Earthquake Resistant Design of Low-Rise Open Ground Storey Framed Building

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Abstract:
Building an open ground floor are considered vertically irregular buildings to IS 1893: 2002 requires a dynamic analysis considering the strength and rigidity of infill walls. IS 1893: 2002 also allows the equivalent static analysis (ESA) of CGO buildings ignoring the strength and rigidity of infill walls, provided a multiplication factor of 2.5 is applied to the design forces (moments bending and shear forces) in the columns and beams of a floor on the ground. The objective of this study is to examine the rationality of this approach. A framed existing RC building (G + 3) open floor land located in seismic zone V is analyzed for two different cases: (a) given the filling strength and stiffness and (b) without taking into account the filling strength and rigidity (frame). The infill weight (and associated masses) has been modeled in both cases by applying the static dead load. The infill walls subjected to lateral load behave like diagonal struts. Thus, a filling wall can be modeled as a "compression only equivalent leg in the building model. Rigid joints connect the beams and columns but pin joints connecting the spacers equivalent to the beam-column joints. Infill stiffness was modeled using a diagonal approach according to Smith and Carter (1969). Linear static and dynamic analyzes of the two building styles are performed to compare the strength of demand in the frames open ground floors. The specified multiplication factor code is compared with the ratio of their strength demands. Two different support conditions are taken into account in the analysis to check the effect of supportive conditions for the relative strength frame application. Considered support conditions are: pinned end and fixed-end conditions. Nonlinear static (pushover) analysis is performed for all building models considered. First pushover analysis is made for incremental gravity loads as load control. The lateral pushover analysis is followed after the pushover of seriousness, movement control.

INTRODUCTION

Because of population growth in recent years the car parking space past for residential apartments in populated cities is a major concern. Hence the tendency has been to use the upstairs floor of the building itself for parking. These types of buildings (Fig. 1.1) does not have to fill masonry walls in ground floor, but infilled in all upper floors are called Open Ground Storey (CGO) buildings. They are also known as "open the first storey of the building" (when the numbering of floors begins with one of the ground floor itself), "piles" or "buildings on stilts."

There are significant benefits of these categories of buildings functionally but a seismic performance point of view of these buildings are considered to have an increased vulnerability. According to the latest earthquakes, it was obvious that the main type of failure that occurred in the CGO buildings included hooking the lateral links, crushing of the concrete core, buckling of longitudinal reinforcement bars etc. Due to the presence of filler walls throughout the upper floor, except for the floor in soil makes the upper floors much more rigid than the open floor ground. Thus, the upper stages move nearly simultaneously in a single block, and most of the horizontal movement of the building takes place in the soft ground floor itself. In other words, this type of buildings to sway back and forth like inverted pendulum (Fig. 1.2) during earthquakes, so the columns in the columns and beams floors on the ground are strongly emphasized. Therefore, it is necessary that the columns of one storey floor must have sufficient strength and adequate ductility. The vulnerability of this type of building is attributed to the sudden drop in lateral stiffness and strength to the ground floor, compared to upper floors with infill walls.
The OGS framed building behaves differently compared to a naked framed building (without filling) or completely backfilled under side loading framed building. A bare frame is much less rigid than a fully backfilled framework; it resists lateral load applied by the frame of action and shows distributed plastic hinge breakage. When this part is fully filled in, the lattice action is introduced. A fully backfilled frame shows less inter-storey drift, although it attracts more base shear (due to increased rigidity). A fully backfilled framework gives less force in the frame members and dissipates more energy through infill walls. The strength and stiffness of infill walls in the infilled frame buildings are ignored in structural modeling in the practice of conventional design. Design in such cases will generally be conservative in the case of the framed completely backfilled construction. But things will be different for OGS framed building. OGS building is slightly stiffer than the bare chassis, a larger drift (especially upstairs floor), and fails because of flexible mechanisms floors on the ground floor. Therefore, it may be unwise to ignore the strength and rigidity of the filling wall when designing CGO buildings.

**LITERATURE REVIEW**

A state review of the literature of art is performed as part of this study. This chapter provides a brief summary of the literature review. The literature is divided into two parts. The first part deals with the seismic behavior of buildings open floors to the ground, while the second part of this chapter on the previous work on linear and nonlinear modeling infill walls.

**SEISMIC BEHAVIOR OF CONSTRUCTION OPEN FLOOR FLOOR**

Lateral loading of the frame and the filling wall remain intact initially. As the side load increases the filling wall separates the frame surrounding the (voltage) corner unloaded, but in the compression wedge filler walls are still intact. The length over which the fill wall and the frame are intact is called the length of contact. The load transfer occurs through a perfectly diagonal which acts as a compression strut. Because of this behavior of the filling wall, they can be modeled as an equivalent diagonal brace connecting the two compression diagonal corners. Property rigidity must be such that the strut is only active when subjected to compression. Thus, under the lateral load single diagonal will be operational at a time. This concept was first put forward by Holmes (1961).

**MODELING STRUCTURAL**

It is very important to develop a calculation model on which linear / non-linear, static / dynamic analysis is performed. The first part of this chapter summarizes the different parameters defining the calculation models, assumptions and basic geometry selected for this study considered building. Accurate modeling of non-linear properties of different construction elements is very important in the nonlinear analysis. In this study, the frame elements were modeled with flex hinges inelastic using the point- plastic model. A detailed description of the non-linear modeling of RC frames is presented in this chapter. Infill walls are modeled as diagonal elements equivalents strut. The final section deals with the calculation of the equivalent strut model including nonlinear modeling. Modelling of Moment-Curvature in RC Sections

Using the modified model of Mander stress-strain curves for concrete (Panagiotakos and Fardis, 2001) and Indian Standard IS curve 456 (2000) stress-strain for reinforcing steel for a specific containment steel moment curvature relationships can be generated for the beams and columns (for different levels of axial load). The assumptions and the procedure used to produce the moment-curvature curves are described below.

**Assumptions:**

i. The strain is linear over the height of the section ("plane sections remain plane).

ii. The tensile strength of concrete is ignored.

iii. Spalling the concrete off to a strain of 0.0035.

iv. The initial tangent modulus of concrete, E, c is adopted from IS 456 (2000), as5000ckf

v. For determining the position of the neutral axis, convergence is assumed to be achieved within an acceptable tolerance of 1%.

**Algorithm for Generating Moment-Curvature Relation**

i. Assign a value to the extreme concrete compressive fibre strain (normally starting with a very small value).

ii. Assume a value of neutral axis depth measured from the extreme concrete compressive fibre.

iii. Calculate the strain and the corresponding stress at the centroid of each longitudinal reinforcement bar.

iv. Determine the stress distribution in the concrete compressive region based on the Modified Mander stress-strain model for given volumetric ratio of confining steel. The resultant concrete compressive force is then obtained by numerical integration of the stress over the entire compressive region.

v. Calculate the axial force from the equilibrium and compare with the applied axial load (for beam element both of these will be zero). If the difference lies within the specified tolerance, the assumed neutral axis depth is adopted. The moment capacity and the corresponding curvature of the section are then calculated. Otherwise, a new neutral axis is determined from the iteration (using bisection method) and steps (iii) to (v) are repeated until it converges.

vi. Assign the next value, which is larger than the previous one, to the extreme concrete compressive strain and repeat steps (ii) to (v).

vii. Repeat the whole procedure until the complete moment-curvature is obtained.

**RESULTS FROM LINEAR ANALYSIS**

Seismic analysis is a subset of structural analysis and is the calculation of the response of the building structure to earthquake and is a relevant part of structural design where earthquakes are prevalent. The seismic analysis of a structure involves evaluation of the earthquake forces acting at various level of the structure during an earthquake and the effect of such forces on the behaviour of the overall structure. The
analysis may be static or dynamic in approach as per the code provisions.

Thus broadly we can say that linear analysis of structures to compute the earthquake forces is commonly based on one of the following three approaches.

1. An equivalent lateral procedure in which dynamic effects are approximated by horizontal static forces applied to the structure. This method is quasi-dynamic in nature and is termed as the Seismic Coefficient Method in the IS code.

2. The Response Spectrum Approach in which the effects on the structure are related to the response of simple, single degree of freedom oscillators of varying natural periods to earthquake shaking.

3. Response History Method or Time History Method in which direct input of the time history of a designed earthquake into a mathematical model of the structure using computer analyses.

**EQUIVALENT STATIC ANALYSIS**

This is a linear static analysis. This approach defines a way to represent the effect of the earthquake ground motion when forces of the series are to act on a building, across a seismic design response spectrum. This method assumes that the building meets in its fundamental mode. The applicability of this method is extended in many building codes by applying factors to account for the rise buildings with higher modes, and for low torque levels. To take account of effects due to a “yield” of the structure, many codes apply modification factors that reduce design forces. In the equivalent static method, the lateral force equivalent to the design basis earthquake is applied statically. Lateral forces equivalent to the level of each stage are applied to the design “center of mass places. It is located to the eccentricity of the design of the “center of rigidity (or stiffness)” calculated.

**RESULTS OF NON-LINEAR ANALYSIS**

It is linear (static and dynamic) analysis that the amplification factor of 2.5, as recommended in the Indian Standard IS 1893 (Part 1): 2002 for the design of beams and floors open floor columns are too conservative for low-rise buildings CGO. An effort was made to verify this conclusion of the nonlinear analysis. Pushover analysis is chosen because it is the simplest among the various nonlinear analysis methods. First half of this chapter presents a detailed description of Pushover Analysis and procedure. Half later in this chapter presents results from analyzes of selected pushover building walk-floor open to both end condition attached and fixed. A non-linear analysis involves modeling all the loading elements with the non-linear resistive material. Nonlinear modeling for frame members and infill walls is discussed in Chapter.

**PUSHOVER ANALYSIS**

The pushover analysis is a nonlinear static method that is used in a performance analysis based. The method is relatively simple to implement, and provides information about the strength, deformation and ductility of the structure and distribution of applications that help identify members likely to reach critical limit states during the earthquake and thus appropriate attention can be given during the design and detailing. This method requires an additional set of side load on the height of the structure. local non-linear effects are modeled and the structure is pushed until a collapse mechanism is developed. With the increase in the amplitude of fillers, weak links and failure modes buildings are. At each step, the base shear and roof of the displacement can be plotted to generate the pushover curve (Fig. 5.1). This process is relatively simple and provides information about the strength, deformation and ductility of the structure and distribution of applications. This identifies members likely to reach critical limit states during the earthquake in the formation of plastic hinges. On the frame of load / displacement of the construction is applied gradually, the formation of plastic hinges, degradation of rigidity and inelastic lateral force versus displacement response for the structure is analytically calculated. But some limitations of this method is that it neglects the variation in the load profile, the influence of higher resonance modes and effects. Despite still above shortcomings this method has gained wide acceptance since it provides a reasonable estimate of the global deformation capacity. And also the decision to renovate may be taken on the basis of these studies.

**RESULTS ANALYSIS PUSHOVER**

Pushover analysis is performed for each of the two construction models. First pushover analysis is done for gravity loads (DL + 0.25LL) incremental load under control. The pushover analysis side (PUSH-PUSH-X and Y) is followed after the pushover of seriousness, movement control. The building is pushed in lateral directions until the formation of the collapse mechanism. The capacity curve (base shear relative to the roof movement) is obtained in the X and Y directions and presented in FIGS. 5.3 (a) 5.3 and (b). These figures clearly show that the overall stiffness of a building walk-in open ground hardly changes even if the rigidity of infill walls is ignored. If there is not substantial change in the rigidity elastic base shear demand for the building will also not change significantly if the rigidity of the filler walls is ignored. The variation curves Pushover in X- and Y-directions is in agreement with the linear analysis results presented in the previous section on the variation of the elastic application base shear for the construction of different models.

**CONCLUSIONS:**

Following are the main conclusions obtained from this study:

1. IS Code gives a value of 2.5 to increase the beam floors ground forces and column when a building must be designed as building walk-in open ground or a building on sitlls. The ratio of IR values for columns and beams of DCR values for the two support conditions and model building were found using ESA and RSA and the two analyzes argues that a factor of 2.5 is too high to be multiplied to the beam and column forces of the ground floor. This is especially true for low-rise buildings CGO

2. Buildings OGS problem cannot be properly identified by the elastic analysis that the rigidity of OGS building and Bare-frame building are almost identical.

3. The nonlinear analysis reveals that the OGS building fails through a walk-in mechanism on the ground at a relatively low base shear and displacement. And the failure mode is proving fragile.
4. Analyzes both elastic and inelastic show that beams forces ground storey to significantly reduce the presence of the filling of rigidity to the adjacent stage. And strength design amplification factor should not be applied to the mass of the beams of stages.

5. The linear (static / dynamic) analysis shows that the forces of the column to the floor increases from one floor to the presence of the filler wall on the upper floors. But the design force amplification factor found to be much less than

6. From the literature, it was found that the support provided for the buildings has not given much importance. Linear and nonlinear analyzes show that support condition greatly influences the response and can be an important parameter to determine the factor of amplification force.

2.5. SCOPE FOR FUTURE WORK:

1. Proposed results must be validated by other case studies. Construction of models considered in this study are low rise and thus the influence of the lag period is negligible. For high buildings move in period may be an additional parameter that is not considered in this study.

2. Another area of research across could be the design of infill walls in light of the door and window openings, which was not included in this research.

3. It is in this paper that the multiplication factor of 2.5 as described in IS 1893: 2002 is not justified by the application of elastic force. However, this factor may be necessary to achieve a ductile mode of failure and avoid localized floors mechanisms. This can be studied minute

REFERENCES


