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SEISMIC ANALYSIS AND RETROFIT OF EXISTING MULTI-STOREYED BUILDINGS IN INDIA – AN OVERVIEW WITH A CASE STUDY

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SUMMARY

After the earthquake in Bhuj, Gujarat, in 2001, there has been a concerted effort to address the seismic vulnerability of existing buildings in India. This paper is part of a project, whose aim is to evolve methodologies to assess the seismic vulnerability of reinforced concrete three- to ten-storeyed, residential and commercial buildings and to propose retrofit measures for the structurally deficient buildings.

For the buildings addressed in the project, the common element deficiencies are inadequate shear capacity, core confinement and rebar splicing of columns; inadequate shear capacity, rebar anchorage and plastic hinge rotation capability of beams and inadequate confinement of beam-to-column joints. The presence of soft and weak storey at the open ground floor, in-plane discontinuity and out-of-plane offset of the ground floor columns and eccentric mass are commonly observed irregularities in the studied buildings. In absence of collector elements in the slab and proper detailing of the connections with the building frame, there is lack of integral action of the lateral load resisting elements.

The local retrofit strategies of column, beam, beam-to-column joint, wall and foundation strengthening are reviewed. Under global retrofit strategies, the addition of infill walls, shear walls and steel braces, and the reduction of the building irregularities are mentioned. A detailed case study is reported. In the conclusion, issues pertinent to retrofit are discussed.

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INTRODUCTION

The earthquake at Bhuj, Gujarat, in 2001 has been a watershed event in the earthquake engineering practice in India. The code of practice for seismic analysis, IS 1893:2002 [1] has been revised to reflect the increased seismic demand in many parts of the country. Many existing buildings lack the seismic strength and detailing requirements of IS 1893:2002, IS 4326:1993 [2] and IS 13920: 1993 [3], because they were built prior to the implementation of these codes. This paper is part of a project, whose aim is to evolve methodologies to assess the seismic vulnerability of reinforced concrete (RC) three-to ten-storeyed, residential and commercial buildings, particularly those located in the urban areas of earthquake zones V, IV and III, and to propose retrofit measures for the structurally deficient buildings. Several case studies have been performed and one such case study is presented in this paper. IS 1893:2002 is referred to as the 'Code' henceforth.

RETROFIT

Goals and objectives of retrofit

Retrofit strategy refers to options of increasing the strength, stiffness and ductility of the elements or the building as a whole. Several retrofit strategies may be selected under a retrofit scheme of a building. The goals of seismic retrofitting can be summarized as follows (IS 13935:1993 [4]; White [5]).

1. Increasing the lateral strength and stiffness of the building.
2. Increasing the ductility and enhancing the energy dissipation capacity.
3. Giving unity to the structure.
4. Eliminating sources of weakness or those that produce concentration of stresses.
5. Enhancement of redundancy in the number of lateral load resisting elements.
6. The retrofit scheme should be cost effective.
7. Each retrofit strategy should consistently achieve the performance objective.

To decide the retrofit scheme, a performance based approach can be adopted. The performance based approach identifies a target building performance level under an anticipated earthquake level. For retrofit of the buildings covered in this project, the basic safety objective can be selected. Under this objective, the dual requirement of life safety under design basis earthquake (DBE) and structural stability under maximum considered earthquake is aimed at.

BUILDING DEFICIENCIES

The following two sections highlight some common deficiencies observed in multi-storeyed RC buildings in India (Sinha and Shaw [6]; Murty et al. [7]). The building deficiencies can be broadly classified as Local Deficiencies and Global Deficiencies.

Local Deficiencies

Local deficiencies lead to the failure of individual elements of the building. The observed deficiencies of the elements are summarized.

Columns

a. Inadequate shear capacity.
b. Lack of confinement of column core. Lack of 135° hooks, with adequate hook length.
c. Faculty location of splice just above the floor, with inadequate tension splice length.
d. Inadequate capacity of corner columns under biaxial seismic loads.
e. Existence of short and stiff columns.

Beams and Beam-to-Column Joints

a. Shear reinforcement not adequate for flexural capacity.
b. Inadequate anchorage of bottom rebar.
c. Inadequate plastic hinge rotation capability due to lack of confinement.

Slab-to-Column Connections

a. Absence of drag and chord reinforcement.
b. Inadequate reinforcement at the slab-to-beam connections.

Structural Walls

a. Lack of adequate boundary elements.
b. Inadequate reinforcement at the slab-to-wall or beam-to-wall connections.

Unreinforced Masonry Walls

a Lack of out-of -plane bending capacity.

Precast elements

a. Lack of tie reinforcement.

Deficient Construction

a. Frequent volume batching.
b. Additional water for workability.
c. Inadequate compaction and curing of concrete.
d. Top 100 to 200 mm of column cast separately, leading to deficient plastic hinge region.
e. Inadequate side face cover, leading to rebar corrosion.
f. Poor quality control.

Global Deficiencies

Global deficiencies can broadly be classified as plan irregularities and vertical irregularities, as per the Code. The items left out are listed under miscellaneous deficiencies. Some of the observed irregularities are as follows.

Plan Irregularities

a. Torsional irregularity due to plan symmetry and eccentric mass from water tank.
b. Frequent re-entrant corners.
c. Diaphragm discontinuity due to large openings or staggered floors, along with the absence of collector elements.
d. Out-of-plane offset for columns along perimeter.
e. Nonparallel lateral load resisting systems (not observed in the building studied).

Vertical Irregularities

a. Stiffness irregularity, soft storey due to open ground storey.
b. Mass irregularity (not observed in the building studied).
c. Vertical geometric irregularity from set-back towers.
d. In-plane discontinuity for columns along the perimeter of the building.
e. Weak storey due to open ground storey.

The miscellaneous deficiencies that were observed are as follows.

Deficiencies in Analysis

a. Buildings designed as only gravity load resisting system.
b. Neglecting the effect of infill walls.
c. Inadequate geotechnical data to consider near source effects.
d. Neglecting the P-Δ effect.

Lack of integral action of the lateral load resisting elements
The building performance is degraded due to the absence of tying of the lateral load resisting elements. The beams are not framed into the elevator core walls and spandrel beams between the perimeter columns are missing.

Failure of stair slab
If the stair slab is simply supported without adequate bearing length, a collapse of the slab closes the escape route for the residents.

Pounding of buildings
Another poor design concept is not providing adequate spacing between adjacent buildings or seismic joints between segments of a building.

RETROFIT STRATEGIES

Retrofit strategies that are viable for the type of buildings considered, are grouped under local and global strategies. These groups need not be watertight and strategies falling in either group are expected.

Local Retrofit Strategies

Local retrofit strategies include local strengthening of beams, columns, slabs, beam-to-column or slab-to-column joints, walls and foundations. Local strengthening allows one or more under-strength elements or connections to resist the strength demands predicted by the analysis, without affecting the overall response of the structure. This scheme tends to be the most economical alternative when only a few of the building's elements are deficient. The local retrofit strategies are grouped according to the elements.

Column Strengthening
Column strengthening techniques include the following.

a. Concrete jacketing
b. Steel jacketing
c. Fibre reinforced polymer sheet wrapping

<u>Concrete Jacketing</u>
This method increases both strength and ductility of the columns. But, the composite deformation of the existing and the new concrete requires adequate dowelling to the existing column. Also, the additional longitudinal bars need to be anchored to the foundation and should be continuous through the slab. Frequently, these considerations are ignored.

<u>Steel Jacketing</u>
Steel jacketing refers to encasing the column with steel plates and filling the gap with non-shrink grout. It is a very effective method to remedy deficiencies such as inadequate shear strength and inadequate splices of longitudinal bars at critical locations (Aboutaha [8]). But, it may be costly and its fire resistance has to be addressed.

Fibre Reinforced Polymer Sheet Wrapping

The use of Fibre Reinforced Polymer (FRP) sheets is becoming popular in India (Mukherjee and Joshi [9]). FRP sheets are thin, light and flexible enough to be inserted behind service ducts, thus facilitating installation. In retrofitting of a column there is no significant increase in the size. The main drawbacks of FRP are high cost, brittle behavior and fire resistance.

Beam Strengthening

Addition of Concrete

There are some disadvantages in this traditional retrofit strategy. First, addition of concrete increases the size and weight of the beam. Second, the new concrete requires proper bonding to the existing concrete. Third, the effects of drying shrinkage must be considered as it induces tensile stresses in the new concrete. Instead of regular concrete, fibre reinforced concrete can be used for retrofit.

Steel Plating

Gluing mild steel plates to beams is often used to improve the beam flexural and shear performances. The addition of steel plate is simple and rapid to apply, does not reduce the storey clear height significantly and can be applied while the structure is in use. Glued plates are of course prone to premature debonding (Swamy et al. [10]).

FRP Wrapping

Like steel plates, FRP laminates are attached to beams to increase their flexural and shear capacities. The amount of FRP attached to the soffit should be limited to retain the ductile flexural failure mode. Bonacci and Maalej [11] listed the failure modes of beams, strengthened with FRP laminates.

Use of FRP bars

FRP bars can be attached to the web of a beam for shear strengthening (Lorenzis and Nanni [12]). FRP bars can be used as tendons for external prestressing.

Beam–To-Column Joint Strengthening

The different methods of strengthening are as follows.

Concrete Jacketing

The joint can be strengthened by placing ties through drilled holes in the beam (Stoppenhagen et al. [13]). But the placement of such ties is difficult.

Concrete Fillet

Bracci et al. [14] suggested the use of a concrete fillet at the joint to shift the potential hinge region away from the column face to the end of the fillet.

Steel Jacketing

Steel jacketing helps in transferring moments and acquiring ductility through confinement of the concrete. Ghobarah et al. [15] proposed the use of corrugated steel jackets. Steel plating is simpler as compared to steel jacketing, where plates in the form of brackets are attached to the soffits of beams and sides of the column.

FRP Jacketing

The studies of El-Amoury and Ghobarah [16] have shown that the retrofitted specimens exhibit better efficiency in terms of strength, energy dissipation, lesser rate of stiffness degradation and ductility levels.

Wall Strengthening

A concrete shear wall can be strengthened by adding new concrete with adequate boundary elements. For the composite action, dowels need to be provided between the existing and new concrete. Steel braces or strips (Taghdi et al. [17]), FRP or steel sheets, external prestressing or reinforced grouted core can be employed for strengthening unreinforced masonary walls.

Foundation Strengthening

Foundation strengthening is done by strengthening the footing as well as the soil (FEMA 356 [18]).

Global Retrofit Strategies

Global retrofit strategies aim to stiffen the building, by providing additional lateral load resisting elements, or to reduce the irregularities or mass.

Structural Stiffening

Addition of Infill Walls

The addition of masonry infill wall is a viable option for the buildings, with open ground storeys, addressed in the project. Of course masonry infill walls increase strength and stiffness of the building, but do not enhance the ductility. Infill walls with reinforced concrete masonry units can act as shear walls. For cast-in-place RC infill walls, the significant parameter that defines the lateral strength of the frame is the presence of dowels between a wall and the bounding frame. The use of modular precast panels involves minimal on-site casting and modest handling equipment. Connections between the panels and the frame are critical. Use of infill steel panels is an alternative to bracing system.

Addition of Shear Walls

New shear walls can be added to control drift. Critical design issues involved in the addition of shear walls are as follows.

a Transfer of floor diaphragm shears into the new wall through dowels.

b Adding new collector and drag members to the diaphragm.

c Reactions of the new wall on existing foundations.

Addition of Steel Braces

A steel bracing system can be designed to provide stiffness, strength, ductility, energy dissipation, or any combination of these. Connection between the braces and the existing frame is the most important aspect in this strategy. The uses of prestressed tendons and unbonded braces have been proposed by some investigators to avoid the problems associated with the failure of connections and buckling of the braces, respectively.

Reduction of Irregularities

Torsional irregularities can be corrected by the addition of frames or shear walls. Eccentric masses can be relocated. Seismic joints can be created to transform an irregular building into multiple regular structures. Partial demolition can also be an effective measure, although this may have significant impact on the utility of the building. Discontinuous components such as columns can be extended beyond the zone of discontinuity. As mentioned earlier, walls or braces can alleviate the deficiency of soft and weak storey.

Mass Reduction

Reduction of mass results in reduction of the lateral force demand, and therefore, can be used in specific cases in lieu of structural strengthening.

Energy Dissipation Devices and Base Isolation

For the multi-storeyed buildings addressed in this paper, the use of energy dissipation and base isolation devices is not cost effective at present. Hence, these devices are not addressed.

A CASE STUDY

The building presented in this paper is a residential, ordinary moment resisting RC framed building, located in Zone III. Figure 1 shows the typical floor plan of the building. The building is a four storeyed building. The height of the roof is 13.1 m from the ground level. Plan dimensions of the building are 20.47m $\times$13.29m. The construction drawings specify that M20 grade of concrete (characteristic cube compressive strength is 20 N/mm^2) and Fe 415 grade of steel (characteristic 0.2 percent proof stress is 415 N/mm^2) were used for the construction.

The floor slabs in the building were assumed to act as rigid diaphragms. This assumption leading to integral action of all the frames is of course debatable in absence of drag and chord reinforcements. The slabs are 150mm thick for all the floor levels. The infill wall thickness was assumed to be 230mm for the exterior walls and 120mm for the interior walls, as is the common practice in India. The subsoil for the building was considered to be Type-I soil (as per the Code), as majority of the standard penetration test values of the soil were more than 30 according to the geotechnical report. The elevation of the building along grid line A-A is shown in Figure 2.

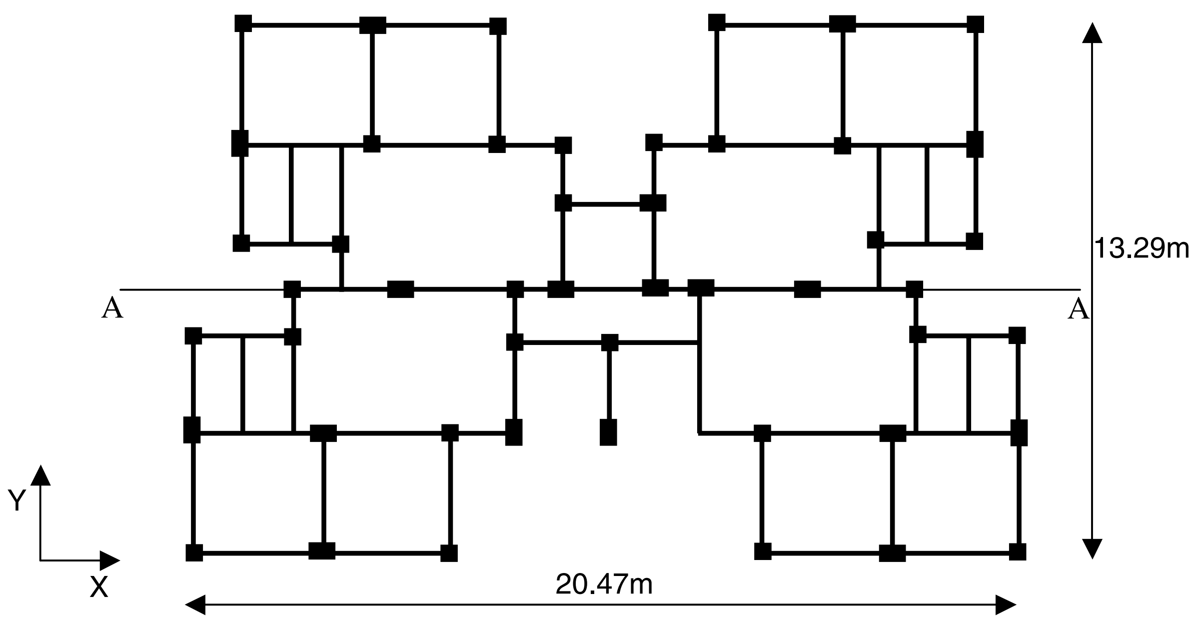
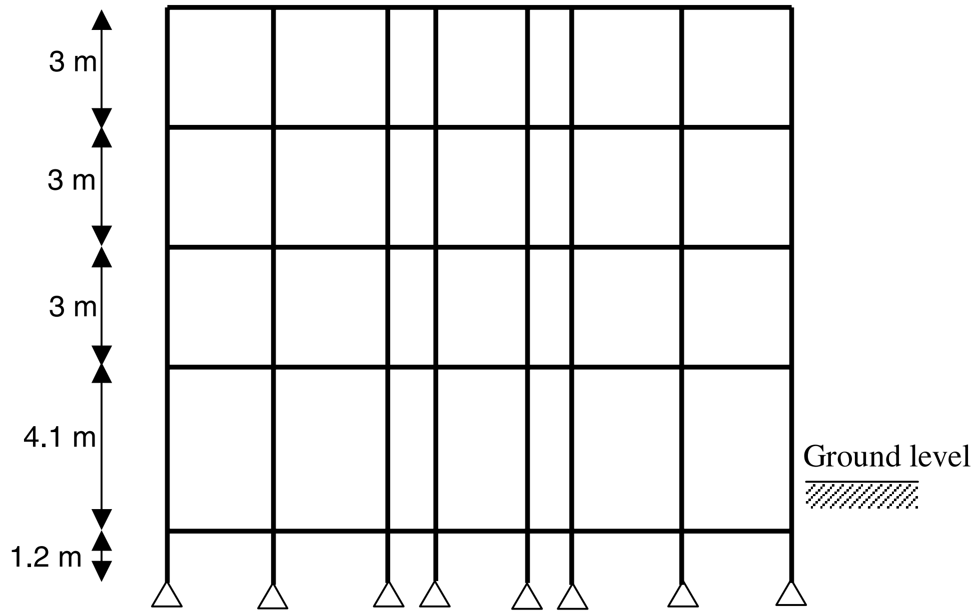


Figure1: Typical floor plan of the building showing column and beam locations



Preliminary Evaluation

The evaluation statements as given in FEMA 178 [19] were used in the preliminary evaluation of the building. Only the negative attributes are mentioned here. The total of cutout areas is less than 50 percent of the plan dimension. In absence of collector elements, this may lead to diaphragm discontinuity. Re-entrant corners are present in the building as both the projections of the structure beyond each re-entrant corner are greater than 15 percent of the corresponding plan dimensions. The beams are stronger than the columns. Reinforcement details like hook lengths, splice lengths, tie spacing in columns, joint reinforcement were not mentioned in the available drawings.

The center line of the plinth beams are located 1.1m below the ground level. This increases the height of the ground storey.

DETAILED ANALYSIS

The building was analysed using equivalent static method (linear static method) and response spectrum method (linear dynamic method) according to the Code. Pushover analysis (non-linear static method) was also carried out to study the deformation of the building. The analyses were done by using the finite element analysis software, SAP2000.

Structural Modelling

The analytical model of the building includes all components that influence the mass, strength and stiffness. The non-structural elements and components that do not significantly influence the building behaviour, were not modelled. Stair cases and water tanks were not modelled, but their masses were appropriately included. Beams and columns were modelled as frame elements with the centerlines joined at nodes. Rigid offsets were provided from the nodes to the faces of the columns or beams. The stiffness for columns and beams were taken as $0.7EI_g$, accounting for the cracking in the members and the contribution of flanges in the beams.

The weight of each slab panel was distributed to the surrounding beams based on the tributary areas. The total mass of the slab at each floor level was lumped at the centre of mass. The later was located at the design eccentricity (based on the Code) from the calculated centre of rigidity. The centre of rigidity for each storey was found out seperately. The columns of the storey were assumed to be fixed at the bottom. A unit force along X direction and a unit moment were applied at a certain test point in the top of the storey and the corresponding rotations were noted down. The distance of the centre of rigidity from the test point, along Y direction, was calculated from the ratio of the two rotations. Similarly the distance along X direction was found out by applying a unit force along Y direction.

The foundation system for the building consists of an isolated footing under each column. The bottom of each column was assumed to be pinned. The effect of soil-structure interaction was ignored in the analyses. Since the slab-on-grade at the ground floor was not connected to the plinth beams, the mass and stiffness of the slab were neglected.

Modelling of infill walls

The weight and mass of all the brick masonry walls were applied on the supporting beams. When an infill wall is located in a lateral load resisting frame, the stiffness and strength contribution of the infill wall were considered by modelling it as an equivalent diagonal compression strut. In a moment resisting frame, the inclusion of equivalent struts leads to a truss frame model. The beams and columns are connected by rigid joints, but the equivalent struts are connected by pin joints at the beam-to-column junctions.

The required properties of an equivalent strut are the effective width, thickness, length and elastic modulus. The thickness is assumed same as that of the infill. The length is available from the dimensions of the corresponding infill panel. The elastic modulus E_i is equated to E_m, the elastic modulus of the masonry. As per UBC 1997 [20], E_m is given as $E_m = 750\, f_m'$, where f_m' is the basic compressive strength of the masonry. Thus, the only remaining property to be determined is the effective width of the equivalent strut. For a nonlinear analysis, such as push-over analysis, in addition to the above properties, the axial load versus deformation behavior along with the failure load of the equivalent strut are also required.

The effective width (w) was found to depend on the following three variables (Smith and Carter [21]).

a. The relative stiffness of the infill to the frame, expressed in terms of λh. λ is the relative stiffness parameter and h is the height of the infill.
b. The magnitude of the instantaneous diagonal load in the infill (R).
c. The aspect ratio of the infill panel.

The following expression of *w*, in terms of λh and R/R_c was found to be adequate (Ramesh [22]). The influence of aspect ratio of the panel is neglected in this expression.

$$w/w' = 1.477 + 0.0356\lambda h - 0.912\left(R/R_c\right) \tag{1}$$

The expression of $w^{/}$ is given as

$$\left(\frac{w'}{d}\right) = \frac{0.43 \sin 2\theta}{\sqrt{\lambda h}} \qquad (2)$$

Here, d is the length of the strut and R_c is the diagonal load at failure. R_c can be calculated based on the crushing and shear modes of failure (Smith and Carter [21]). The building was also checked without considering the equivalent struts, for the lower bound case of all infill walls undergoing out-of-plane bending failure.

Loads

The dead loads were calculated from the member sections. The live load was taken as specified in the construction drawings. The design spectrum for Type-I soil as specified in the Code, was selected to calculate the spectral acceleration coefficient (S_a/g). For zone III, the seismic zone factor Z is 0.16. The building was considered to be an ordinary moment resisting frame, with a base shear reduction factor R equal to 3. Considering standard occupancy, the importance factor I was taken as 1. The orthogonal effects were considered by applying 100 percent of the design lateral forces in the X direction and 30 percent in the Y direction and vice versa. Wind load was not considered in the analyses. Absolute demands for each frame element under dead load, live load, and earthquake load were calculated from the load combinations given in the Code.

Pushover analysis

The fundamental time period was found to be less than one second, and as per ATC 40 [23] recommendations, the pushover analysis is applicable for this building. For pushover analysis, the beams and columns were modelled with concentrated plastic hinges at the column and beam faces, respectively. Beams have only moment (M3) hinges, whereas columns have axial load and biaxial moment (PMM) hinges. The moment-rotation relations and the acceptance criteria for the performance levels of the hinges were obtained from ATC-40 [23]. As the shear strengths of all the beams and columns were found to be more than the respective shear demands (from equivalent static and response spectrum methods), no shear hinge was modelled in the frame elements. The equivalent struts were modeled with axial hinges (entire length of the strut was considered as hinge length), that have a brittle load-deformation relation only for compression.

Pushover analysis was performed in presence of gravity loads, with monotonically increasing lateral loads, distributed according to the Code. Analyses were performed independently in the X and Y directions. To achieve life safety (LS) performance level under DBE, the target displacement at the roof was taken as 1.2 percent of the building height. The value of the target displacement was 172mm. The coefficients C_a and C_v in SAP 2000 were adjusted to model the design spectrum as per the Code. Geometric nonlinearity of the structure due to P-Δ effect was considered in the pushover analyses.

RESULTS AND DISCUSSIONS

Table 1 shows the comparison of design base shears of the building, with and without infill stiffness. $\overline{V_B}$ is the base shear by equivalent static method. EQ_x and EQ_y represent the earthquake loads acting in the X and Y directions, respectively. The base shear from response spectrum analysis (V_B) was calculated from the modal combination of first ten modes, by the SRSS method. These modes give more than 99% mass participation in both the directions. V_x and V_y are the components of EQ in X and Y directions, respectively. As V_B was less than $\overline{V_B}$, the seismic force demands in the frame elements from response spectrum analysis were scaled up by a factor equal to the ratio of the two base shears $(\overline{V_B}/V_B)$.

Table 1: Comparison of Base Shears

	Without infill stiffness		With infill stiffness	
	V_x (kN)	V_y (kN)	V_x (kN)	V_y (kN)
Equivalent static method $\left(\overline{V_B}\right)$				
EQ_x	554.6	–	746.2	–
EQ_y	–	528.5	–	711.1
Response spectra method $\left(V_B\right)$				
EQ	264.1	237.9	484.6	569.2
$\overline{V_B}/V_B$	2.10	2.22	1.54	1.25

For beam sections, positive and negative bending moment and shear demands were compared with the respective capacities. A number of beam sections were found to have deficient flexural capacity. However, all the sections have sufficient shear capacity.

The adequacy of each column section for flexure and axial compression was checked using a three dimensional interaction surface for axial compression and biaxial bending, generated according to IS 456:2000 [24]. The maximum demand from all the load combinations was plotted along with the corresponding interaction surface. A straight line was drawn joining the demand point and the origin. The ratio of the length of the straight line to the distance between the point of intersection of the straight line with the interaction surface and the origin, is termed as the capacity factor. Analysis results show that a number of column sections do not satisfy the code criteria as the capacity factor exceeds 1. For none of the columns, shear demand exceeds shear capacity.

For some of the ground storey struts, axial force demands exceed the respective capacities. The elastic storey drift for every storey due to the design lateral force, with partial load factor of 1, was calculated. The building satisfies the Code drift limitation of 0.004. Since the modelled height of the ground storey is high compared to the other storeys, the ground storey has higher drift. This indicates excessive ductility demand in the ground storey columns.

From the pushover analysis, it was found that the building has larger strength as compared to the factored base shear $1.5\overline{V_B}$ (Figure 3). But, the building did not reach the target displacement of 172mm. Thus the building does not have adequate ductility. Moreover, the pushover analyses in either direction failed to give a performance point before the formation of mechanism. So the performance of this building is not acceptable. It needs to be retrofitted. The flexural hinges in ground storey columns went beyond life safety performance level.

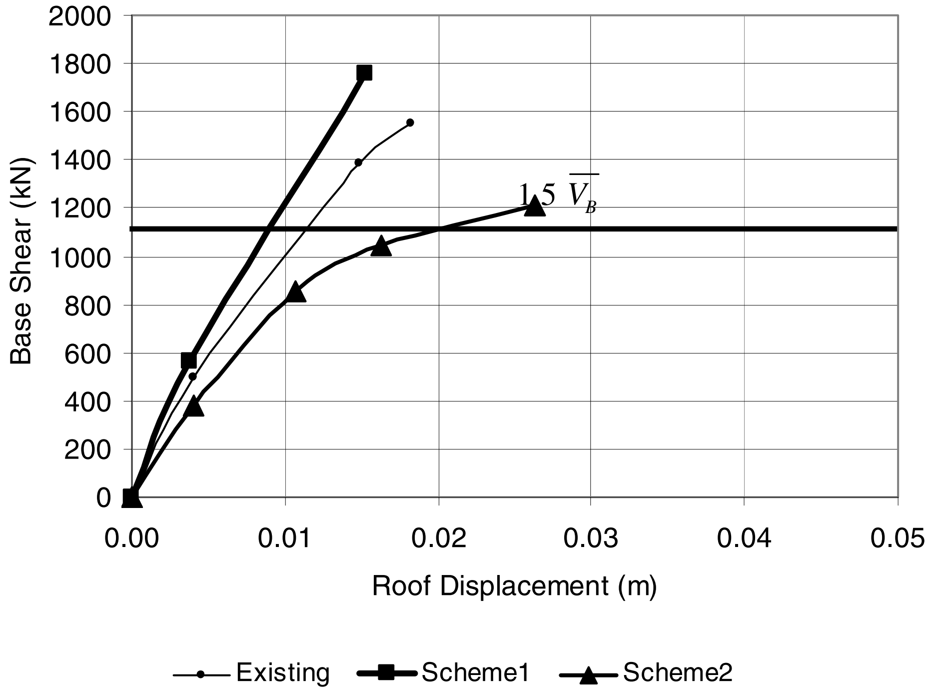


Figure 3: Pushover curves for existing and retrofitted models along X direction

RETROFITTING OF BUILDING

From the seismic evaluation it was determined that building needs retrofit to resist the design earthquake demand. To retrofit the building a number of possible retrofitting schemes were tried. Two schemes that are economically feasible are presented here. In Scheme 1, to enhance the capacities of the columns in the ground storey (including the parts between the plinth beams and top of the footings), local retrofitting of the columns by concrete jacketing was adopted. An equivalent section of each jacketed column was used in the analysis. From Figure 3 it is observed that Scheme 1 is adequate regarding strength, but it does not enhance the ductility. Performance point was achieved in the X direction, but not in the Y direction.

In Scheme 2, to reduce the stiffness of the building and hence the lateral force demand, selected ground storey infill walls were made non-integral to the frame. This can be achieved by drilling gaps at the corners of the walls and filling those gaps with caulking material. This scheme gives adequate strength, as well as ductility before the mechanism formed. The pushover analyses in both the directions gave performance points. However, at each performance point, hinges in a few columns and beams were beyond life safety performance level. These columns and beams need additional local retrofit. Figure 4 shows the fundamental periods and the corresponding spectral acceleration coefficients (as per the Code) for the existing and retrofitted models of the building. It is observed that compared to Scheme 1, Scheme 2 gives less lateral load demand.

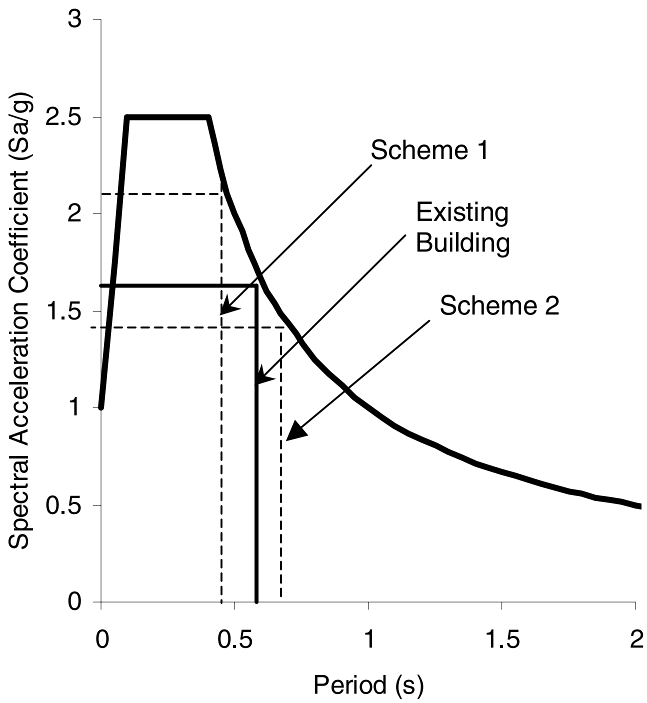


Figure 4: Comparison of fundamental periods

CONCLUDING REMARKS

The paper presents a review of the existing retrofit strategies that are applicable for multi-storeyed residential reinforced concrete buildings addressed in the project. It also presents a case study of a three storeyed building, located in an urban area in earthquake zone III.

The following aspects are noteworthy from the case study.
1. The absence of plinth beams at the ground storey level increases the vulnerability of the ground storey columns. Although the ground storey was not a weak or soft storey as per the Code definition, it is susceptible to large drift.

2. Inclusion of the infill walls by the equivalent strut method shows substantially high base shear. The effect of openings in the infill walls was not considered. This effect needs to be further investigated.

3. In a retrofit scheme, a combination of local and global retrofit strategies may be required. The appropriate scheme to be adopted is of course building specific. The case study of this building shows that a scheme which increases ductility is more effective. The inclusions of shear walls or steel braces were not investigated in this case study. The effects of the lateral loads on the soil bearing pressure and the demands on the footings are not reported.

4. A retrofit strategy is a solution to a specific problem in a building or in its components. The challenge is to develop guidelines that are generic in nature.

5. When a new element is added to an existing structure or component, the load transfer and the compatibility of deformation between the new and the existing elements are crucial. A retrofit strategy will only be successful when the new element is able to share the load as well as can deform along with the existing components of the building.

6. The quality of construction for a successful retrofit scheme cannot be overemphasized. Any sort of patchwork will be a wasted effort. The participation of the owners, real estate promoters, architects, engineers and contractors is necessary to allay the fear of the residents of the multi-storeyed buildings.

ACKNOWLEDGEMENT

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