken as 3000MPa, 0.0025 and 0.004 respectively. The analyses are carried out in PERFORM-3D.

The hysteresis loop for the masonry strut is taken as the same as that for the inelastic concrete

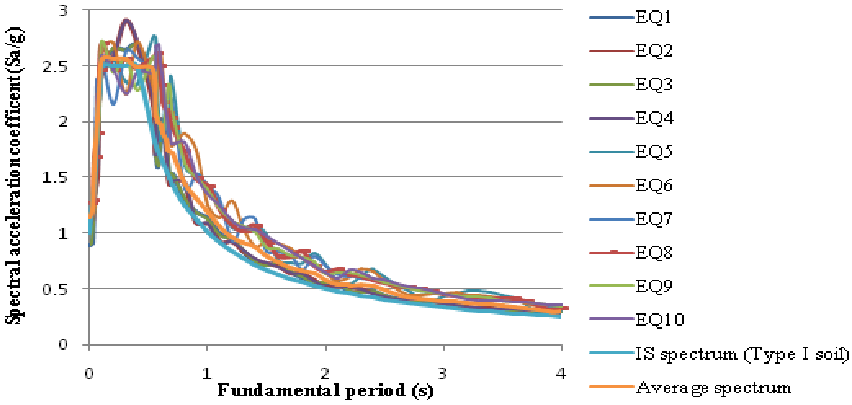


Fig. 1 Response spectra of selected ground motions

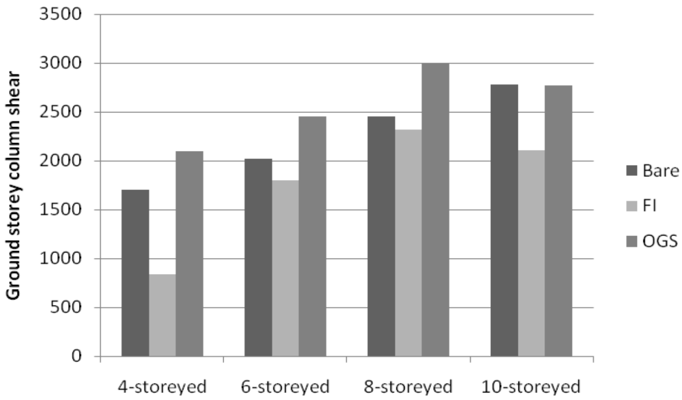


Fig. 2 Increase in ground storey shear for OGS frames (mean spectral values)

material energy degradation factors of 1.0, 0.9, 0.7, 0.4 and 0.3 are adopted at *Y*, *U*, *L*, *R* and *X* points where *Y* corresponds to yield point, *U*- ultimate point, *L* loss of strength, *R* point of minimum strength and *X* corresponds to the point of maximum deformation so as to get the degraded loop (Powell 2007). Inelastic panel, diagonal strut model is used for modelling infill walls. The hysteresis loop for a strut is the same as for the inelastic concrete material

4. Ground motion input

Ten ground motions are selected based on the criteria given in FEMA P695 (2009) from the Strong motion database available in the website of Centre for Engineering Strong Motion Data, USA (http://www.strongmotioncenter.org/). The accelerograms are made compatible with IS 1893:2002 design spectrum for Type I soil and normalised to a PGA of 0.6 g. The response spectra of the selected ground motions along with the IS spectrum is shown in Fig. 1. All the frames were analysed using these spectrum-compatible ground motions and the average responses are recorded.

Table 1 Proposed MF for OGS frames considering mean spectral values

Frame	Mean (μ) of $V_{b,\ OGS} / V_{b,\ bare}$	Standard deviation (σ)	μ+1.0σ
4-storeyed	1.264	0.062	1.326
6-storeyed	1.218	0.065	1.284
8-storeyed	1.229	0.059	1.289
10-storeyed	0.995	0.118	1.114

5. Ground storey shear from time history analyses

Maximum storey shear resisted by the columns in FI and OGS frames are evaluated for the ten ground motions considered and their mean value is compared with the corresponding value of bare frames (Fig. 2). For FI frames, the shear force in ground storey columns are comparatively less as a major portion of the shear is resisted by the infill present in the ground storey. But for OGS frames, there is an increase in the ground storey column shear compared to bare frame analysis results due to stiffness irregularity and hence, the design load distribution should consider this magnification in ground storey shear.

The ratio of ground storey column shear for OGS frame to that of fully infilled frame is calculated as 2.5, 1.4, 1.3 and 1.3 for 4, 6, 8 and 10-storeyed frames respectively. But, the ratio of ground storey column shear for OGS to bare frame is only 1.26, 1.21, 1.23 and 1.00 for the above frames. The mean (μ) and standard deviation (σ) of the ratio of ground storey shear of OGS to bare frames, for all the 10 analyses are calculated and are shown in Table 1.

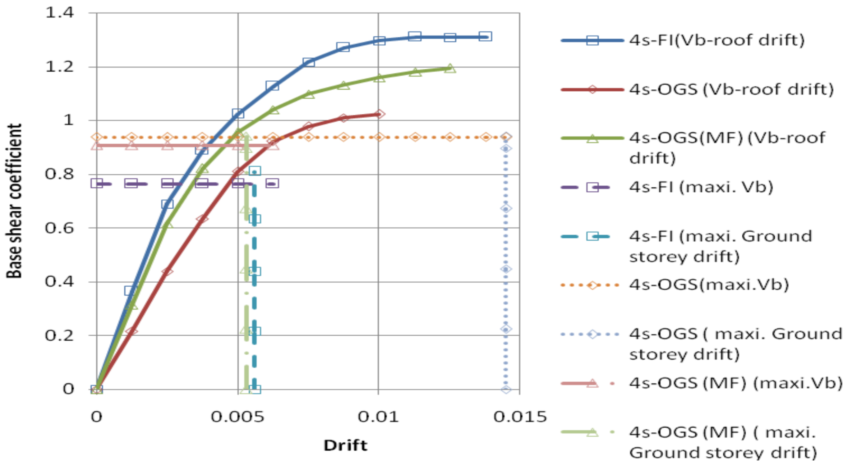
To get an upper bound of this quantity, (μ+1.0σ) of the ratio is considered for the design. Accordingly, an expression for load distribution pattern for OGS frames is proposed.

6. Base shear capacity based on pushover analysis

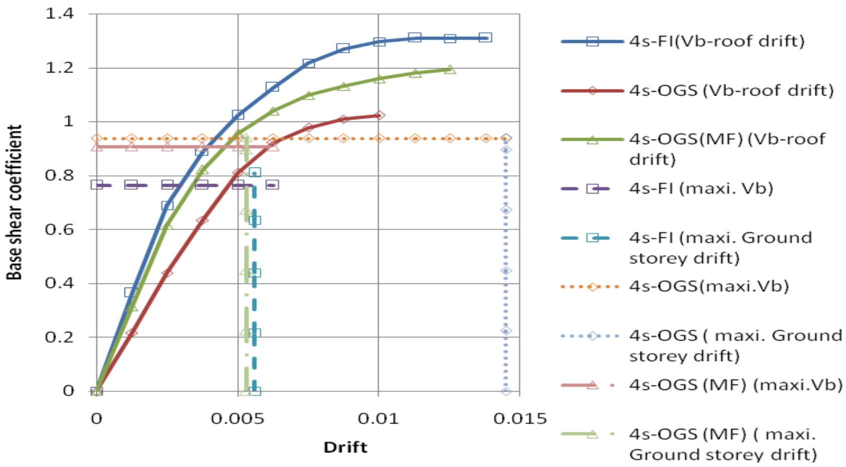
To understand the extent to which the frames are loaded during seismic excitation, the maximum base shear capacity of all the frames is evaluated using pushover analysis. Displacement controlled lateral loads, with distribution as per IS 1893:2002, are applied to the frames and the base shear versus roof displacement graphs are drawn (Fig. 3) for all frames. Fully infilled frames are considered as the benchmark frames.

Fig. 3 shows the pushover curves for 4-, 6-, 8- and 10-storeyed frames. The mean of maximum base shear and the mean of maximum ground storey drift obtained from the ten non-linear time history analyses (NLTHA) are also marked in the figure to see the fraction of the base shear capacity utilized during design-basis earthquakes. Even though the base shear demand and ground storey drift is more for OGS frames, an increase in ground storey column size (OGS (MF)) reduced the drift for short frames. It seems that tall OGS frames are not as vulnerable as short frames, as the drift in ground storey is comparatively less for tall frames. It is interesting to note that as the height of the frames increases, the difference between the graphs reduces and for 8- and 10- storeyed frames, the graphs merges for OGS and FI frames. Since there is not any difference between OGS and FI of 8 and 10-storeys, no modification is required for OGS frames of 8 and 10-storeys; hence, OGS (MF) is not shown in Fig. 3(c) and (d).

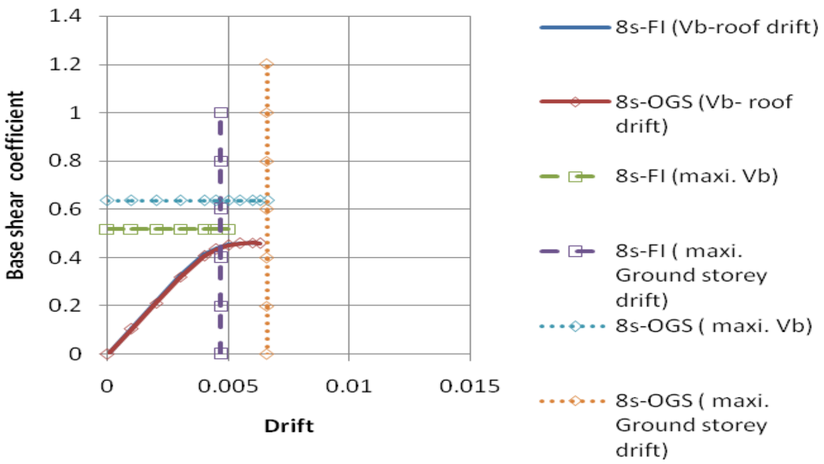
The stiffness difference between FI and OGS frames can be seen from the slope of the initial



(a) 4-storeyed frames

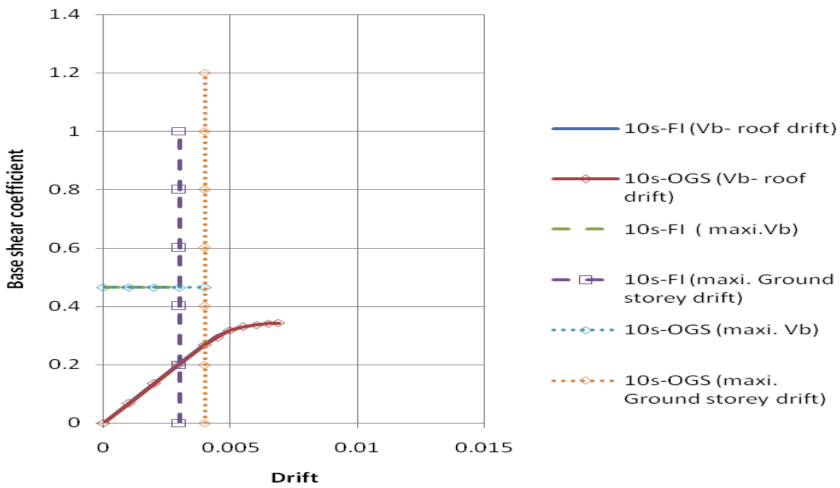


(b) 6-storeyed frames



(c) 8-storeyed frames

Fig. 3 Base shear vs. drift as per pushover analyses and time history analyses



(d) 10-storeyed frames

Fig. 3 Continued

portions of the pushover curves. For 4-storeyed frames, there is considerable difference between the slopes of the two graphs, whereas as the height of frame increases, the difference in stiffness reduces. For 10-storeyed frames, the two graphs are almost the same indicating that there is no effect of open ground storey on the stiffness and strength capacities.

7. Displacement-based design of regular frames

As per DDBD (Priestley *et al*. 2007), an inelastic system is modelled as an equivalent linear elastic analogue having substitute properties of effective stiffness (k_{eff}), effective damping (ζ_{eff}), and effective period (T_{eff}). The effective period of the equivalent SDOF system is estimated by using the substitute structure properties together with an elastic displacement spectrum for equivalent damping. Inelastic displacement of regular frames is given by

$$\Delta_i = \omega_\theta \theta_c H_i \frac{4H_n - H_i}{4H_n - H_1} \tag{4}$$

where the drift reduction factor

$$\omega_\theta = 1.15 - 0.0034 H_n \leq 1.0 \tag{5}$$

and H_i is the height of i^{th} floor level above base, H_1 is the height of first storey and H_n is the total height of frame. The critical drift θ_c is taken as 2% corresponding to life safety performance level.

The properties of the substitute structure are given by

$$\text{Design displacement } \Delta_d = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i}$$

$$\text{Effective mass } m_e = \frac{\sum m_i \Delta_i}{\Delta_d}$$
$$\text{Effective height } H_e = \frac{\sum m_i \Delta_i H_i}{\sum m_i \Delta_i} \tag{6}$$

where m_i and Δ_i are the seismic mass and inelastic displacement of i^{th} floor. The fundamental period corresponding to the secant stiffness at design displacement is used to calculate the base shear, V_B as follows

$$V_B = \frac{4\pi^2 m_e \Delta_d}{T_e^2} \tag{7}$$

This base shear is distributed to various floor levels as follows

$$F_i = F_t + 0.9V_b \frac{m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{8}$$

where F_t=0.1V_b at roof and F_t=0 at all floors.

8. Displacement-based design of OGS frames

From the time history and pushover analyses carried out on FI, bare and OGS frames, it is clear that the stiffness offered by masonry infill walls should also be considered while calculating the design forces for OGS frames. A properly designed OGS frame can attain the displacement profile as that of regular frame and hence no change is required for the inelastic displacement profile (Eq. (4)). Thus, the equivalent structure properties can be determined similar to that of regular frames (Eq. (6)). Also, since there is no change in the displacement demand spectrum, fundamental period of OGS frames obtained from the demand diagram will be the same as that of regular frames. The nominal base shear (V_b*), can be calculated based on this fundamental period using Eq. (7). The actual base shear (V_b) will be more than V_b* and the increase in base shear is due to the increase in lateral force at the first floor level.

While distributing the base shear to various floor levels, the influence of relative stiffness of floors on the base shear distribution should also be considered, in addition to the effect of inelastic displacement i.e., Eq. (8) should be modified based on relative floor stiffness of ground storey. The stiffness ratio parameter for each floor is calculated as

$$\eta_i = \frac{k_i}{k_{av}} \tag{9}$$

where k_i is the stiffness of i^{th} floor and k_{av} is the average stiffness of all the infilled storeys.

Stiffness of any floor can be calculated using Rayleigh’s method (Rayleigh 1945). Lateral loads equal to the seismic weight of each floor ($m_j\,g$) shall be applied at each floor level to find the lateral floor displacement (D_j) of the bare frame by conducting a linear static analysis. The storey stiffness of the bare frame can be found out by

$$K_{j-1} = \frac{\sum_{j}^{n_s} m_j g}{D_j - D_{j-1}} \tag{10}$$

The stiffness of infill walls (Paulay and Priestley 1992) can be determined by

$$K_j^{IF} = \sum_{i=1}^{n_b} 0.25 t_{ij} (E_s)_{ij} \rho_{ij} \cos^2 \theta_{ij} \tag{11}$$

where t_{ij} is the thickness of the infill wall in the i^{th} bay and j^{th} storey, $(Es)_{ij}$ is the modulus of elasticity of infill wall in the i^{th} bay and j^{th} storey, ρ_{ij} is the reduction factor for opening in the infill wall in the i^{th} bay and j^{th} storey and θ_{ij} is the angle made by the equivalent diagonal strut with horizontal. The storey stiffness can be taken as approximately equal to the sum of the storey stiffness of the bare frame and the stiffness of equivalent diagonal strut.

$$k_j = K_j + K_j^{IF} \tag{12}$$

The ground storey stiffness and the average stiffness of upper infilled storeys of the example frames are shown in Fig. 4. It is clear from the figure that the ground storey stiffness increases as height of the frame increases. But the increase in stiffness of infilled storeys with respect to height is only nominal.

The values of $V_{b,OGS}/V_{b,bare}$ given in Table 1 is used for deriving the expression for load distribution for OGS frames. The lateral load at first floor level is given by

$$F_1 = V_b^* \left[\left(\frac{k_{av}}{k_1} \right)^{0.3} - 0.9 \frac{\sum_{i=2}^{n} m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \right] \tag{13}$$

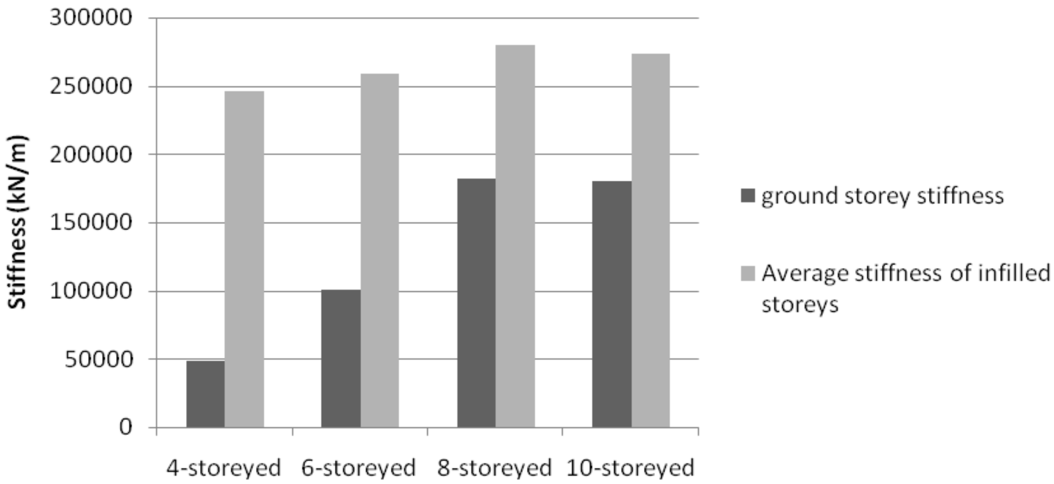


Fig. 4 Ground storey stiffness and infilled storey stiffness of OGS frames

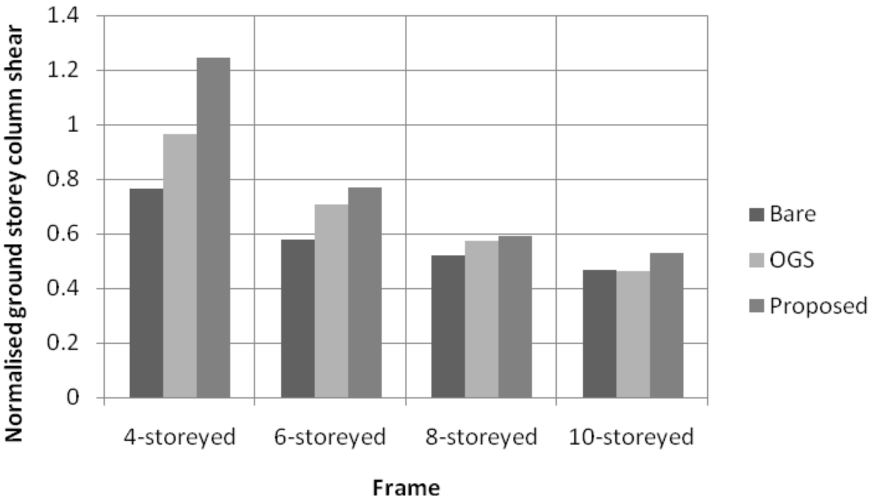


Fig. 5 Normalized ground storey shear as per the proposed load distribution

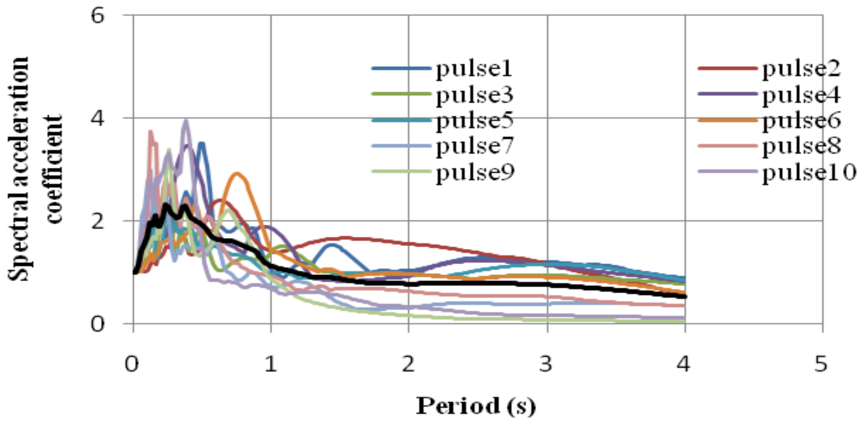
where k_{av} is the average stiffness of all the infilled storeys and k_1 is the stiffness of open ground storey. It is very important to do a preliminary design of the bare frame, for the various load combinations with the nominal base shear applied as the earthquake load, for finding out the storey stiffness of the bare frame. This will give a reasonable estimate of k_1 and k_{av}. As the lateral load for the first floor is very sensitive to the values of k_1 and k_{av}, greater care should be taken to provide the optimum sections for the frame members satisfying the design requirements.

Eq. (8) can be used for the lateral load at any level, except *t* at first floor level. The design base shear V_b will be the sum of all the lateral forces and will be higher than the nominal base shear V_b*, depending on the stiffness of open ground storey relative to the average stiffness of infilled storeys. After getting the lateral forces at various floor levels, the design procedure of OGS frames remains the same as that of regular frames. It is sufficient to design the first floor beams for the nominal base shear V_b* and not for the design base shear V_b.

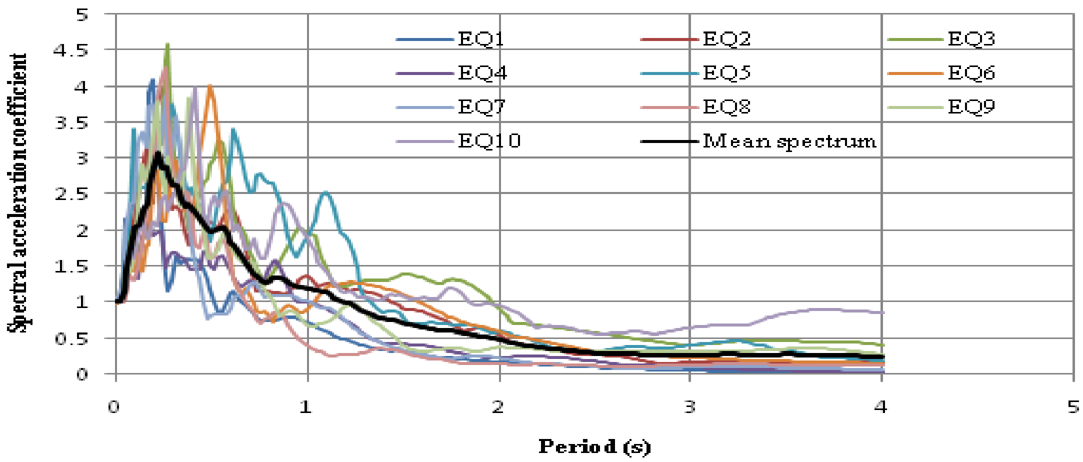
Fig. 5 shows the comparison of storey shear as per the proposed load distribution pattern with the time history analysis distribution for bare frame and OGS frame. The proposed distribution conservatively estimates the ground storey shear for low-rise (4-storeyed) frames, but it predicts the storey shear almost exactly for medium rise (6 and 8-storeyed) frames. For high rise (10-storeyed) frames, the increase in storey shear due to open ground storey is not so significant; the proposed distribution gives higher values of storey shear for such frames.

9. Behaviour of OGS frames under near-field earthquakes

Characteristics of near-fault motions are greatly different from that of ground motions away from the fault. Ground shaking near a fault rupture is characterized by a pulse with very high energy input. Although the response spectrum provides the basis for specification of design ground motions, there is a growing recognition that the response spectrum is not capable of adequately describing the seismic demands presented by brief impulsive near-fault ground motions (Somerville 2000). Near-fault ground motions contain distinct velocity and displacement pulses.



(a) Near field ground motions



(b) Far-field ground motions

Fig. 6 Response spectra for a suite for near field and far-field earthquakes

These pulses can cause high levels of inter-storey drift in structural systems.

A comparison of response spectra obtained using the program “Seismospect” for 10 near-filed and far-field ground motions taken from PEER database are shown in Fig. 6. It is clear from the figure that near-field pulses have higher spectral accelerations at longer periods.

There are cases wherein failure of buildings occurred, even when designed as per the prevalent seismic codes. The newly constructed Olive View Medical Centre in Sylmar (California), was damaged during the 1971 San Fernando Earthquake (Moment magnitude, *Mw*=6.7 and PGA recorded at Pacoima Dam, 1.25 g). The hospital was located 6 miles south-west of the epicentre and 1.5 miles north of the surface faulting. It was a six storeyed RC structure with shear walls in the upper four storeys and not in the ground and first storeys (Chopra *et al*. 1973, Bertero *et al*. 2002). Three acceleration pulses each of 2/3 to 1 second duration were recorded during the earthquake. The design storey shear for ground and first storeys were 0.08 and 0.086 times the dead load of the respective floors whereas the coefficient of seismic resistance during earthquake was calculated as 0.3 and 0.4 respectively. The hospital was rebuilt using RC and steel shear walls

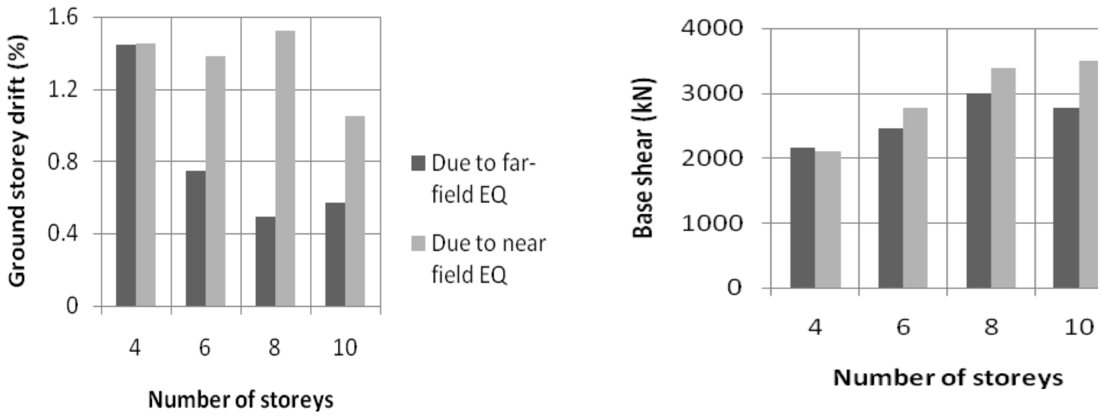


Fig. 7 Response of OGS frames under near-field and far-field ground motions

around the perimeter and the building resisted floor acceleration of 2.8 g without any significant structural damage during the 1994 Northridge earthquake of Mw=6.7 (Bertero and Bertero 2002).

To study the behaviour of OGS frames under near-field earthquakes, the 4, 6, 8 and 10-storeyed OGS frames are subjected to the near-field time histories scaled to a PGA of 0.6 g. The ground-storey drift and base shear of all the frames due to near-field pulse like motions are compared with that due to far-field ground motion and are shown in Fig. 7.

As evident from the response spectra (Fig. 6), long period structures, when subjected to near field motions, showed higher acceleration response, compared to their response under far-field earthquakes. The same is reflected in the inter-storey drift of OGS frames (Fig. 7(a)), i.e., there is an increase in ground storey drift for an increase in frame height, under near field motions in contrast with the response under far-field earthquakes. Based on Fig. 7(b), it can be inferred that the base shear demand of tall frames under near-field earthquakes is more than that under far-field earthquakes, though there is not much increase in the case of short frames. Hence, OGS buildings located near the fault (distance within 10-15 km) should be designed, considering the amplification in response due to the pulse content of near-field earthquake. Inclusion of shear walls at appropriate locations is a better retrofitting measure for OGS buildings due to the high in-plane stiffness offered by the shear walls.

10. Conclusions

Buildings designed by the traditional force-based design (FBD) are prone to damage during earthquakes, as they are designed for a single performance objective and hence may not satisfy other performance objectives. Alternative design methods which design for multiple performance objectives are being evolved. One such alternative is the displacement-based design (DBD) method which aims to design the structure to have a displacement capacity, for a predefined hazard level and can be performed for several limit states, thus ensuring controlled response during all levels of seismic hazard.

Vertically irregular buildings such as open ground storey (OGS) buildings are common types of constructions and they need special design considerations to account for the concentrated damages in the open storey due to stiffness and mass irregularities. The paper presents a modification for displacement-based design (Priestley *et al*. 2000) for application to OGS buildings. The increase in ground storey shear in OGS buildings is determined by performing non-linear time history analyses on bare, fully-infilled and OGS frames. A new load distribution pattern, which depends on the stiffness of ground storey relative to the infilled upper storeys, is proposed. Using this load distribution, DBD of OGS buildings can be done similar to that of regular buildings with simple modification. It is found that tall OGS buildings are not as vulnerable as short buildings, if they are not located in near-fault regions.

For OGS buildings located in near-fault regions, the pulse content of the ground motion may result in increased demand in the ground storey, irrespective of the height of the building. Hence, under such circumstances, it is recommended to perform non-linear time history analyses using ground motions containing pulses and to design the building accordingly. Provision of shear walls in the ground storey will stiffen and strengthen the open ground storey.

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