# 5. SOFTWARE FOR DAMAGE AND RETROFITTING

## 5.1 GENERAL

Performance Based Analysis is now being used the world over to assess the performance capacity of buildings under the influence of earthquake forces. Inventory of different categories of buildings along with their performance categories have been prepared for most important towns in the earthquake prone zones. Thus the evaluation and inventories of structures in earthquake zones are definitely in place. It is important now that while planning for the disaster event, these categories of seismic performance of the structures, are taken into account while implementing the advanced techniques.

Emergency response after a moderate or large earthquake generally follows three phases [83]. The first phase is the immediate response, when emergency actions, including damage assessment, need to be taken following a prescribed plan. Swift action is needed to evacuate personnel from hazardous conditions, and retrieval of important inventories and records. The second phase involves the detailed assessment of all facilities to determine their long term safety and usability. The third phase covers the needed repairs and at times reconstruction. The first phase can last up to thirty-six hours, the second up to seven days, and the last phase can extend for many years depending on the extent of damage.

The building inspection portion of any emergency preparedness program can be augmented with a set of post-earthquake inspection notes that contain procedures for investigations, lists of deficiencies, and drawings for all the buildings. For each building which does not meet the life-safety standards (or any other performance criteria that has been fixed for it), the strengthening exercise should be undertaken, so that the structure is able to resist future earthquakes, as no structure can be made economically earthquake-proof [84].Thus emergency response planning involves both pre and post-earthquake preparedness in order to minimize losses.

The outcomes would include :

- A systematic seismic evaluation and upgrade program for buildings.
- A complete seismic inventory that allows for postulating scenario events as a long-term planning tool.
- A rigorous process for the seismic anchorage of equipment, piping, and other non-structural elements to provide greater assurance of post-earthquake functionality.
- A continuous monitoring process to ensure that the latest advances in the field are incorporated into program.

The present chapter examines various aspects of disaster preparedness in terms of evaluation methods for determining the damage due to Earthquakes, Various inherent local and global deficiencies in buildings are also highlighted in the context of performance during an earthquake. The retrofit strategies which are most commonly adopted are then examined. The software prepared consists of modules to evaluate the demand capacity ratio, design and details of various retrofit options, and finally the output files for each option selected. The next module is in 3DS Max for implementing the local retrofit options for beam and column strengthening on a virtual reality platform.

## 5.2 EVALUATION OF DAMAGE DUE TO EARTHQUAKE

The purpose of a comprehensive assessment should be "to assess the seismic resistance of the structure for the present criteria and to check the feasibility of alternative practical schemes of retrofitting keeping in mind the aspects of economy and convenience to the owners". The seismic evaluation is nothing but a comparison between the "demand" that the earthquake places on the structure to a measure of the "capacity" of the building to resist the same. This will also help assess the possible seismic response of a building, which may be seismically deficient or earthquake damaged, for its possible future use. The general approach recommended is one of examining the deficiencies in the existing structure in order to determine the principal requirements for additional strength, stiffness, or deformation capacity.

### 5.2.1 Identification of Seismic Damage

Identification of a single cause of damage to buildings is not possible. There are combined reasons, which are responsible for multiple damages and may lead to progressive collapse. It is difficult to classify damage, and even more difficult to relate it in a quantitative manner due to the dynamic character of the seismic action of the structures. The principal causes of damage to buildings are listed below [85]

#### 5.2.1.1 Soft storey failure

Owing to the high cost of land and small sizes of plots, parking is often accommodated in the ground storey area of the building. In contrast to the infilled upper stories, the open ground story may cause:

1) Soft storey effect: as it has smaller stiffness and causes increased deformation demand in the frame members of the open ground story; and
2) Weak storey effect: it may have lower lateral strength and cause a discontinuity in the flow of lateral seismic shear in the open ground storey.

**Fig. 5.1 Open Ground Storey**

The interesting feature in the **Fig.5.1** is the transverse dimension of the column of 230 mm. Some buildings were provided with the RC walls of elevator cores.

These stiff elements drew large seismic shears and experienced severe diagonal shear cracking at the ground floor (**Fig. 5.2**).

**Fig 5.2 Diagonal Shear Cracks in RC Elevator Core**

#### 5.2.1.2 Floating columns

In the upper stories, the perimeter columns of the ground storey are discontinued, and floating columns are provided along the overhanging perimeter of the building. These floating columns rest at the tip of the tapered overhanging beams without considering the increased vulnerability of lateral load resisting system due to vertical discontinuity. Buildings have balconies overhanging in the upper stories beyond the column footprint area at the ground story (**Fig. 5.3**); These buildings with open ground story, floating columns and randomly infilled upper stories have both in-plane and out-of-plane irregularities in strength and stiffness.

**Fig 5.3 Building with Floating Columns**

### 5.2.1.3 Plan and mass irregularity

With no expansion or separation joint, irregularities in plan (**Fig.5.4- 5.5**) (C and U shape), mass; stiffness and strength result in significant torsional response. These associated torsional effects may be attributed to collapse of buildings.

**Fig 5.4 Collapse of Flexible Side of Apartment Due to Plan Irregularity**

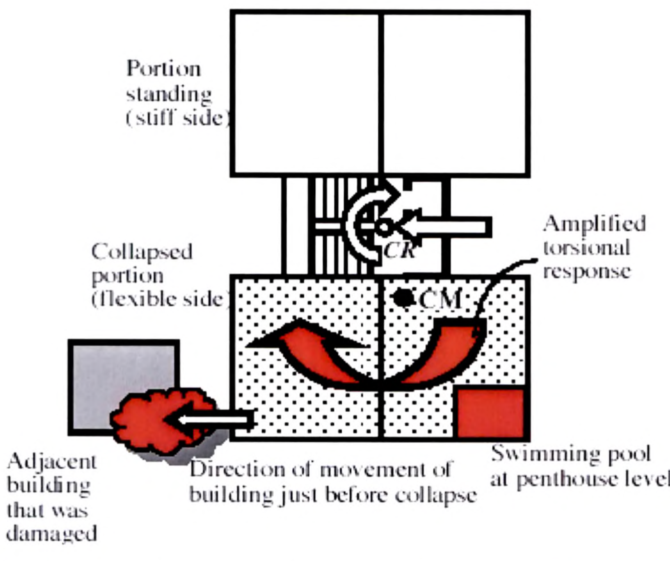



**Fig. 5.5 Schematic Representation of Failure**

### 5.2.1.4 Short column effect

Sometimes, infills of partial heights create short columns, especially around window/ventilator openings or at balconies (**Fig. 5.6**). Unreinforced masonry infills placed up to partial height in frame panels adjoining the columns reduce column height. Such columns draw shear forces larger than they are designed for and lead to failure. This effect is called the short column effect.

**Fig. 5.6 Short Column Effect**

#### 5.2.1.5 Lack of confining reinforcement

Detailing practice for transverse ties in columns in the affected area offers very light confinement to the core concrete against the large compressive stress generated by the extreme lateral deformation demands during strong seismic shaking.

**Fig. 5.7 Shear Cracking of Non-Ductile Columns**

Many times, the MS ties were found to be 5 mm in diameter. The 90° hooks of transverse reinforcement (at 200mm centers) opened-up and resulted in buckling of longitudinal bars and consequent dilatation of core concrete. Shear cracking of non-ductile columns in an open ground storey building is shown in **Fig. 5.7**.

#### 5.2.1.6 Water tanks on roofs

Water required in buildings is usually stored at the roof level in RCC tanks either resting on the roof slab or only nominally connected to it (**Fig. 5.8**). These tanks experience large inertia forces due to amplification of the ground acceleration along the height of the building.

**Fig. 5.8 Water Tank Failure**

### 5.2.2 Methods of Seismic Evaluation

Available methods for seismic evaluation can be categorized into Qualitaitve and Analytical methods as indicated below.

**Methods for seismic evaluation**

- **Qualitative Methods**
  - Condition Assessment
  - Visual Inspection
  - Non-Destructive Testing
- **Analytical Methods**
  - Demand Capacity Ratio
  - Push Over Analysis
  - Inelastic Time History

The qualitative methods involve field tests, wherein the structural components are tested on site or in the laboratory. Each of the analytical methods has its own significance and limitations. One of the major differences between the analytical methods is the method of analysis adopted. The Demand Capacity Ratio method adopts linear dynamic (Response Spectrum) analysis, Nonlinear static analysis is performed in Push Over analysis, while Inelastic Time History takes up Nonlinear dynamic analysis which is the most detailed method for evaluation.

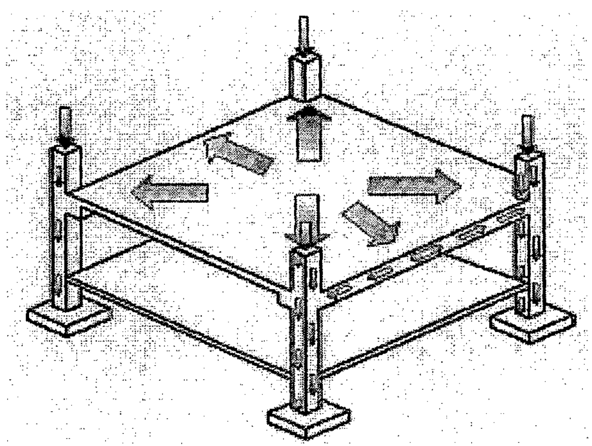
### 5.2.3 Preliminary Evaluation

The main purpose of the preliminary evaluation is to quickly identify buildings that comply with the provisions of code. In addition, it helps the design professional to familiarize himself with the building, its potential deficiencies and its potential behavior. It is an approximate procedure based on conservative parameters to identify all potential earthquake risk buildings and can be used to screen buildings for detailed evaluation. Besides this, a site visit will be conducted by the concerned engineer to verify existing building data or collect additional data, determine the condition of the building and its components.

### 5.2.4 Configuration Related Checks

#### 5.2.4.1 Load path

Lateral force-resisting systems should be continuous and should run from foundation to top of the building. The flow of seismic forces originated in the structure, should be such that these forces are delivered through structural connections to horizontal diaphragms; the diaphragms distribute these forces to vertical lateral force resisting elements such as shear walls and frames; these vertical elements transfer the forces through the foundation into the soil as can be seen in Fig.5.9.



.Fig. 5.9 Load Path

### 5.2.4.2 Geometry

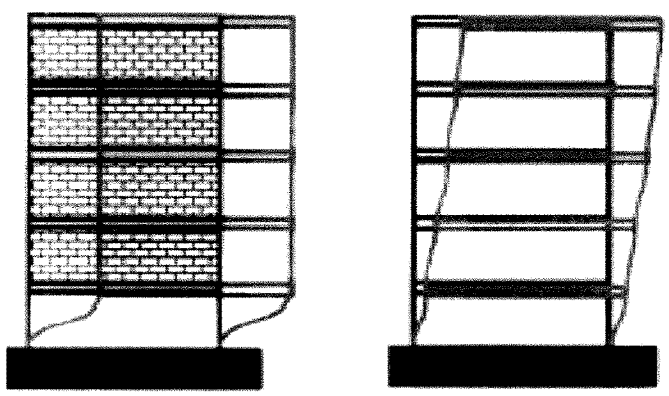
The plan configuration should always be symmetrical with respect to two orthogonal directions. It is also recommended that the criteria specified in I.S: 1893 (Part 1) 2002 for irregular buildings should be followed.

### 5.2.4.3 Weak storey

The strength of the lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent storey. Weak stories are usually found where vertical discontinuities exist, or where member size or reinforcement has been reduced. Compliance can be achieved if the elements of the weak storey can be shown to have adequate capacity near elastic levels.

### 5.2.4.4 Soft storey

The stiffness of lateral load resisting system in any storey shall not be less than 60% of the stiffness in an adjacent storey or less than 70% of the three storeys above (**Fig.5.10**). A clear-cut distinction between soft and weak storey is usually not possible but there is a stark contrast between stiffness and strength. Stiffness is the force needed to cause a unit displacement and is given by the slope of the force-displacement relationship. Whereas, strength is the maximum force that a system can take. Soft storey refers to stiffness and weak storey refers to strength. Usually, a soft storey may also be a weak storey. A change in column size can affect both strength and stiffness, and both need to be considered.



**Open Ground Storey** **Bare Frame**

**Fig 5.10 Severe Soft-Storey Deformation Due to Seismic Shaking**

### 5.2.4.5 Vertical discontinuities

Vertical discontinuities are usually found where elements are not continuous to the foundation but stop at an upper level and can be detected by visual observation. The most common example is a discontinuous shear wall (**Fig. 5.11**) or braced frame. Compliance can be achieved if an adequate load path to transfer seismic forces exists, and the supporting columns can be demonstrated to have adequate capacity to resist the overturning forces generated by the shear capacity of the discontinuous elements.

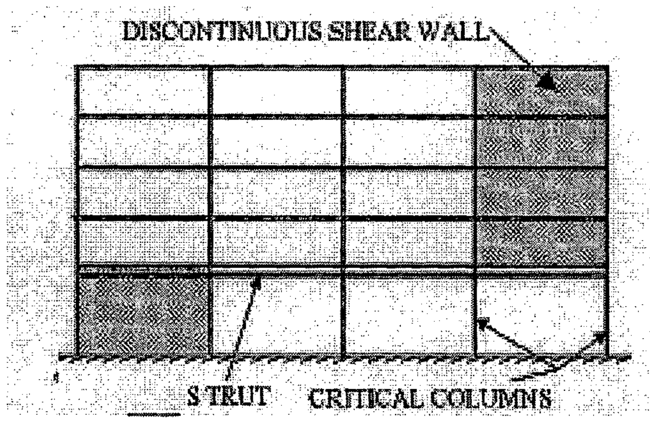



**Fig. 5.11 Discontinuous Shear Wall**

### 5.2.4.6 Mass

Mass irregularity is the presence of heavy mass on a floor or when one floor is much heavier than rest of the others, e.g. heavy machinery installed on any intermediate floor of the building. In case of unavoidable situations or non-compliance, the ratio of mass to stiffness of two adjacent storeys should be made equal.

### 5.2.4.7 Torsion

Theoretically it is required that the center of mass and center of stiffness of a building should coincide with each other, but in most of the cases it is not possible to fulfill this criteria. However, the eccentricity should be kept to its minimum in order to minimize the torsional effects. Buildings not complying with the criteria are not expected to perform well in earthquakes.

#### 5.2.4.8 Adjacent buildings

The builders some times make buildings too close to each other in order to make full use of the plot size. But during seismic shaking two adjacent buildings may hit each other due to lateral displacements. This is known as pounding or hammering. Building pounding affects the dynamic response of both buildings and additional inertia loads are induced on both structures. Buildings shown in Fig 5.12 (b) have more damaging potential compared to that shown in Fig 5.12(a)

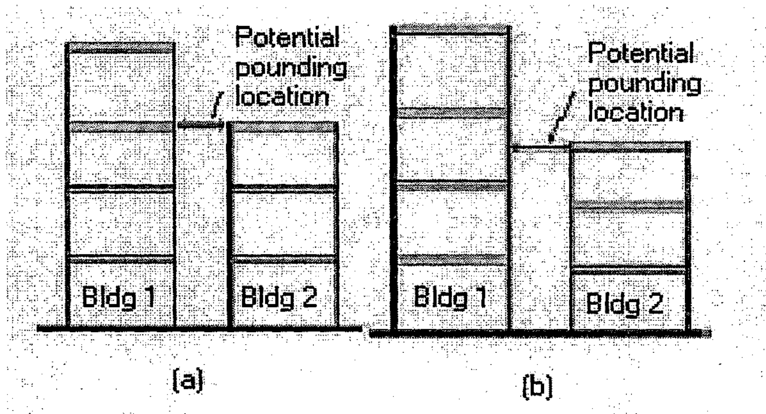



**Fig. 5.12 Pounding in Building**

### 5.2.5 Supplementary Evaluation

In addition to the detailed evaluation for buildings, a few more criteria need to be considered for RC buildings as described below:

#### 5.2.5.1 Shear failure

Shear capacity of frame members shall be adequate to develop the moment capacity at the ends, and shall be in accordance with provisions of IS:13920 [101] for shear design of beams and columns. When the column attains shear capacity before its moment capacity, it develops a potential for a non-ductile sudden failure, leading to collapse. Thus the column should be designed in such a way that it does not reach its shear capacity before reaching its flexural capacity. As shear capacity of a column is affected by axial loads also, the design should be based on most critical combination of axial load and shear.

#### 5.2.5.2 Strong column - weak beam

The sum of the moment of resistance of the columns shall be at least 1.1 times the sum of the moment of resistance of the beams along each principal plane of the frame joints. Failure of columns before failure of beams can lead to a storey mechanism and catastrophic failure. This may cause large displacement, which also can lead to instability of the whole structure and progressive failure. So for ductile failure the failure of the beams are preferred to failure of columns. Hence capacity of columns should be larger than the capacity of beams with due consideration of the over strength of the beams also.

### 5.2.6 Acceptability Criteria

A building is said to be acceptable if it meets all the configuration-related checks, as they ensure presence of coherent load path for lateral loads, and identification of structural features or conditions, which may lead to poor performance of structures in earthquake scenario [21].

### 5.2.7 Detailed Evaluation

The detailed evaluation procedure is based on determining probable strength of lateral load resisting elements taken as the seismic capacity of the individual component and comparing them with the expected seismic demands. The probable strengths determined from conventional methods and applicable codes shall be modified with appropriate knowledge factor defined later. Further, seismic demand on critical individual components shall be determined using seismic analysis methods described in IS: 1893 for lateral forces.

The key steps of the detailed evaluation procedure are as follows:

1. Estimate the probable flexural and shear strengths of the critical sections of the members and joints of vertical lateral force resisting elements. These calculations shall be performed by the conventional design methods as per respective codes for various building types.
2. Calculate the fundamental period of the building and total lateral force (design base shear) by calculation of seismic loads.

3. Perform a linear static, linear dynamic or non-linear static analysis of lateral load resisting elements of the building, in accordance with IS: 1893 or in its absence ATC 40 norms [80], for the base shear determined in the previous step and determine resulting member actions for critical components.
4. Evaluate the acceptability of each component as well as the structure as a whole by comparing its probable strength with the member actions.
5. Estimate the inter-storey drifts and decide whether it is acceptable in terms of the requirements of IS:1893 [.

## 5.3 BUILDING DEFICIENCIES

Building deficiencies can be classified into three categories as: Local, Global and Miscellaneous deficiencies.

### 5.3.1 Local Deficiencies

Local deficiencies lead to the failure of individual elements of the building such as flexural and shear failure in beams, columns and shear walls, crushing of masonry walls and failure of beam to column joints or slab to beam or slab to column connections.

### 5.3.2 Global deficiencies

Certain structural design concepts that may work adequately in non-seismic scenarios, perform poorly when subjected to earthquake. Examples are frame structures with strong beams and weak columns, or frame structure employing open ground storeys. Global deficiencies can broadly be classified as plan irregularities and vertical irregularities, as per IS 1893 (Part I): 2002 [4].

### 5.3.3 Miscellaneous Deficiencies

Various miscellaneous deficiencies are (a) Deficiencies in analysis such as designed as a gravity load resisting system, neglecting the effect of infill, inadequate geotechnical data, neglecting the P-delta effect. (b) Lack of integral action due to poor detailing (c) Failure of stair slab (d) Pounding of building.

## 5.4 SEISMIC RETROFIT STRATEGIES

There is no list or catalog for seismic retrofit strategies, a variety of systems and materials are being used as stand alone or in combination. The retrofit strategies that are viable for buildings, are grouped under local and global strategies (**Fig.5.13**). The groups need not be watertight and strategies falling in either group may be expected [26] to work well in a combination.

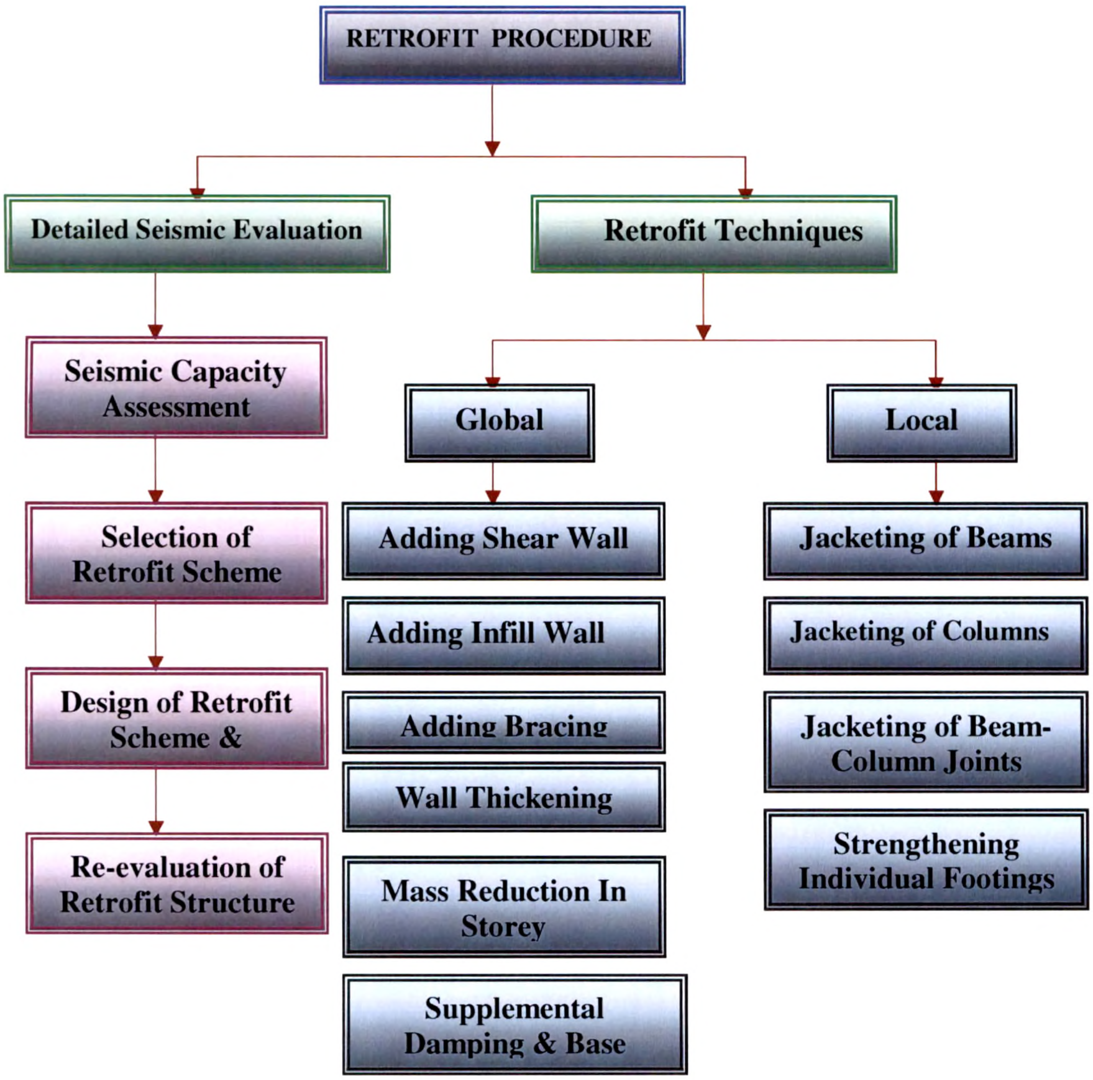



**Fig. 5.13 Retrofitting Strategies**

### 5.4.1 Local Retrofit Strategies

Local retrofit strategies include local strengthening of beams, columns, slabs, beam-to-column, slab-to-beam 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 significantly affecting the overall response of the building. This scheme tends to be an economical alternative when only a few elements of the building are deficient.

#### 5.4.1.1 Column strengthening

*a.* ***Concrete jacketing*** involves addition of a thick layer of RC in the form of a jacket, using longitudinal reinforcement and closely spaced ties with ductile detailing (**Fig. 5.14**). The method is comparatively straight forward and increases both strength and ductility.

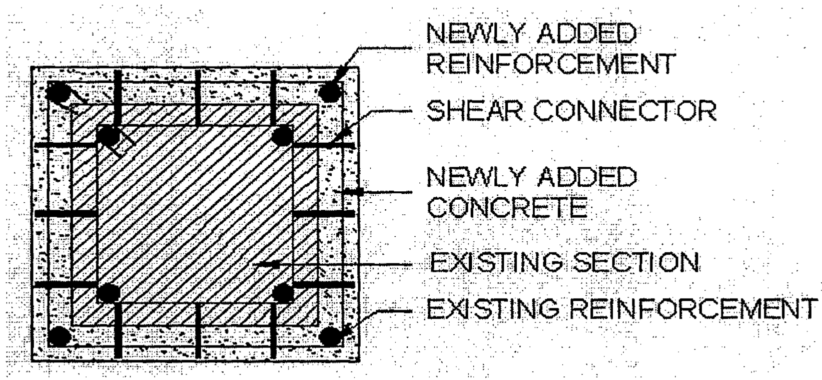



**Fig. 5.14 Concrete Jacketing**

*b.* ***Steel jacketing*** refers to encasing of the column with steel plates and filling the gap with non-shrink grout (**Fig. 5.15**). Steel jacketing is a very effective method to remedy deficiencies such as inadequate shear strength and inadequate splicing of longitudinal bars at critical locations.

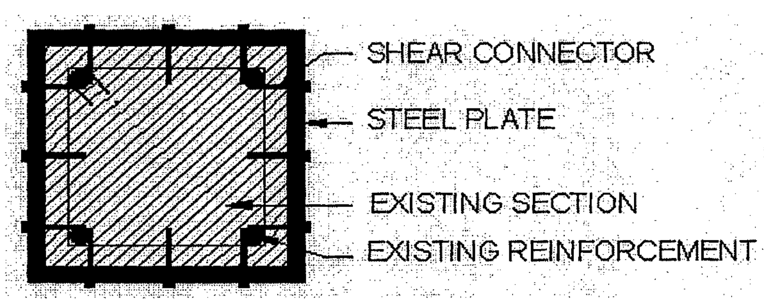



**Fig. 5.15 Steel Jacketing**

*c.* ***Fibre reinforced polymer sheet wrapping*** Fibre reinforced polymer (FRP) is a composite material forming a matrix of polymeric material reinforced with uni-directional or multi-directional fibres. FRP sheets are thin, light and flexible enough to be inserted behind pipes, electrical cables and other service ducts, thus facilitating installation. In retrofitting of a column there is no significant increase in size. The main drawbacks of FRP are high cost, brittle behaviour and low fire resistance.

*d.* ***Prestressed wire wrapping*** An enhanced form of confinement may be achieved by wrapping prestressing wire in tension onto a column. This technique is not suitable for columns of small diameter in small sized buildings.

### 5.4.1.2 Beam strengthening

*a.* ***Addition of concrete*** Concrete is added to increase the strength and stiffness of the beam. Several options are available for adding concrete (Fig. 5.16). There are some disadvantages in this traditional retrofit strategy such as : Addition of concrete increases the size and weight of the beam; New concrete requires proper bonding to the existing concrete; Effect of drying shrinkage must be considered as it induces tensile stress in the new concrete. Instead of regular concrete, fibre reinforced concrete can be used for retrofit.

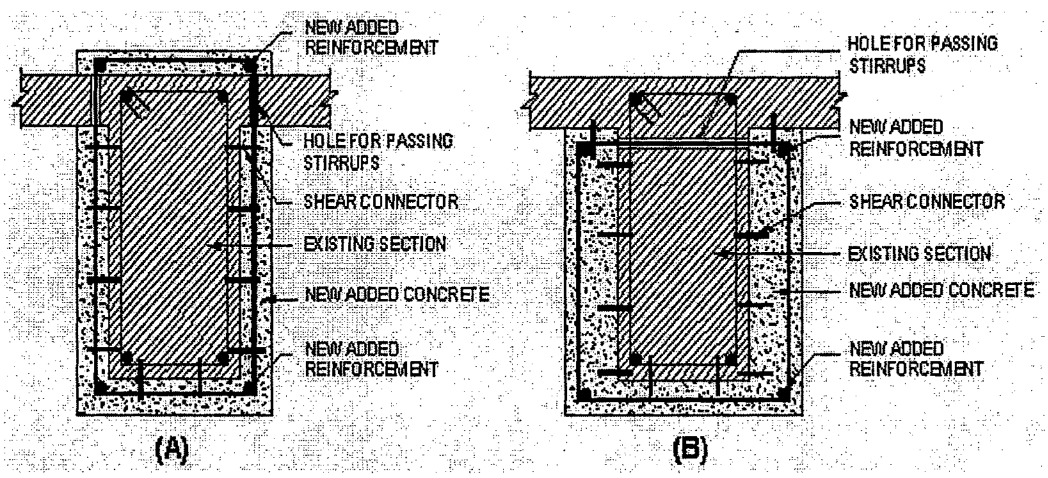



**Fig. 5.16 Additional RC around Existing Beam**

*b.* ***Steel Plating*** The technique of gluing steel plates to beam surface is used to improve its flexural and shear performance (Fig.5.17) It increases the strength and stiffness of the beams and subsequently, reduces the crack width. The addition of steel plates is simple method and is rapid to apply, besides it does not affect storey clear height significantly and can be applied while the structure is in use.

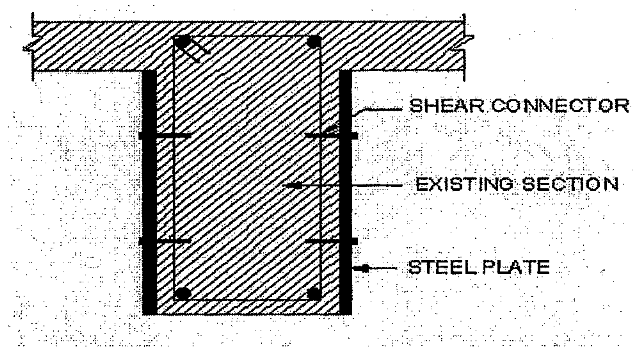



**Fig. 5.17 Steel Plating on Beam Sides**

*c.* ***FRP wrapping*** Like steel plates, FRP laminates are attached to beams in order 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.

*d.* ***Use of FRP bars*** FRP has been used not only as sheets but also as rebar. FRP bars can be attached to the web of a beam for shear strengthening.

*e.* ***External prestressing*** Post-tensioned reinforcement is used to increase the flexural capacity or to replace prestressing strands.

### 5.4.1.3 Beam to column joint strengthening

Under seismic excitation, the beam to column joint region is subjected to very high horizontal and vertical shear forces. Hence, the joint should be carefully detailed to meet the shear strength requirements. Different methods of strengthening include Concrete Jacketing, Concrete Fillet, Steel Jacketing, Steel Plating, and FRP Jacketing.

### 5.4.1.4 Shear 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 (**Fig.5.18**). Other alternatives include Steel braces or stirrups bolted to the shear wall and FRP or steel sheets added to strengthen walls for out-of-plane bending.

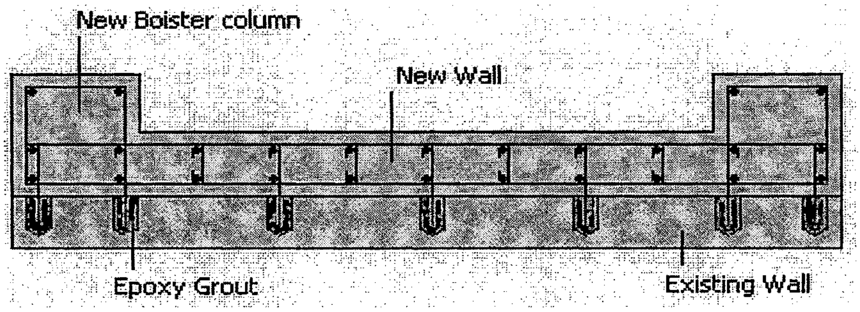



**Fig. 5.18 Strengthening of Shear Wall**

#### 5.4.1.5 Foundation Strengthening

Foundation strengthening is done by strengthening the footing as well as the soil (FEMA 356, 2000). The following measures may be effective in the rehabilitation of footings.

a) New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames.
b) Existing spread footing may be enlarged to increase the capacity
c) Existing spread footing may be underpinned to increase the bearing capacity. Underpinning improves the bearing capacity by lowering the contact level of footing.
d) Uplift capacity may be improved by increasing the resisting soil mass above the footing.
e) Lateral displacements of the footings can be mitigated by interconnecting them with grade beams, grade slab or ties.

### 5.4.2 Global Retrofit Strategies

An effective way to improve the performance of a building is to stiffen the structure by providing additional lateral load resisting elements, so that it has reduced lateral deformation. Construction of new braced frames, infill walls or shear walls is an effective measure for adding stiffness. Reduction of irregularities or mass in a building is another method of global retrofit. Sometimes it may be necessary to combine both local and global retrofit strategies.

#### 5.4.2.1 Structural stiffening

A building can be stiffened by addition of infill wall or shear wall or addition of steel bracing. The addition of new shear wall is used to retrofit lightly reinforced concrete frame buildings to control displacements that can guarantee post-earthquake operability. A steel bracing system can be designed to provide stiffness, strength, ductility, energy dissipation, or any combination of these. The brace can be added to the existing structure either on the exterior frames or interior frames without disrupting the building use too much.

#### 5.4.2.2 Reduction of irregularities

Plan and vertical irregularities are a common cause of undesirable performance under an earthquake. Removal of the irregularities may be sufficient to reduce demands to acceptable levels. Torsional irregularities can be corrected by the addition of moment frames, braced frames, or shear walls to balance the distribution of stiffness and mass. Discontinuous components such as columns or walls can be extended beyond the zone of discontinuity.

#### 5.4.2.3 Mass reduction

Reduction in mass results in direct reductions of the amount of forces and deformation demand, and therefore, can be used in lieu of structural strengthening. Mass can be reduced through demolition of upper storeys, replacement of heavy cladding and interior partitions, removal of heavy storage and equipment loads, or change in the use of the building.

#### 5.4.2.4 Energy dissipation devices and base isolation

Energy dissipation devices and Base isolation can be used to improve the performance of the building against earthquakes. Such devices not only add damping but also provide supplemental stiffness and strength to a building from the ground motion.

## 5.5 COMPUTER IMPLEMENTATION FOR RETROFITTING

The program for Seismic evaluation and retrofit of existing buildings has been developed in VC++ using Microsoft Foundation Classes (MFC) functioning on the windows platform. The basic source code has been developed in the OOP environment with a pre- and post- processor.

### 5.5.1 Pre Processor

This takes the required data from the user for evaluation and strengthening of existing elements along with that for design of new elements. The cross section details, material details along with the reinforcement requirements for each component are to be provided through dialog boxes. Dialog Box is an interactive and effective tool for getting user input and assigning user defined values to the variables. To create each dialog box, one C++ class is developed.

### 5.5.2 Evaluation

Evaluation module of the program uses the data entered through the dialog boxes. The evaluation results are presented in different files for individual components. The final output file presents the entire evaluation methodology in detail for the given building. User can visualize output file after successful evaluation using Output option from the main menu of the screen.

### 5.5.3 Retrofit Options

This module includes various conventional methods adopted for retrofitting. The sub-modules include Concrete and Steel jacketing for Beam and Column, Shear Wall Strengthening, addition of New Column, Shear Wall and Bracing. The choice of option is based on the deficiencies present in the individual components. The final choice is based on the experience and judgment of the user. For every option, the input data is entered through dialog boxes; and the output file of each is prepared separately. The final output file gives the retrofit option selected by the user.

### 5.5.4 Post Processor

Post processor gives the detailed drawings and output files for Beam, Column, Bracing and shear wall. These drawings are prepared by taking the input from the output files prepared by the respective VC++ modules. They are not to scale but maintain the aspect ratio to give good visual effect.

### 5.5.5 Classes Used

The AppWizard is first used to generate an SDI (Single Document Interface) application, which results in development of four main classes namely; Application class, Document class, View class and MainFrame window class. Each class is provided with one header file (.h) and one implementation file (.cpp). The primary tasks are divided among these four classes. In addition to these four classes, the application has also uses other derived classes as shown in **Figure 5.19**.

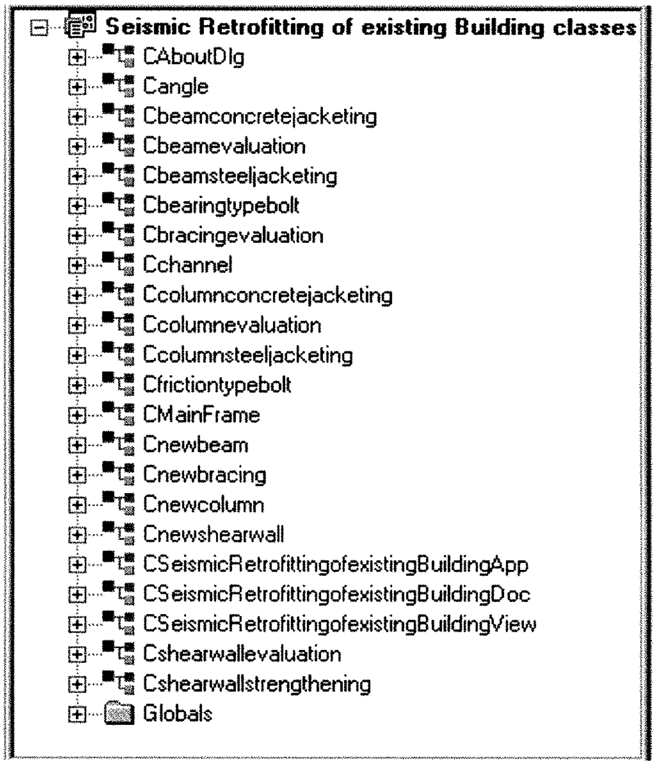



**Fig. 5.19 Classes used in Evaluation and Retrofit Modules**

### 5.5.6 Design of New Element

#### 5.5.6.1 New beam design module

This module uses the input data taken through the dialog box shown in **Fig. 5.20**. Various data taken by this module are width, depth, top and bottom cover, grade of concrete and steel, design moment and shear force acting on the beam. Using this data program will find out area of steel required at top and bottom face of beam along with the shear reinforcement details based on IS: 456 (2000) and IS:13920 ( 1993 )

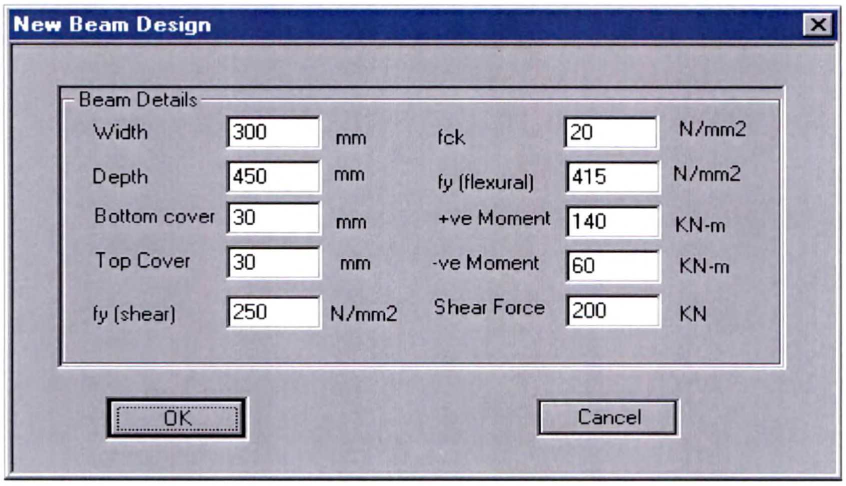



**Fig. 5.20 Input Dialog box for New Beam Design**

### 5.5.6.2 New column design module

This module uses the input data taken through the dialog box as shown in Fig.5.21. Data taken by this module includes width, depth, length, cover, grade of concrete and steel, design axial force, top and bottom moment and shear force acting on the column, along with steel distribution pattern and type of column whether braced or unbraced. Program will give transverse and longitudinal reinforcement required to resist given loading on the column based on IS: 456-2000 and IS: 13920-1993.

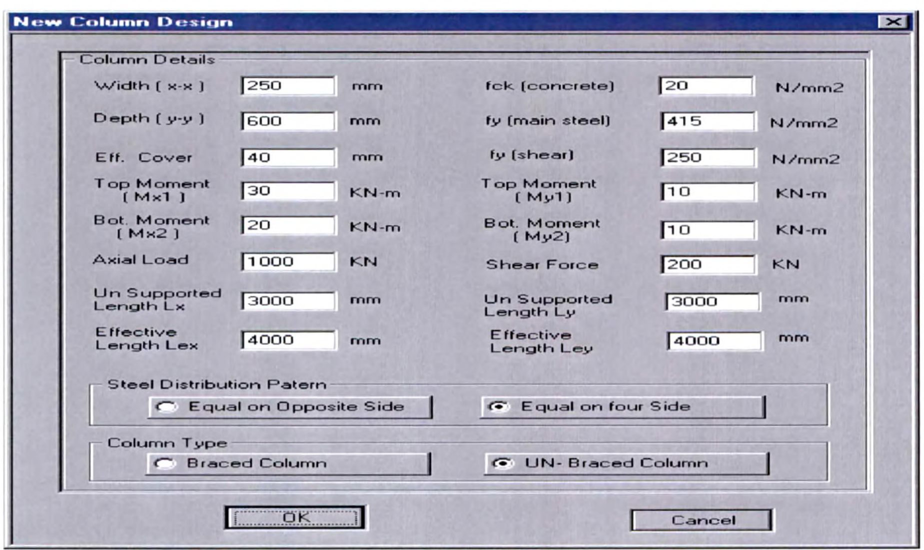



**Fig. 5.21 Input Dialog box for New Column Design**

### 5.5.6.3 New bracing design module

This module takes input through the dialog box shown in Fig.**5.22**. Data taken includes width, depth, length of column and beam, grade of steel, design axial tensile and compressive force, along with type of section for bracing and type of bolt to be used for connection. Another dialog opens to get the parameters for bolts (**Fig. 5.23**).

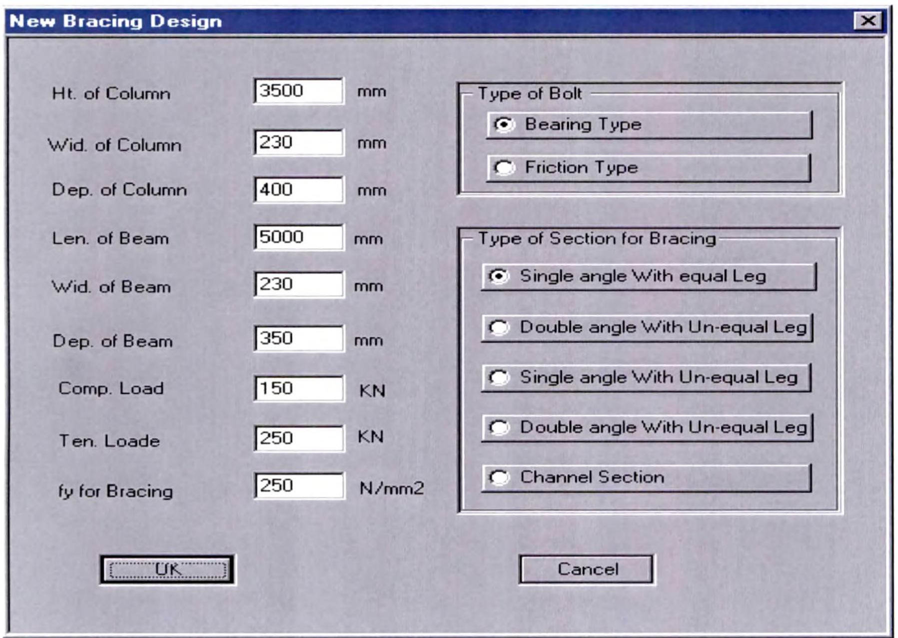



**Fig. 5.22 New Bracing Design Input**

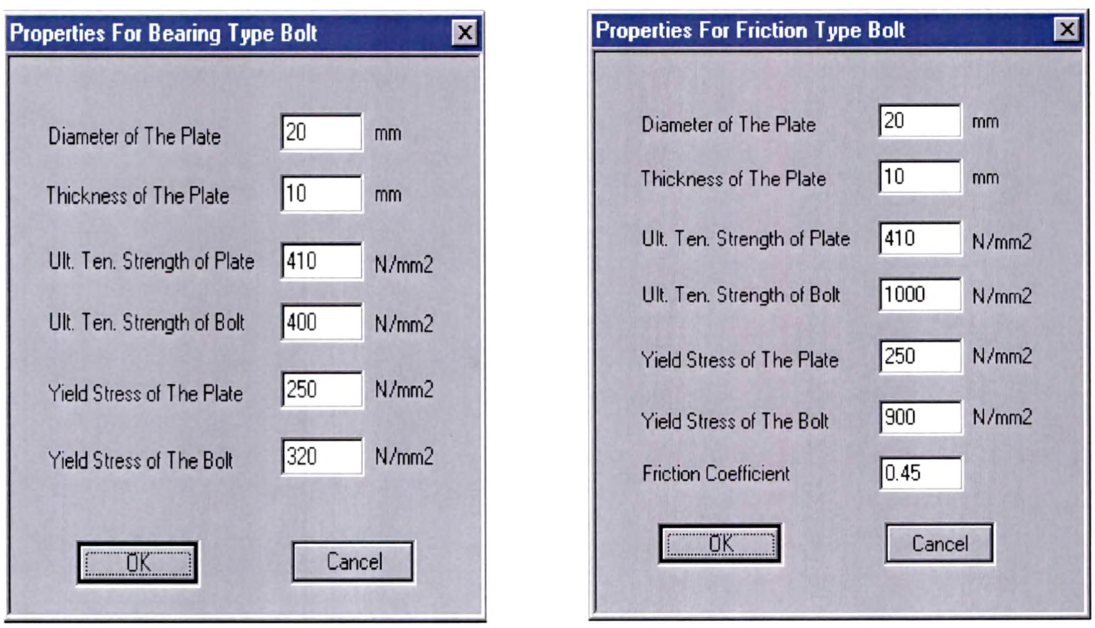



**Fig. 5.23 Properties of Bolt**

Depending on the type of bolt the user wishes to adopt, the program will open the relevant dialog box , which is in turn linked to the back-end data base file corresponding to selected type of section. Using input data, the module will find out the most critical section based on IS: 800 draft code to resist design axial compressive and tensile forces.

#### 5.5.6.4 New shear wall design module

Data take by this dialog box (**Fig. 5.24**) for new shear wall design includes wall details like length, thickness, height, cover, diameter of bar etc, details of boundary element along with material and load detail. The entire design of shear wall is as per IS: 13920. This module starts with minimum amount of reinforcement to check all the requirement of IS: 13920-1993 and gradually increases the amount of reinforcement till it is finalized when all criteria of IS: 13920 for shear wall design are fulfilled.

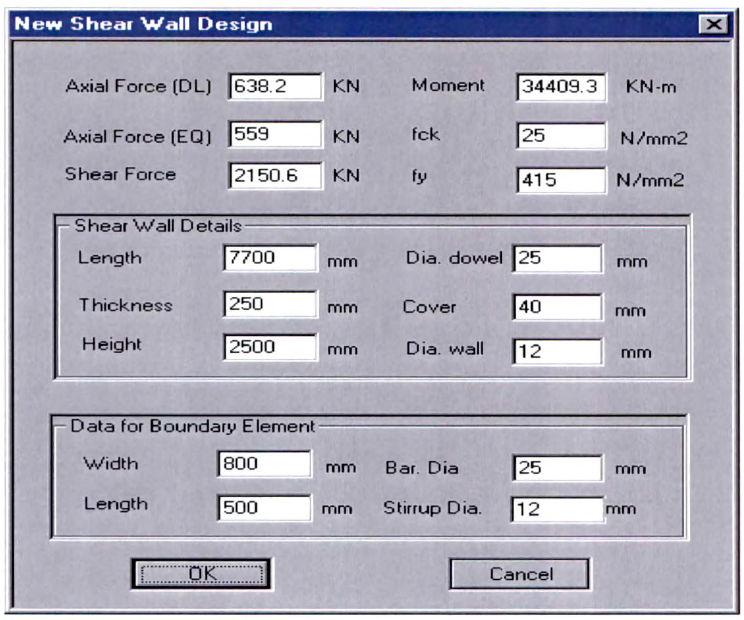



**Fig. 5.24 Input Dialog Box for Shear Wall Design**

### 5.5.7 Evaluation of Existing Elements

#### 5.5.7.1 Beam evaluation module

This module uses the input data taken through the dialog box in **Fig. 5.25** Using this data, module calculates the moment of resistance in hogging and in sagging for the given beam as per the design aid SP:16. It then compares the M.R. with the expected demand moment and decides the beam deficiency for flexure. Finally the module evaluates for shear capacity in the same manner.

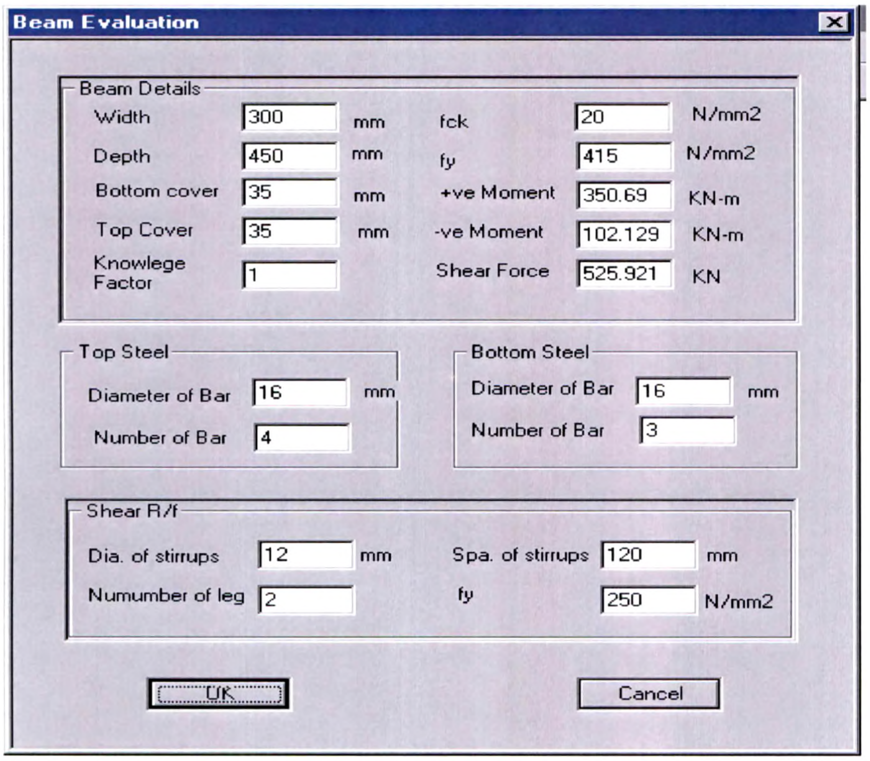



**Fig. 5.25 Input Dialog Box for Beam Evaluation**

### 5.5.7.2 Column evaluation module

This module uses the input data (**Fig. 5.26**) to evaluate the existing column. Based on this data module will evaluate whether given column is axial, uniaxial or biaxial, considering minimum eccentricity, slenderness ratio and given loading.

### 5.5.7.3 Bracing evaluation

During execution of bracing evaluation (**Fig. 5.27**) this module also calls two other modules to get the details of connection and section to be used for bracing. This module finds out tensile and compressive resistance of bracing and compares it with the expected demand forces. The process of evaluation of bracing is the same as that for new bracing excluding the iterative process of increasing the section when it fails to resist the expected demand. If the demand is higher than the capacity, bracing is deficient to resist given loading and needs to be replaced

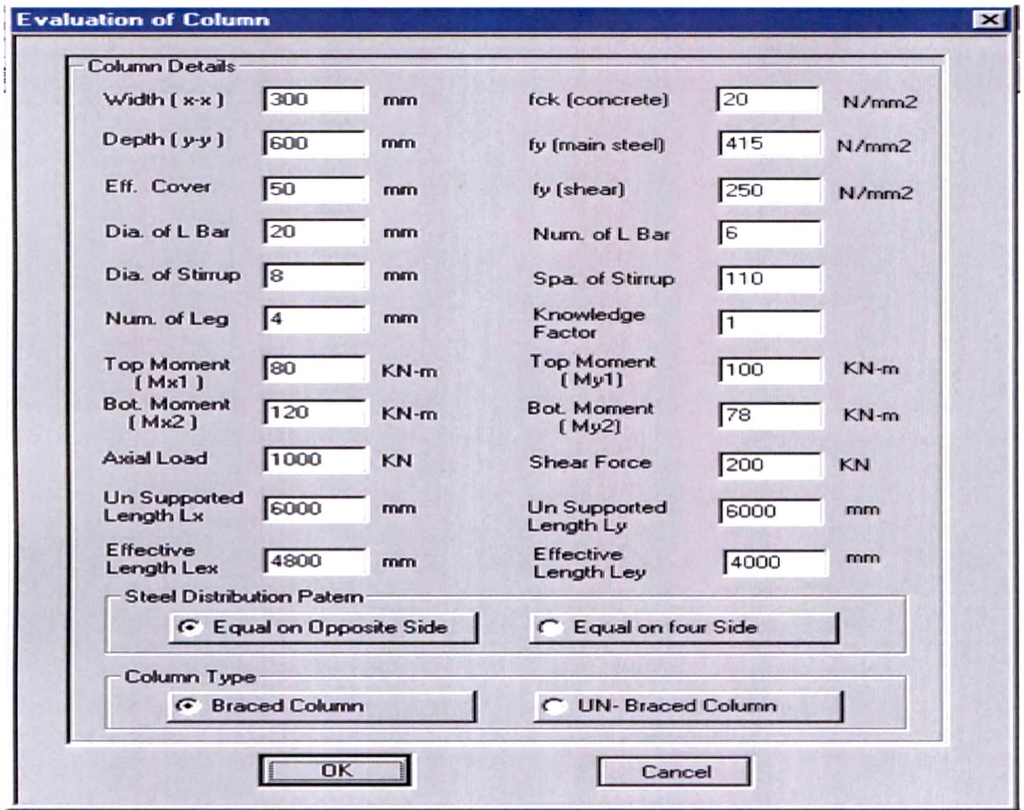



**Fig. 5.26 Input Dialog Box for Column Evaluation**

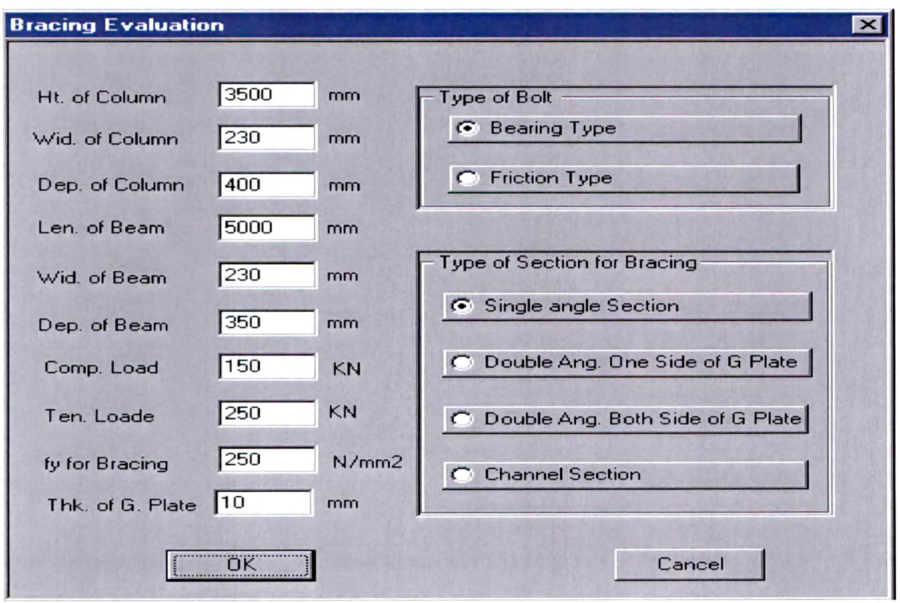



**Fig. 5.27 Input Dialog Box for Bracing Evaluation**

### 5.5.7.4 Shear wall evaluation

This module evaluates the flexural and shear capacity of shear wall (Fig. 5.28) and compares it with the expected demand. It also executes all checks with reference to IS: 13920 like spacing of bar, requirement of min/max amount of steel, requirement of boundary element, thickness of wall. If the shear wall is found deficient there is a need to retrofit in order to enhance the capacity of wall. After evaluating the shear wall, the module sends the evaluated data to the strengthening module of shear wall. It also generates the output file which explains the comprehensive detail of shear wall evaluation procedure.

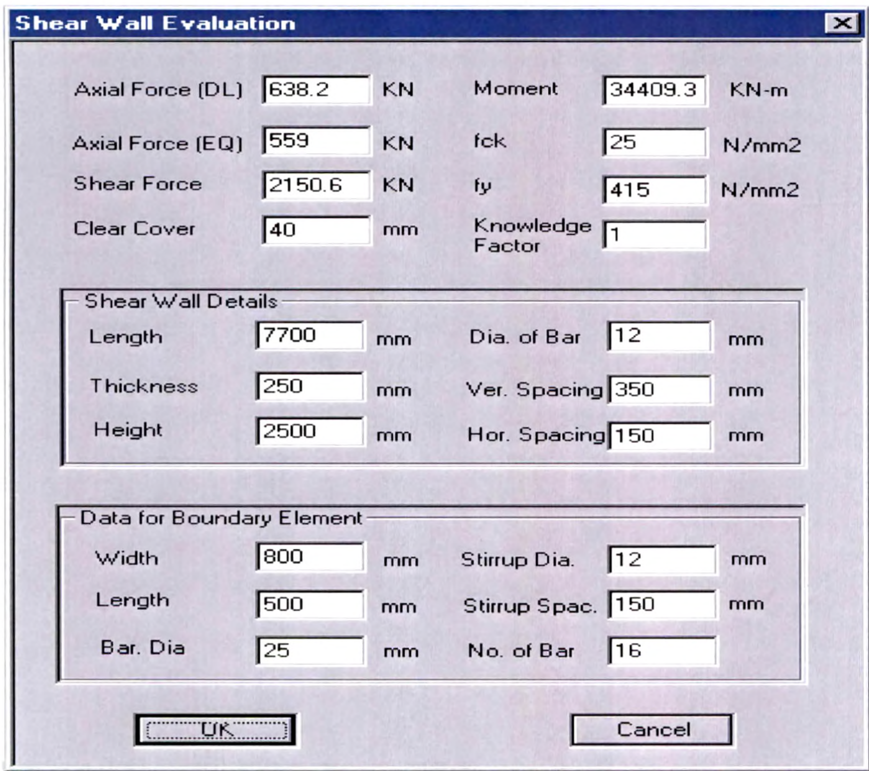



**Fig. 5.28 Input Dialog Box for Shear Wall Evaluation**

## 5.5.8 Strengthening of Existing Elements

### 5.5.8.1 Beam concrete jacketing module

Here, in addition to the dialog box the input data is also taken from the input file prepared by the beam evaluation module. Various additional data required for beam strengthening by concrete jacketing are shown in **Fig.5.29**. If the evaluated beam is found deficient than this module is execute otherwise it sends the message "Strengthening of existing beam is not required".

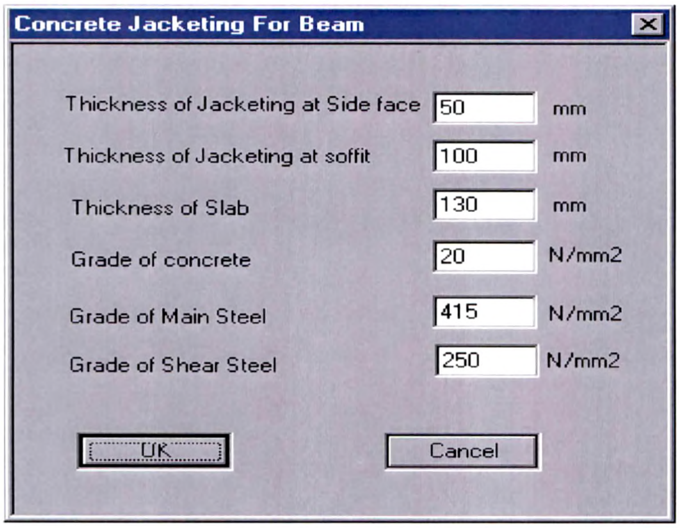



**Fig. 5.29 Concrete Jacketing for Beam**

### 5.5.8.2 Beam steel jacketing module

Here, in addition to the dialog box the input data is also taken from the input file prepared by the beam evaluation module. Various additional data required for beam strengthening with steel jacketing are shown in Fig. **5.30.** Module gives three options for location of plate from which user can choose the appropriate location with respect to various constraints of execution of work. If the evaluated beam is found deficient than this module is executed otherwise a message stating strengthening not required, is sent. Theoretical background for analysis is given in output file as shown in **Fig. 5.31.**

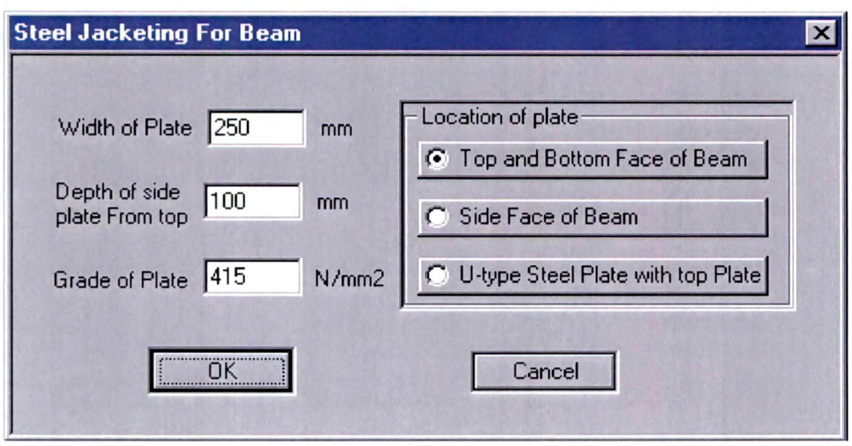



**Fig. 5.30 Steel jacketing for beam**

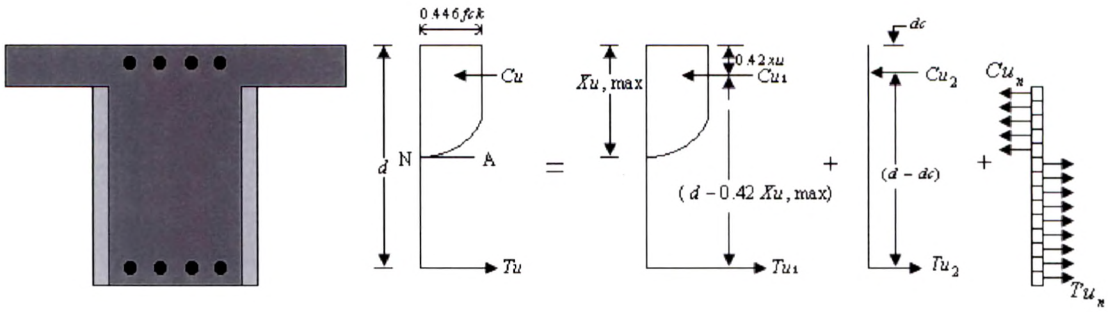



**Fig. 5.31 Analysis of Steel jacketed beam**

### 5.5.8.3 Column concrete jacketing module

Here too, in addition to the dialog box data is taken from the input file prepared by the column evaluation module. Various additional data required for column strengthening by concrete jacketing are shown in Fig. **5.32**. If the evaluated column is found deficient than this module is executed otherwise it sends the message that strengthening of existing column is not required. Based on this data, the module calculates the additional area of longitudinal reinforcement along with size and section of column. It also determines spacing and diameter of ties based on IS 456, IS 13920 and number of bent down bars required for transferring the axial load from old to new concrete based on draft code. The analysis of strengthened column is performed using interaction curves. The analysis assumes that there is perfect bond between old and new concrete. Minimum specifications of draft code for column concrete jacketing are also checked by the module.

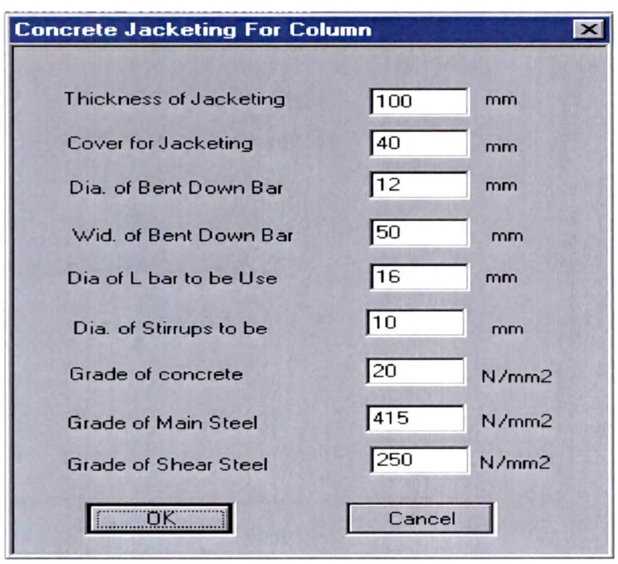



**Fig. 5.32Concrete Jacketing for Column**

### 5.5.8.4 Column steel jacketing module

Steel jacketing is done to the column for enhancing the flexural and shear strength of column. Module uses the data of column evaluation module. For analysis the steel jacketed column module determines the P-M interaction chart for an equivalent section assuming there is perfect bond between steel plate and old concrete section. The equilibrium, compatibility and the constitutive relationship have to be satisfied. Finally module finds the thickness of plate required for expected demand forces.

### 5.5.8.5 Shear wall strengthening

This module gives additional reinforcement required to strengthen existing shear wall by addition of concrete section (**Fig. 5.33**). Analysis is done as per IS 13920 assuming that there is perfect bond between old and new concrete. Module also finds out the shear transfer reinforcement perpendicular to shear plane as per draft code.

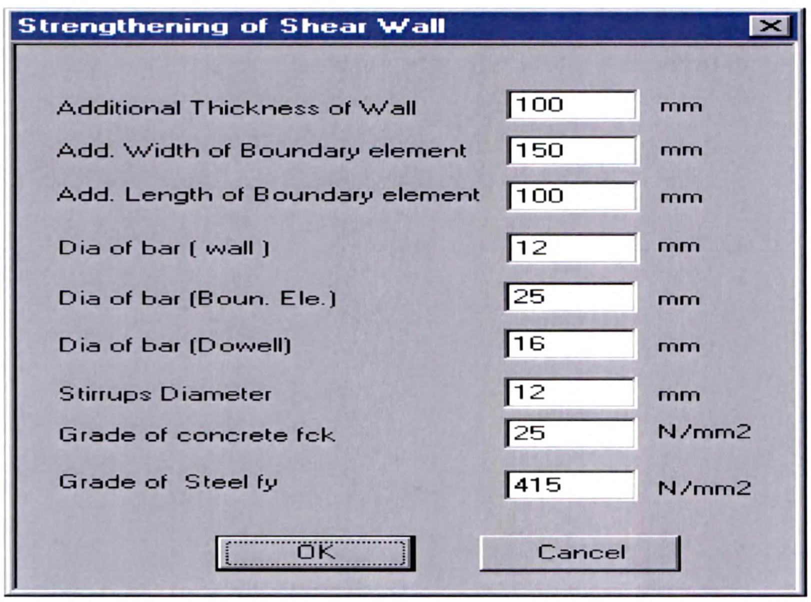



**Fig. 5.33 Strengthening of Shear Wall**

## 5.6 IMPLEMENATION IN VIRTUAL REALITY

### 5.6.1 Introduction

Virtual Reality aspects of damage and retrofitting in 3DS MAX environment are described in this section. As a first step geometry is created from excel file which is output of the SAP2000 software. MAXScript has been extensively used for this purpose. Deformations are also attached with the geometry from the same excel output file. Design of Beam, column, slab and stair is carried out and detailed results are employed to show typical beam, column, beam-column joint, slab and stair details. Observations of damage in these structural components are illustrated using a medium so far unexplored in the field of structural engineering. Finally traditional methods of retrofitting for beam, column and beam-column junction are also captured in virtual reality mode.

### 5.6.2 Development Strategy

The program for applications in Virtual Reality has been developed in MAXScript environment on MAX Studio platform which is very well suited on the windows operating system. The program has the following three modules which are discussed in the same order:

(a) Creating Geometry in MAX Environment,
(b) Viewing Displacements for the Chosen Load Case,
(c) Visualizing Typical Member Detailing, Damage and Retrofit Methods

### 5.6.3 Creating Geometry in MAX Environment

While working in 3D environment, user has to keep in mind at least 6 parameters of any object which incorporate its length, width, height, X-position, Y-position and Z-position. Scripting is an easy way to carry out simple object creation and manipulations as well as to do repetitive jobs but it is not very flexible for micro level detailing and yet it is a very powerful tool for programmers. SAP has inbuilt ability to export any desired input and output data in the obligatory sorted order in separate excel sheets. With the use of ActiveX controls in MAXScript, data can be read and manipulated easily from other applications.

With the use of ActiveX controls, program is prepared in MAXScript to create the geometry and attach displacements from SAP output files. General layout of the program is shown in **Fig. 5.34**. All data required for creating geometry like member number, its XYZ position, j-end, k-end, member length, width, height etc. are obtained from input excel files. The program is built in such a way that it will show the progress of the present activity through progress bar (**Fig. 5.34)** and will keep on messaging for required input from user. By proper interpretation of the stored data from relevant variables and arrays, complete geometry is built up in 3D environment as shown in **Fig. 5.35**. Later Standard concrete material is assigned to all the members

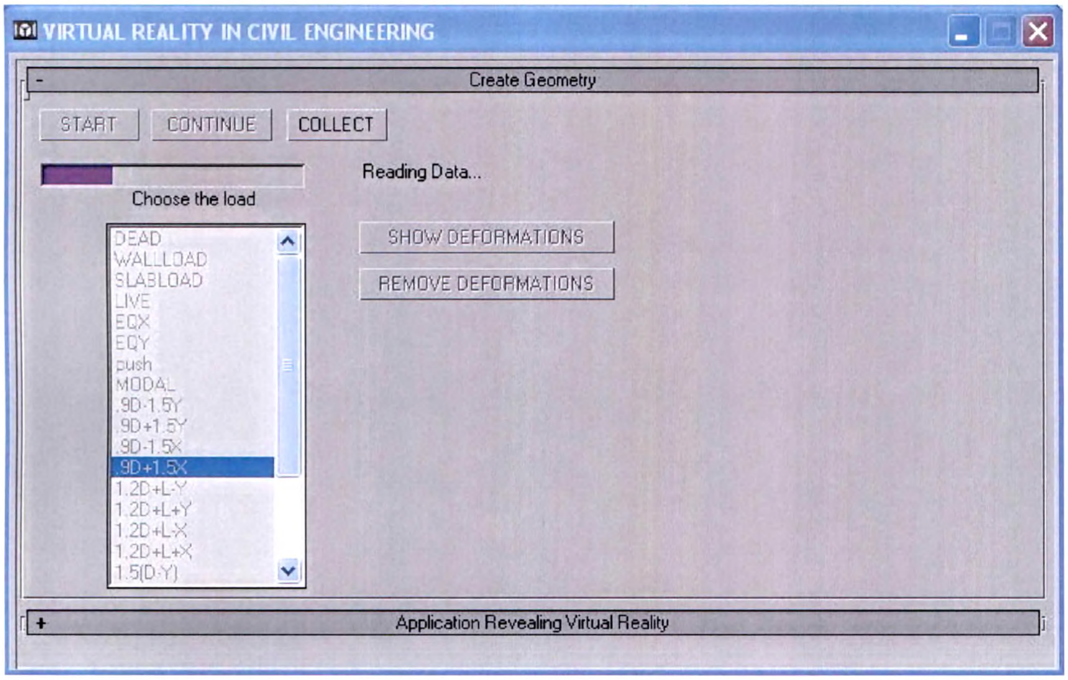



**Fig. 5.34 Program Layout for Geometry in 3DS MAX**

### .Viewing Displacements for the Chosen Load Case

After creating geometry, program will list all the load cases and combinations in the provided list box (**Fig. 5.34**). The list box dialog will be activated after receiving the message of "Now Select any Load Case". The program will then accumulate the member deformations at defined intervals for the specified load case in selected set of arrays from the excel sheets and on pressing "Attach" button, it will attach all the deformations to their respective locations i.e. at specific vertices of the members. Also it will play the animation showing the deformations in an actual 3D geometry which most of the analysis software show as line diagram only. User can again view deformations for any of the load case by removing the attached displacements, reselecting the load case and attaching the deformations for the new load case yet again.

**Fig. 5.35 Built up Geometry from MAXScript**

### 5.6.4 Visualizing Typical Member Detailing, Damage and Retrofit

After viewing the member displacements in the first program, user can move to the second program named “3DS MAX Application”, where the detailing, damage or retrofit of any of the items enlisted can be visualized by selecting desired option button, as shown in **Fig. 5.36**. Design Details section in this program includes details of beam, column, slab, beam-column junction and stair; while Damage Patterns section consists of beam shear and flexure failure, column buckling, column shear failure, damage to stair cases and a general view of the

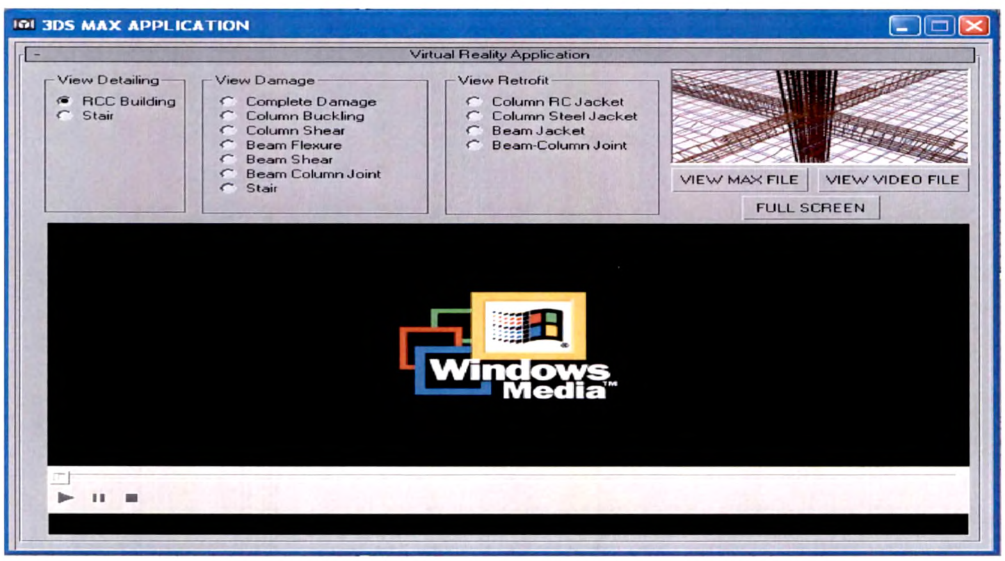



**Fig. 5.36 Visualization Options and Windows Media Player in 3DS MAX**

damage to entire building frame. Section for Retrofit Methods takes into account beam, column, beam-column joint and slab retrofit methods.

With the use of ActiveX Controls in MAXScript, Windows Media Player is brought into the application (**Fig. 5.36**), which will play the animation movie for the above selected case. Here program also shows the original MAX file from its interface itself. Various cameras are used to capture typical detailing, damage or retrofitting sequence in different MAX files. On choosing any of the above cases, the selected files are opened with certain camera view port and its animation sequence.

## 5.7 IS:456-2000 AND IS:13920-1993 DETAILING CRITERIA

Various structural members are designed and followed by their 3D details. The general requirements of the member for ductility and special confining reinforcement follow IS: 13920-1993 [86] and longitudinal and shear reinforcement is designed according to IS: 456-2000. The detailing for beam and column has been incorporated in 3DS MAX view (**Fig. 5.37 to 5.39**) from the outcome provided at the end of design. For the beam and column member design, the SAP analysis results have been used while slab and stair case are designed for the proportionate dimensions (**Fig. 5.40 to 5.42**) from the building taken into consideration.

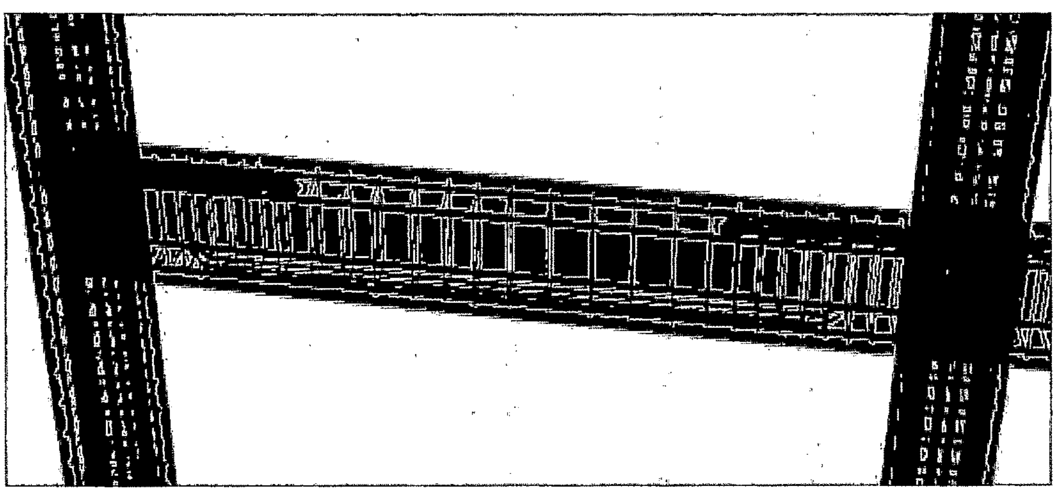


**Fig. 5.37 View of 3D Beam Detailing**

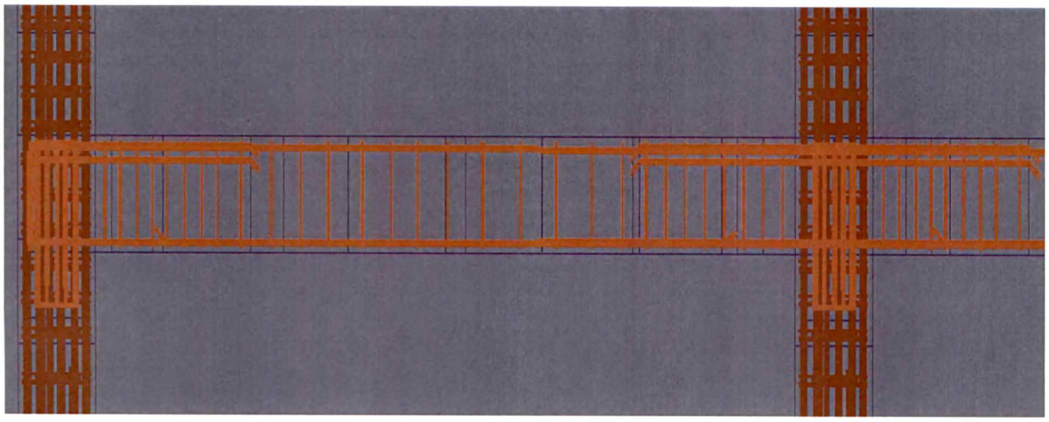


**Fig. 5.38 Beam Sectional Detailing at Continuous Edge**

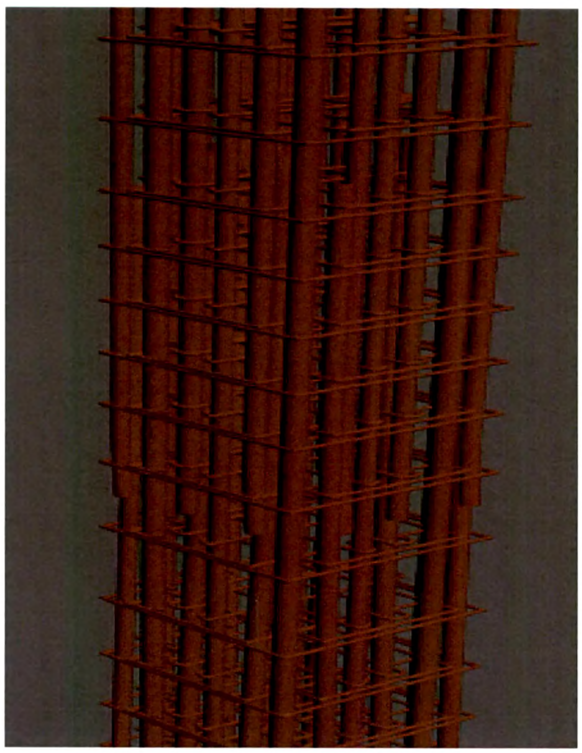


**Fig.5.39 Alternate Curtailment of Bars at Mid-height of Column**

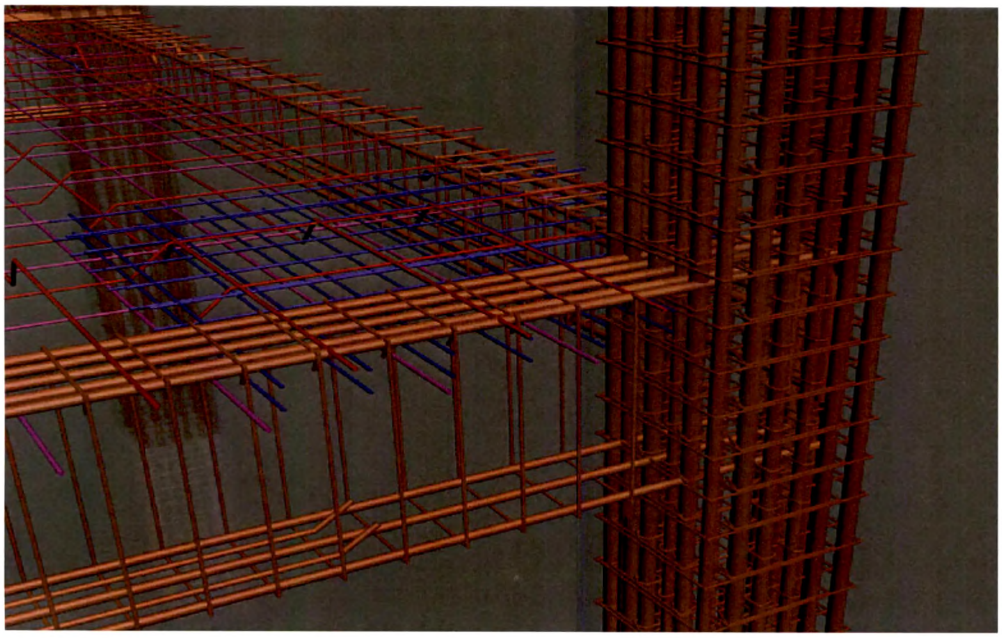


**Fig. 5.40 Torsion Reinforcement in Slab–Beam Corner Element**

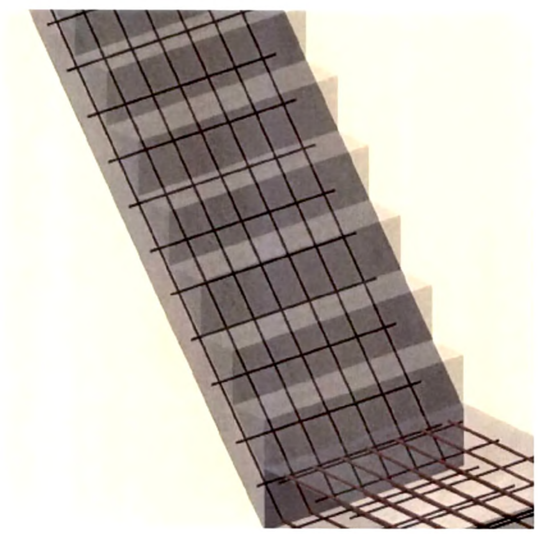


**Fig. 5.41 Details of the Waist Slab**

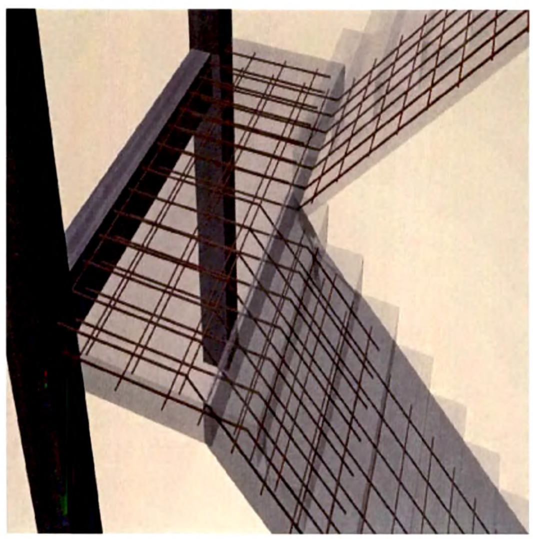


**Fig. 5.42 Details at the Landing**

### 5.7.1 Beam-Column Joint Detailing

When subjected to earthquake forces, the beams adjoining a joint are subjected to moments in the same direction. Under these moments, the top bars in the beam-column joint are pulled in one direction and the bottom ones in the opposite direction. These forces are balanced by bond stress developed between concrete and steel in the joint region. If the column is not wide enough or if the strength of concrete in the joint is low, there is insufficient grip of concrete on the steel bars. In such circumstances, the bar slips inside the joint region, and beams lose their capacity to carry load. Practically joints are the weakest links of the structure. Distress of the joint region is the most frequent cause of failure rather than failure of individual elements. Prime aim of detailing the joint is that it must not yield before the joining members reach their ultimate capacity. Special confining reinforcement as required at the end of column shall be provided through the joint as specified in the ductility code (**Fig. 5.43 & 5.44**).

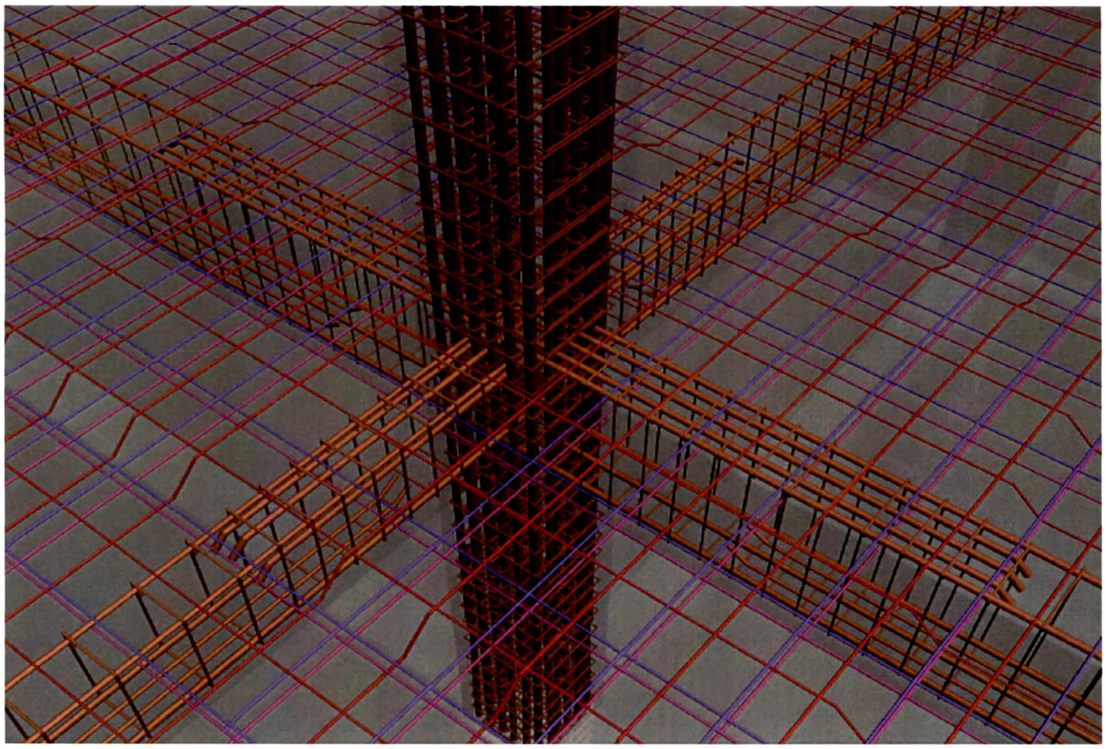


**Fig. 5.43 Detailing of Internal Beam-Column Joint**

## 5.8 DAMAGE OF ELEMENTS IN THE VIRTUAL WORLD

A strong earthquake puts the whole structure through a tough test. As a result, all the weaknesses of the structure, due to either imperfections in analysis, design errors, or even bad construction, become readily apparent. It is difficult to classify the damage caused by an earthquake, and even more difficult to relate it in a quantitative manner to the cause of the damage. This is due to the dynamic character of the seismic action and the inelastic response of the structure while damage is being induced.

**Fig. 5.44 Confining Reinforcement in the Column Joint Region**

### 5.8.1 Inelastic Behavior of Beams

Damage to beams, although, does not jeopardize the safety of the structure, is the most common type of damage in R.C buildings. Damage in the beams may be in the form of micro cracks in tension zone, shear failure of beams near supports, flexural cracks on the upper and lower face of the beam at supports and flexural or shear failure at the junction of secondary beams. There is little evidence that a building would collapse due to beam failure. This may be attributed to redistribution of moments and the inherent redundancy in beams.

### 5.8.2 Micro-cracking in Beams

As the beam sags under increased loading, it can fail in two possible ways. If relatively more steel is present on the tension face, concrete crushes in compression causing brittle failure. If relatively less steel is present on the tension face, the steel yields first and redistribution occurs in the beam until eventually the concrete crushes in compression causing a ductile failure and hence is desirable. The ductile failure is characterized with many vertical cracks starting from the stretched beam face and going towards its mid-depth. Micro cracking in beams is most the common damage. Micro-cracks are always present due to bending of the tension zone of the beam and they widen due to vertical component of earthquake. These cracks (**Fig. 5.55-5.56**) are orthogonal to beam axis along the tension zone of the span. Generally they do not jeopardize the safety of the structure.

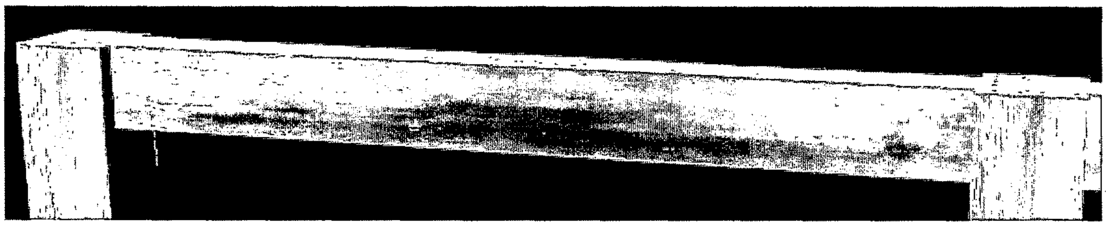


**Fig. 5.55 Beam before Micro-cracks**

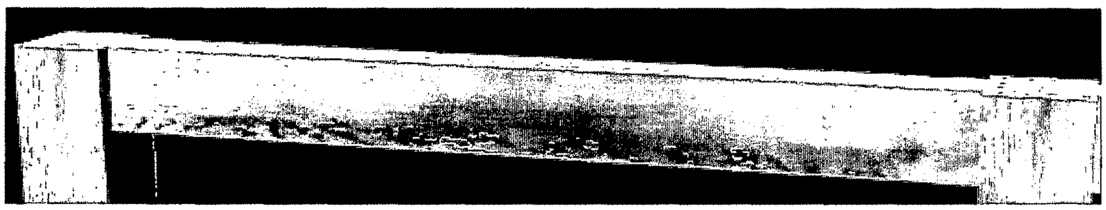


**Fig. 5.56 Beam after Micro-cracks in Bending Zone**

### 5.8.3 Beam Flexural Failure

This failure is less common than shear type failure. Flexural cracks may immerge in the vertical direction on the upper and lower face of the beam near supports. Cracking at lower face occurs due to bad anchorage of the bottom reinforcement into the support, which results in inadequate resistance to lateral forces during seismic excitations near supports, in that case one or two wide cracks form close to the support (**Fig. 5.57**).

### 5.8.4 Beam Shear Failure

A beam may fail due to bending-shearing action. A shear crack is inclined at 45° to the horizontal; it develops at mid-depth near the support and grows towards the top and bottom faces. Shear damage occurs when the area of the stirrups near support region is insufficient. Shear failure is brittle in character and second most frequent type of damage (**Fig. 5.58-5.59**) which may sometimes jeopardize the overall stability of the structure. Closed loop stirrups are provided to avoid such shearing action near the support.

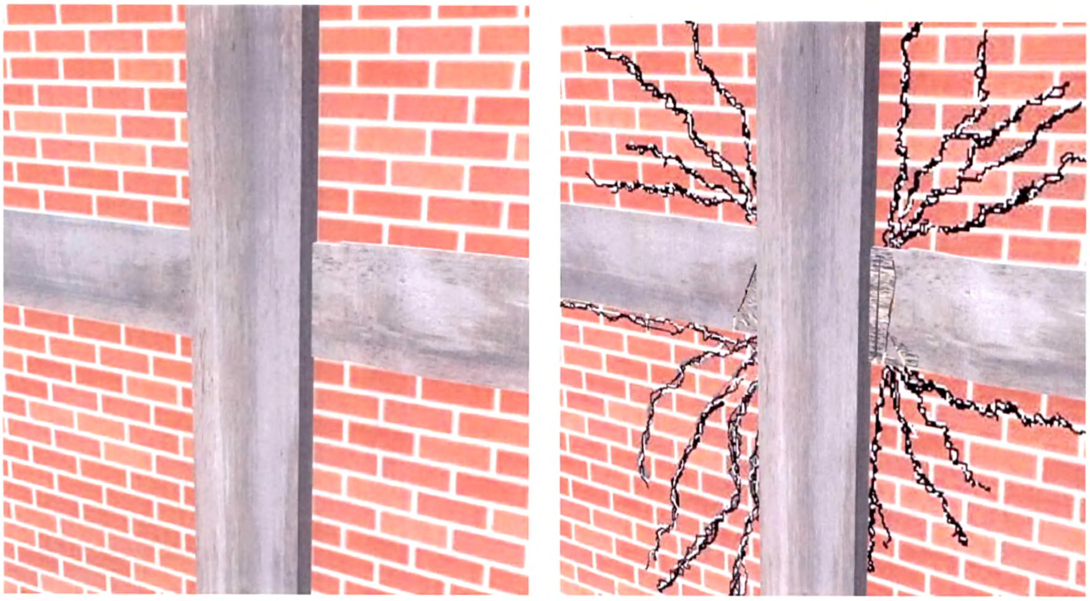


**Fig.5.57 Beam Before and After Flexural Failure at Continuous Face**

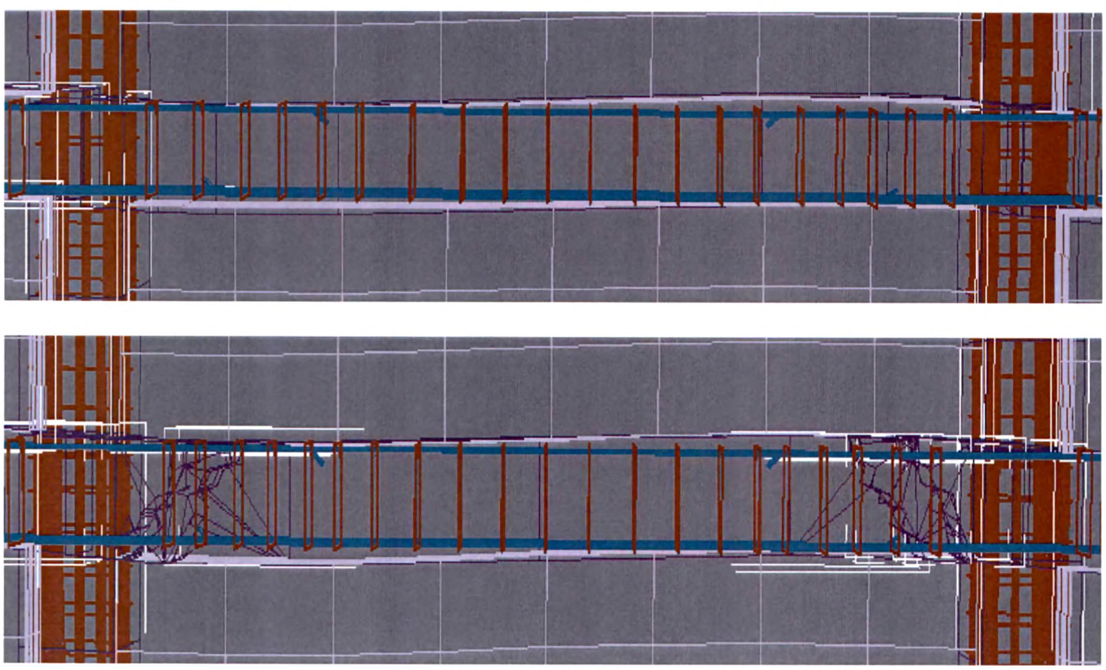


**Fig. 5.58 Beam Before and After Shear Failure in MAX**

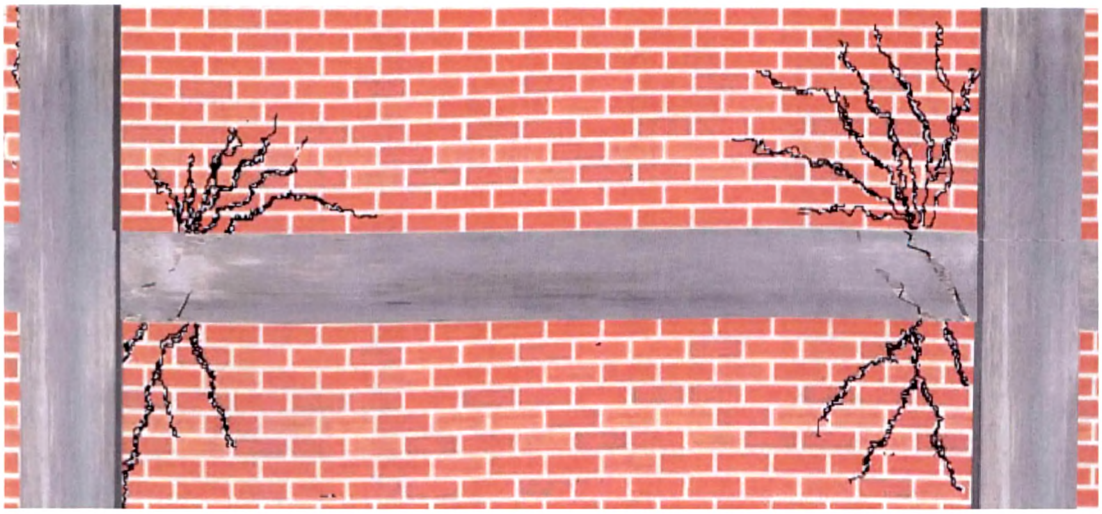


**Fig. 5.59 Rendered View of Beam Shear**

### 5.8.5 Inelastic Behavior of Columns

Failure of columns before failure of beams can lead to a storey mechanism and catastrophic failure. This may cause large displacements leading to instability of the whole structure. Hence column damage is to be given more importance than beam damage. Columns damage mainly due to lack of confinement, large tie spacing, insufficient development length, inadequate splicing of all column bars at the same section, hook configurations of reinforcement etc.

### 5.8.6 Column Shear Failure

Shear failure mostly occurs in the constrained short columns in which only a short length is effectively free and due to their larger stiffness they attract large forces. Columns with moderate to small slenderness ratios, i.e. $L / 2h < 3.5$ show this type of damage. Column mainly damages in the form of X-shaped cracks in the weakest zone due to high shear and low bending moment under strong axial compression. So if it is not adequately designed for such large forces, it can suffer significant damage (**Fig.5.60-5.61**). The ultimate form of this type of damage is the explosive cleavage failure of short columns, which usually leads to a spectacular collapse of the building. Its frequency compared to column buckling is low; however, shear damage is brittle and must be avoided in columns by providing transverse ties at close spacing in the compression zone.

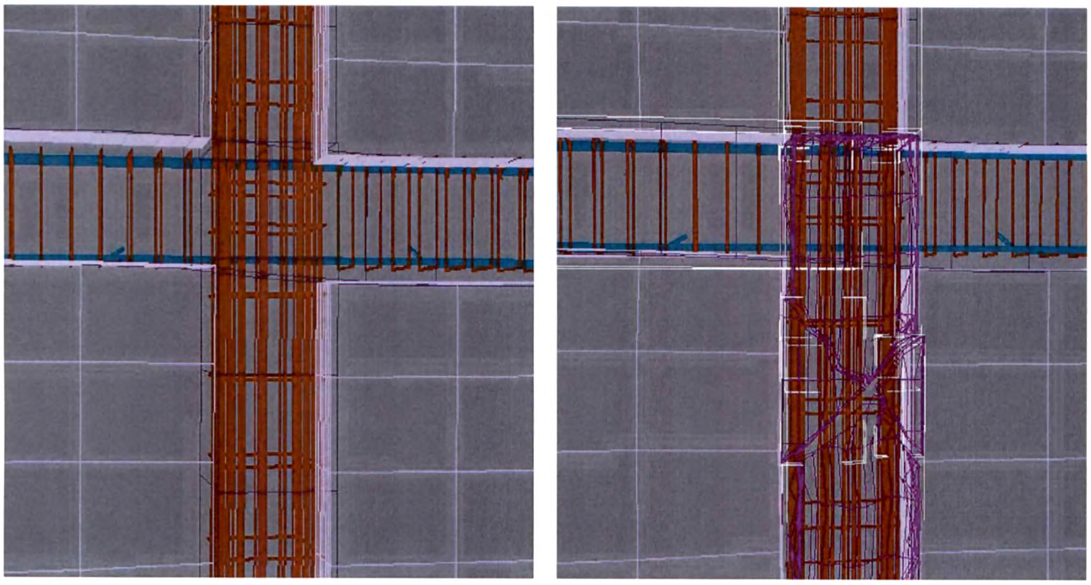


**Fig. 5.60 Column Before and After Shear Failure in MAX**

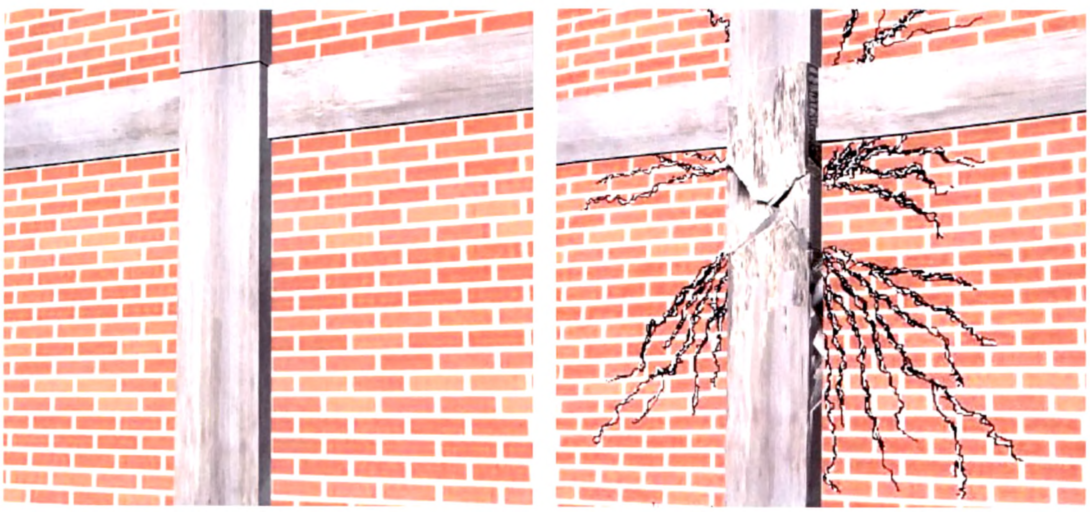


**Fig. 5.61 Rendered View of Column Before and After Shear**

### 5.8.7 Column Buckling

Column buckling in soft storey structures is a matter of growing concern. Damage occurs at the top and bottom of the column due to high bending moment and low shear under strong axial compression. This type of failure is also called axial-flexural or combined compression-bending failure. It occurs in columns of moderate to high slenderness ratio, i.e. L / 2h > 3.5. This type of damage is characterized by crushing of compression zone and manifested by spalling of concrete cover to the reinforcement successively on both faces of the column and then concrete core expands and crushes.

This phenomenon is usually accompanied by buckling of bars in compression and by hoop fracture (**Fig. 5.62**). The fracture of ties and the disintegration of concrete lead to shortening of the column under the action of the axial force. The column thus not only loses its stiffness, it also loses its ability to carry vertical loads. Some major reasons for this type of failure may be due to low quality of concrete, inadequate ties in critical areas, presence of strong beams leading to columns failing first, and finally, the strong seismic excitation inducing many loading cycles in the inelastic range.

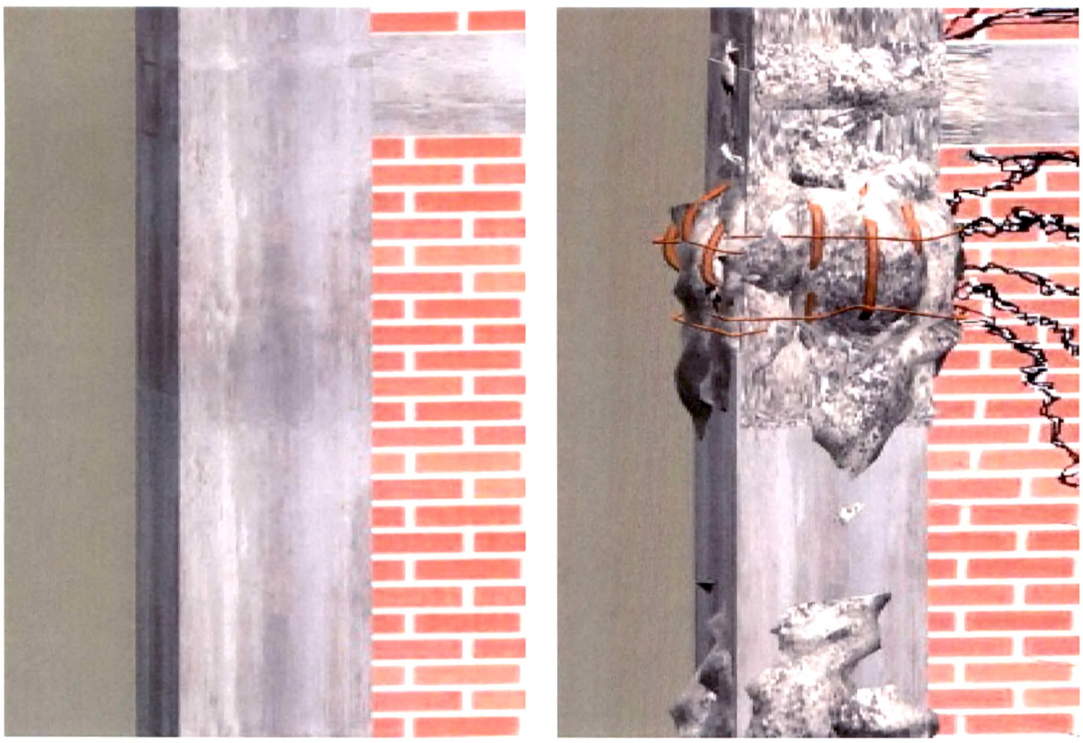


**Fig. 5.62 Rendered View of Column Before and After Buckling**

### 5.8.8 Damage to Beam-Column Joints

Beam-column joints are often the weakest link in a structural system due to lack of adequate anchorage of bars extending into the joints from the columns and beams. The joints should be so detailed that they do not fail before capacity of individual members framing in is exhausted. Distress in the joint region (**Fig. 5.63-5.64**) is the most frequent cause of failure rather than the failure of the connected elements. Damage in beam-column joints, even at the early stages of cracking, must be considered extremely dangerous for the structure and be treated accordingly. Damage of this type reduces the stiffness of the structural element and leads to uncontrollable redistribution of load effects. Inadequate or absence of reinforcement in beam-column joints, absence of hoop reinforcement confinement or inappropriate location of bar splices in column are some of the common causes of failure of beam-column joints.

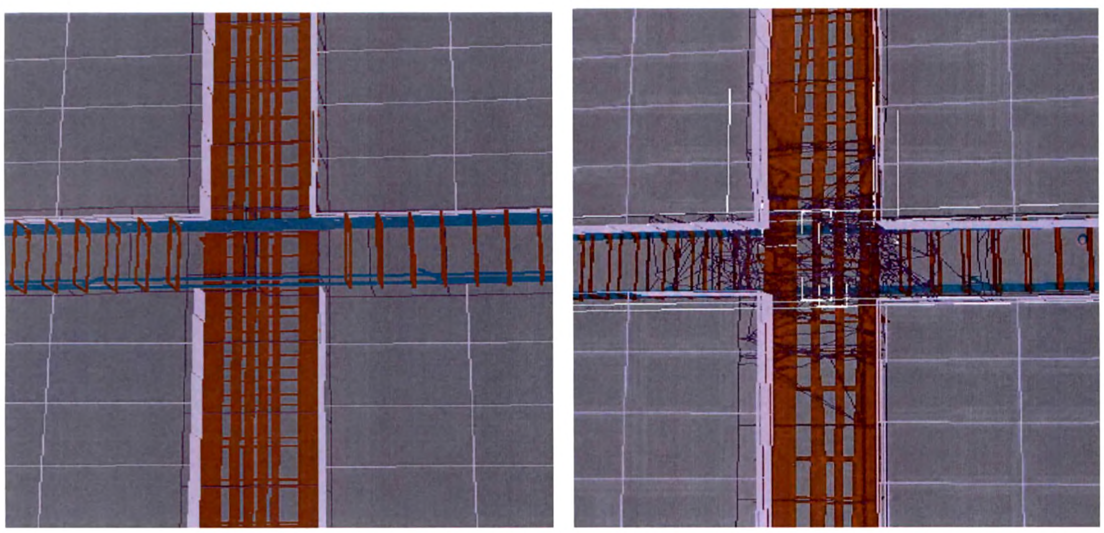


**Fig. 5.63 Beam-Column Joint Before and After Earthquake in MAX**

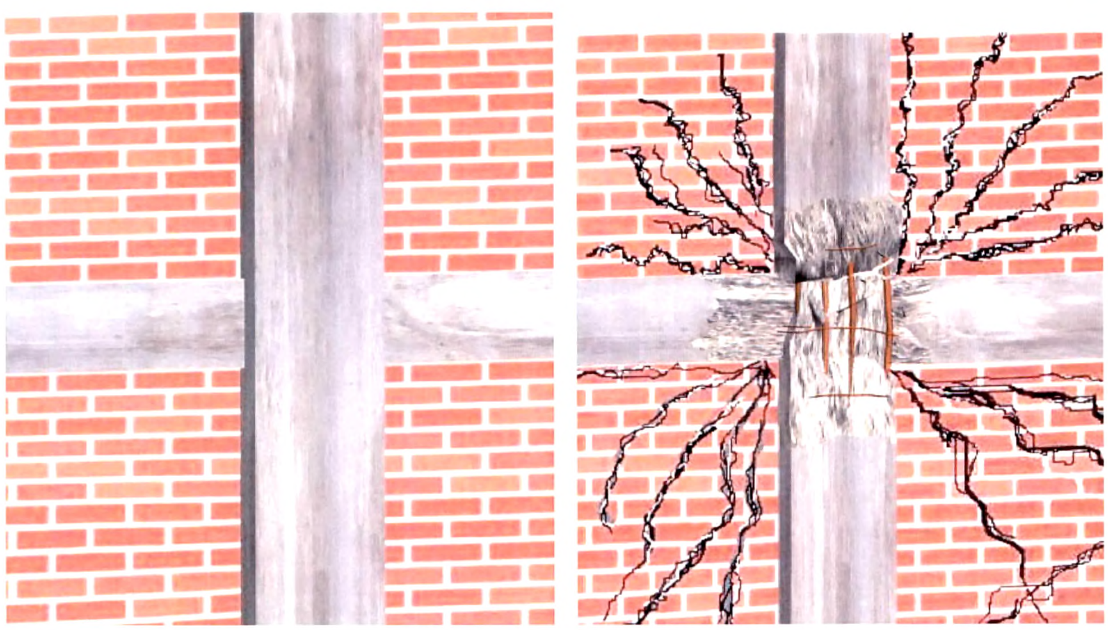


**Fig. 5.64 Rendered View of Beam-Column Joint Before and After Earthquake**

The sequence of failure includes spalling of concrete cover, formation of diagonal shear cracks, crushing of core concrete, buckling of reinforcing bars and finally settlement or inclination of floor takes place. Closely spaced stirrups in the form of confining reinforcement must be provided in the joint region and care must be taken to avoid congestion of the steel bars especially when the joint is confined on all four sides by connecting beams.

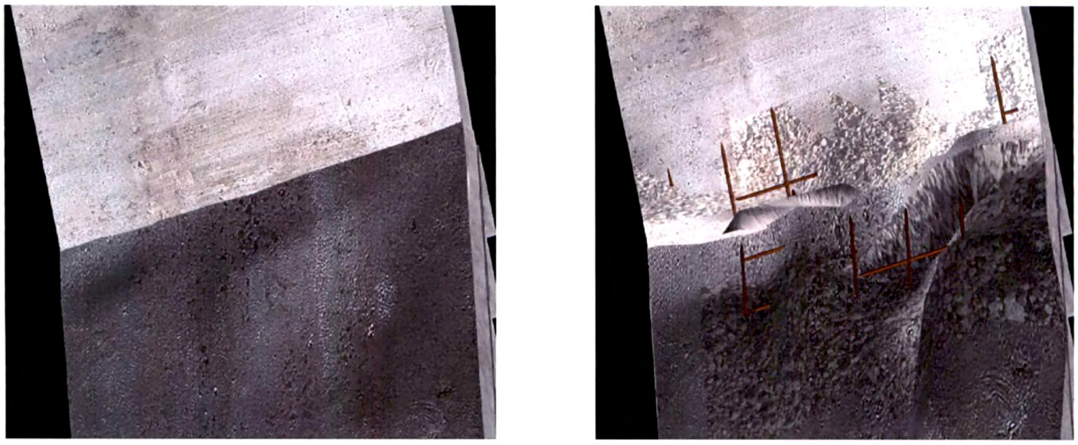
### 5.8.9 Damage to Staircase

Stair cases are important structural members which may be badly damaged at the landings if not properly connected at the joints (**Fig. 5.65-5.66**). Cracks parallel or transverse to the reinforcement may be seen at random locations for low damaged stairs. Spalling of concrete cover at the locations of floor discontinuities e.g.corners may be very common during earthquakes of moderate shaking. Complete collapse of the stair cases may also occur in areas of concentration of large seismic load effects, which is usually related to punching shear failure, aggravated by the cyclic bending caused by the earthquake and the thrust action from the inclined portion of the stair flights. This failure can be considered as one of the most dangerous for the stability of the structure.

**(a)**

**(b)**

**Fig. 5.65 Stair Mid Landing Level Before and After Earthquake**



(a) (b)

**Fig. 5.66 Stair Top Landing Level Before and After Earthquake**

### 5.8.10 Damage to Infill Panels

Almost all the walls are constructed with masonry, in contact with the surrounding structural members of the frame. As they are constructed with materials of lower strength and deformability than the structural members, they are the first to fail. First cracks appear on the plastering along the lines of contact of the masonry with the frame during the initial excitation as shown in **Fig. 5.67 (a)-(b).**

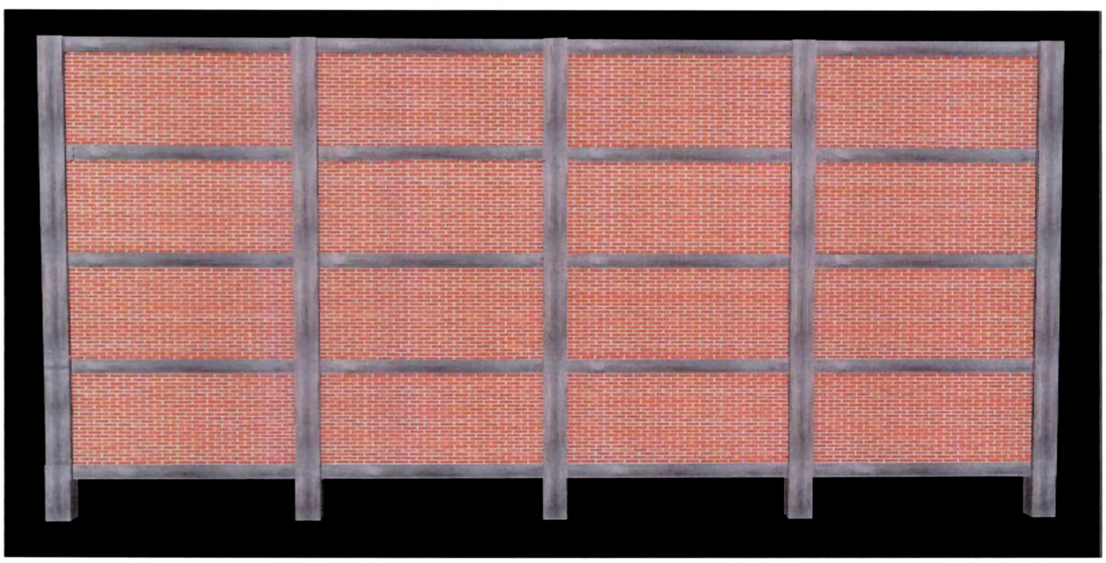


**Fig. 5.67 (a) Rendered View of Infill Walls Before Earthquake**

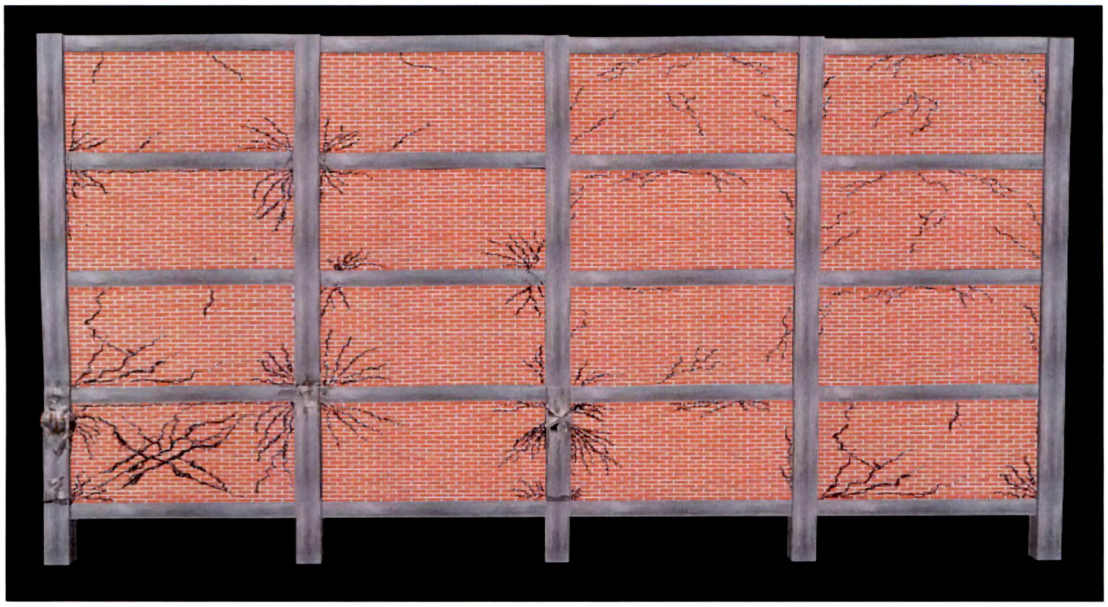


**Fig. 5.67 (b) Rendered View of Infill Walls After Earthquake**

As the deformation becomes larger, the cracks penetrate into the masonry, and this is manifested by the detachment of the masonry from the frame. Subsequently, X-shaped cracks appear, small at the beginning which become larger later, in the masonry itself, in a step-wise pattern following the joint lines.

## 5.9 LOCAL RETROFITTING OPTIONS IN VIRTUAL REALITY

Methods for retrofitting of beams and columns in light of the predominant failure patterns and different parameters that govern the choice of retrofit are presented here. Some of the aspects that are not well laid out in the codes and text books call for careful judgment from an experienced designer.

### 5.9.1 Beam Retrofitting

Deficiency in seismic behavior of beams may be in shear, flexure or joint shear. Enhancing flexural capacity of integral beams is difficult because of the constraints placed by the existing superstructure on the sides. To augment the positive moment capacity (which is usually not a critical issue except at the end supports under seismic loading). Additional bottom reinforcement may be provided by introducing additional beams on either side of the existing beam (**Fig. 5.68-5.71**).

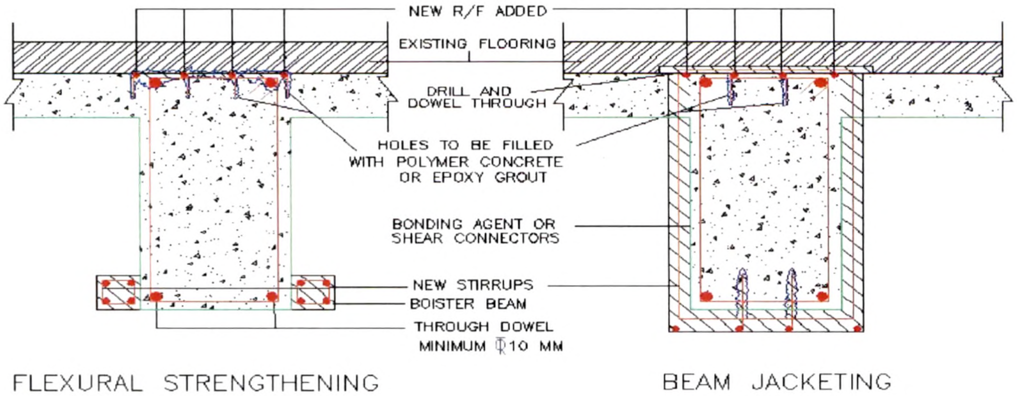



**Fig. 5.68 Alternative Beam Retrofit Methods**

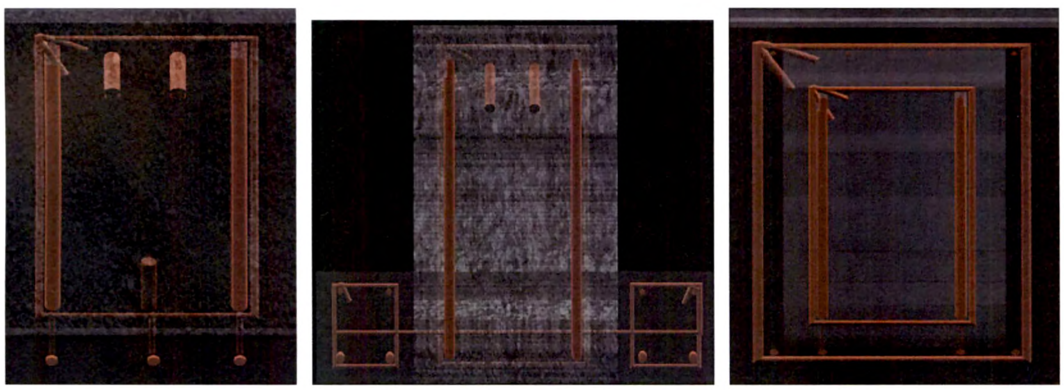


**Fig. 5.69 Rendered Side Views for Alternative Beam Retrofit**

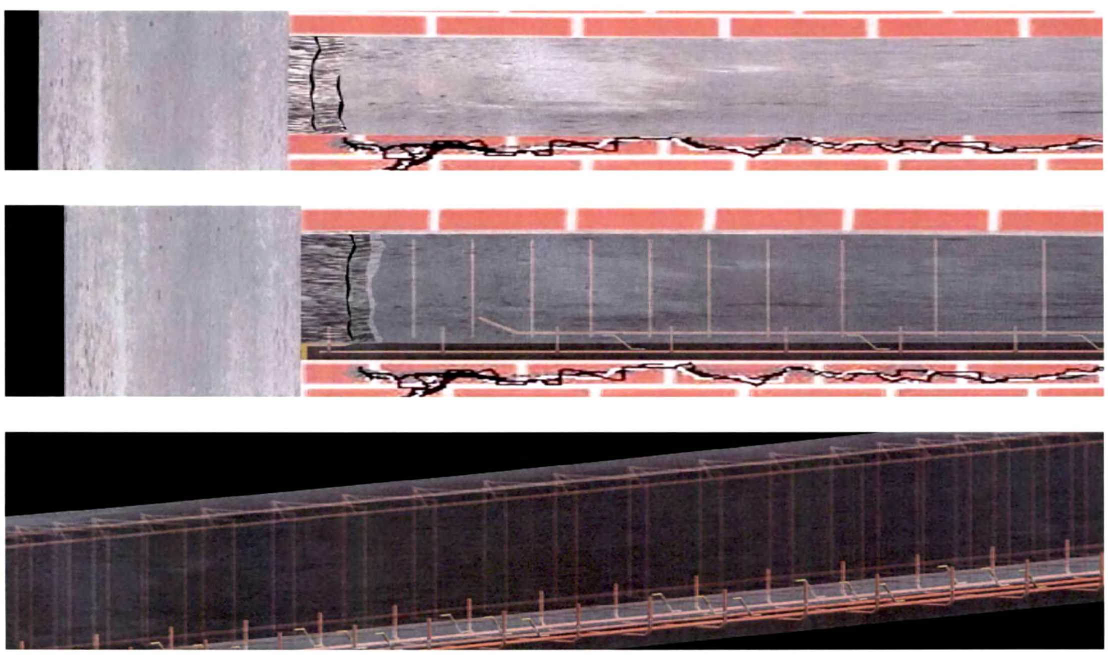


**Fig. 5.70 View of the Beam Added at Bottom**

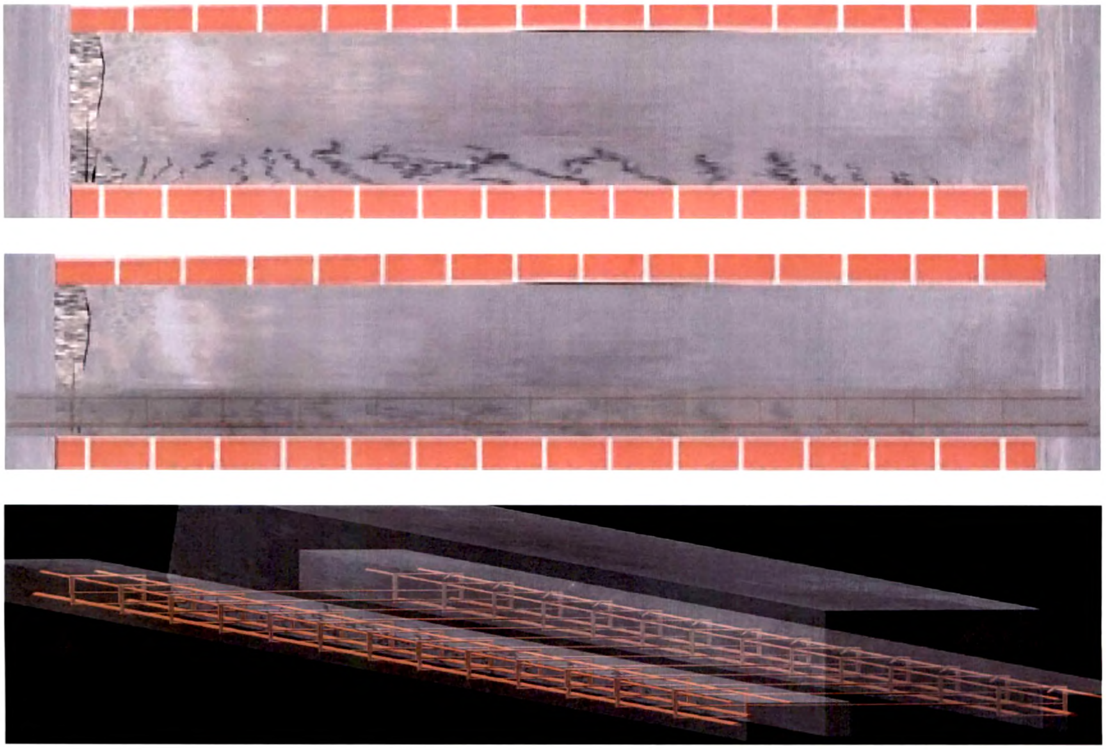


**Fig. 5.71 Addition of Bolster Beams on both Sides**

Alternatively by providing additional steel on the bottom (**Fig. 5,72**) and enhancing the depth of the beam by means of polymer concrete, resin concrete or micro concrete depending on the suitability for the beam under consideration. Negative moment capacity can be increased by removing top concrete and providing additional reinforcement. Most often, both moment and shear capacity need to be enhanced for ductile behavior and in such cases a beam jacket, on similar lines to column jacketing, complete with additional stirrups may be provided. This may require propping of the adjacent slabs.

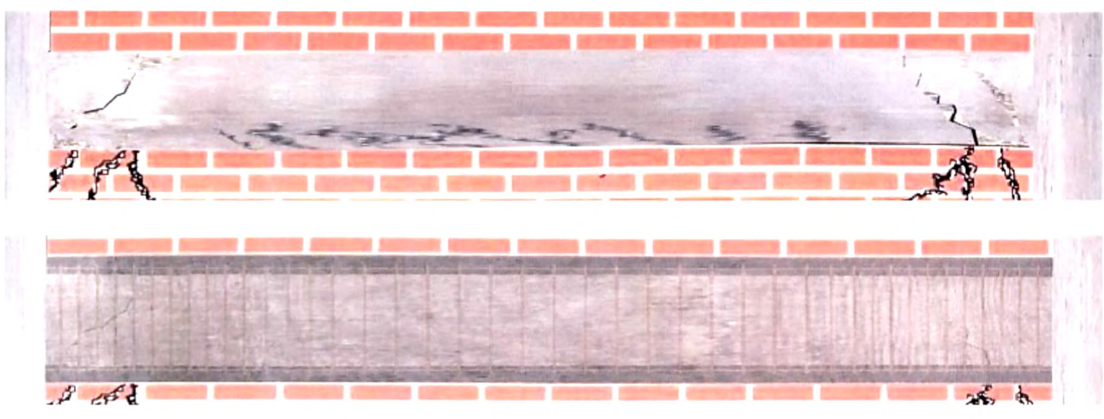


**Fig. 5.72 Beam Jacketing**

### 5.9.2 Column Strengthening

The philosophy behind seismic column strengthening is that the axial load strength of a column after concrete has spalled off should be at least equal to the axial load strength before spalling, plastic hinges should occur at the column ends and ductility must be ensured by confining the concrete with special confining steel. Depending on the degree of damage, desirable seismic resistance, type of connections and best optimum solution, different techniques may be applied, such as resin injections, removal and replacement or jacketing, steel profile cages or FRP encasement. Resin injections and resin mortars are applied only for the repair of columns with small cracks or peelings, without crushing of concrete or damage in the reinforcement. Removal and replacement of columns may be needed with high degree of damage which includes crushing of concrete, breaking of ties and buckling of longitudinal reinforcement. Good bonding between old and new concrete is absolutely necessary. It should be kept in mind that changes in the sectional area of the structural elements lead to a redistribution of stress due to resulting changes in the stiffness of the various structural elements.

### 5.9.3 Column Jacketing

Column jacketing is popularly used when the columns are deficient in strength and are seriously damaged. Depending on the existing local conditions, jackets are added on the perimeter of the column or sometimes on one or more sides. Where the jacket is limited to the storey height, an increase in the axial and shear strength of the column is achieved with no increase in flexural capacity at the joints. It involves addition of a comparatively thick layer of reinforced concrete in the form of a jacket around an existing column (**Fig. 5.73**). Adequate confinement of rectangular columns by a rectangular jacket requires extensive doweling to connect the jacket to the existing column (which also acts as lateral ties for the newly introduced column vertical reinforcement) and use of suitable bonding chemicals at the interface of the old and new concrete. In order to accomplish force transfer between old and new concrete, roughening of the surface of the old concrete is required, as well as welding of connecting bars to the existing and new reinforcement bars. This connection is necessary when the column has completely deteriorated or when its height is too large.

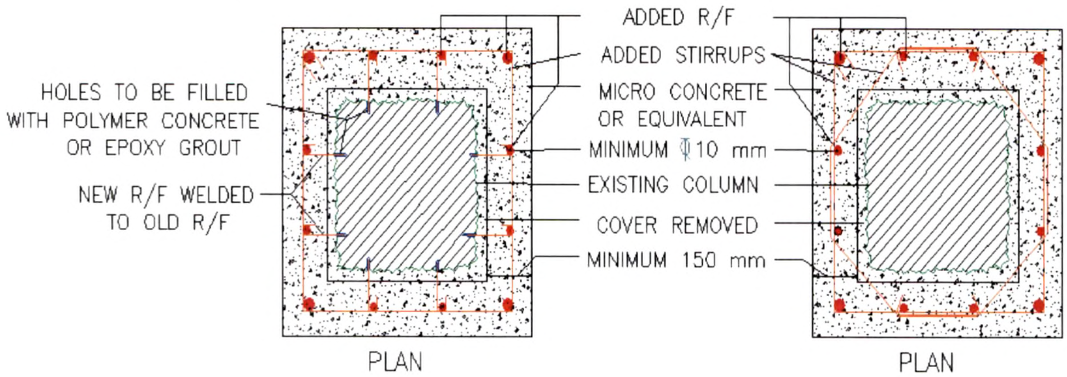



**Fig. 5.73 Alternatives of Column Retrofitting**

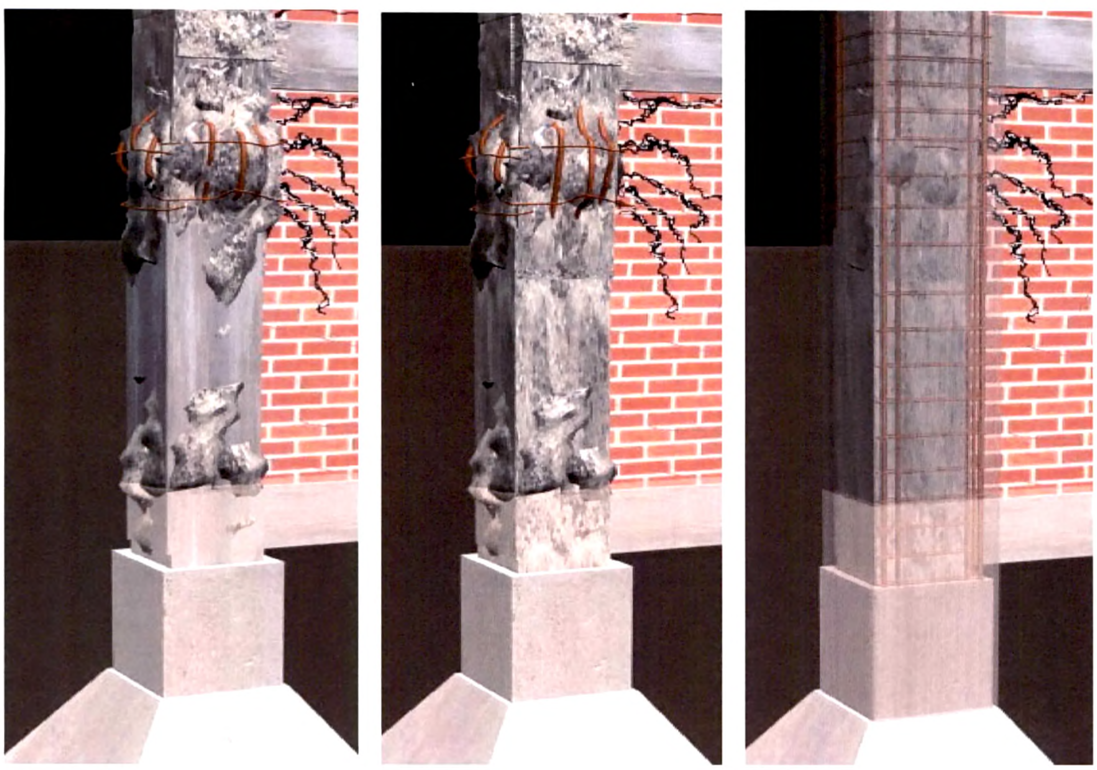


**Fig. 5.74 Damaged, Cover Removed and Retrofitted Column**

The jackets must protrude through the ceiling and the floor slabs of the storey where column repair is necessary. If this is not done, only local strength enhancement is achieved and the beam-column junctions may continue to remain vulnerable. The advantage of this retrofit method is that it enhances both ductility and overall flexural strength. Additional fire and corrosion protection are not required. It is also conducive to a holistic repair where even beam and joint retrofitting is required as compared to a steel jacket which may need additional fire protection and corrosion resistance. The disadvantage is that the column size is greatly enhanced.

### 5.9.4 Steel Jacketing

The technique of steel jacketing is suitable when column is adequate for gravity loads and requires only seismic enhancement. Additional ductility is provided through steel jacket but increase in overall flexural strength of the column is not achieved. This type of intervention offers the possibility of only a small increase in column size. Steel sheets with 4-6 mm thickness are welded together throughout their length and located at a distance from the existing column. The gap between the jacket and the column is grouted with a non-shrink cement grout, after flushing with water (**Fig. 5.75-5.76**). With the above interventions, retrofitted rectangular columns have performed exceptionally well locally in resisting axial and shear forces. Attempts to retrofit rectangular columns using reinforced concrete jackets are not as effective as steel jacketing but are used more frequently. Steel jackets have proved to be very satisfactory in inhibiting splice failures, providing flexural ductility, and enhancing shear strength. However, the flexural strength of the framed structure cannot be improved because it is impossible to pass the encasement through the floors and they offer insufficient fire resistance.

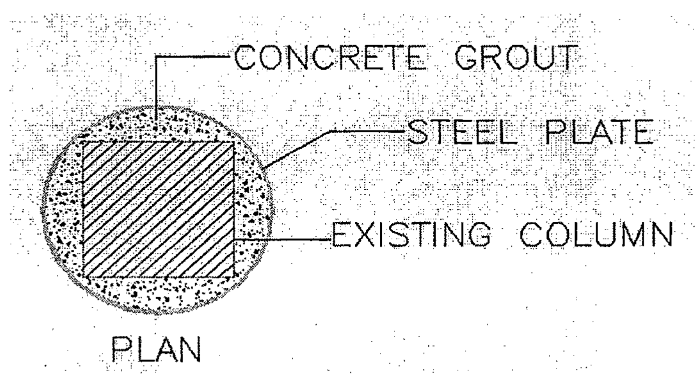



**Fig. 5.75 Steel Jacket Details**

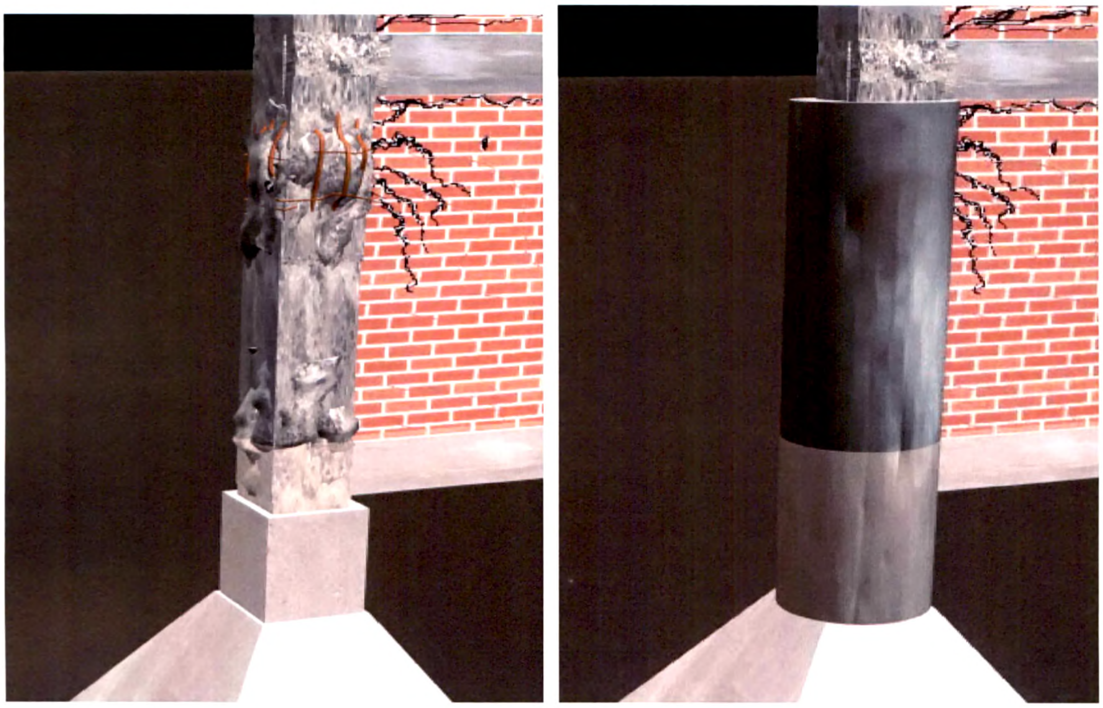


**Fig. 5.76 Steel Jacket for Concrete Column**

### 5.9.5 Beam-Column Joint Retrofitting

Beam-Column joints may be effectively retrofitted by means of resin injection, X-shaped prestressed collars, glued steel plates or R.C jackets depending on the degree of damage. Resin injections are applied in case of fine and moderate cracks, without degradation of concrete or buckling of the reinforcement bars. However, restoration of bond between steel and concrete with the aid of epoxy resin is questionable so the joint should be strengthened at the same time with one of the other techniques. After the cracks are filled with resin injections or epoxy or non-shrinking mortar, the joint is strengthened with external ties that are prestressed with tension couplers. Joint is then covered with welded wire fabric and a jacket of gunite concrete. Application of this technique is not feasible when four beams are framing into the joint.

Glued metal plates can also be applied to plane joints. Local repair precedes the gluing of the plates and then the plates are tied with prestressed bolts. This method provides strengthening to the joint without altering its dimensions. R.C jackets are applied in case of extensive damage to the joint. The jacket needs to be sufficiently thick to allow for providing additional reinforcement required for enhancement of flexural and shear strength at the junction (**Fig. 5.77-5.78**). In most cases joint jacketing is done in conjunction with beam and column jacketing. In view of the congested reinforcement at the joint, sufficient care is required to ensure proper placement of concrete/grout at this location. Use of carbon fibers may prove to be effective for isolated strengthening of joints.

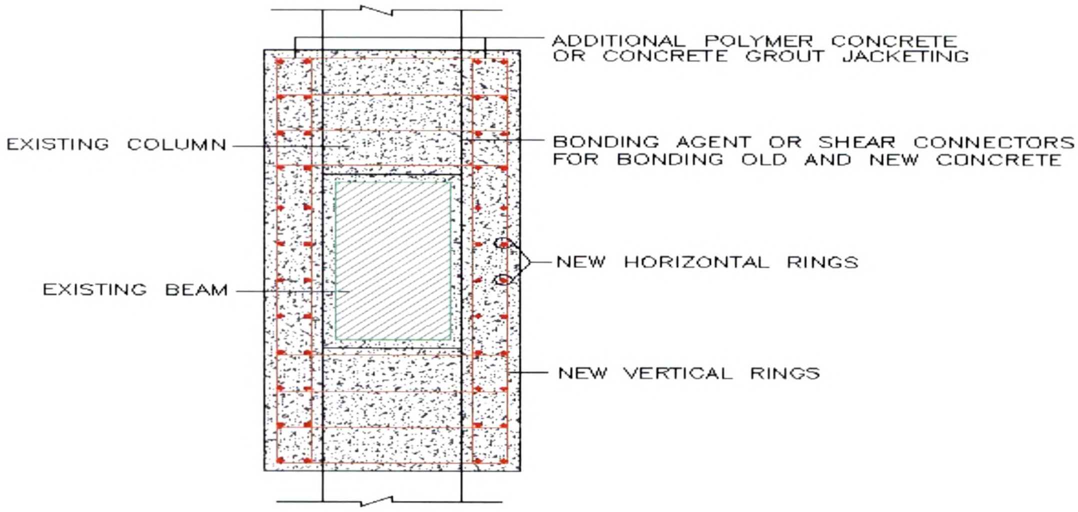



**Fig. 5.76 Beam-Column Joint Retrofit Details**

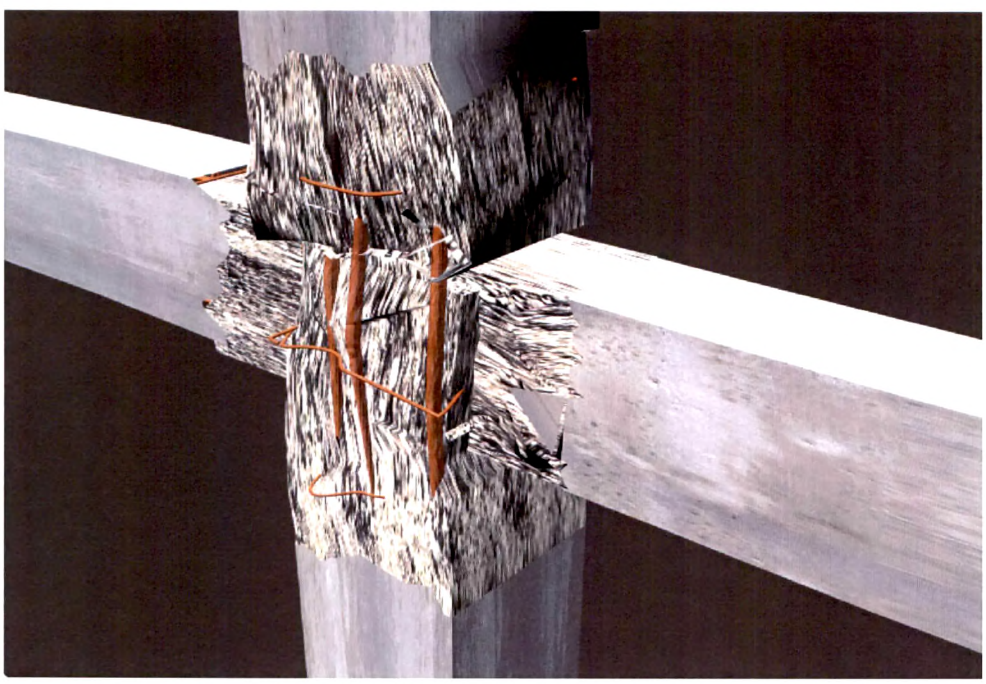


**Fig. 5.77 (a) Beam-Column Joint Before Retrofit**

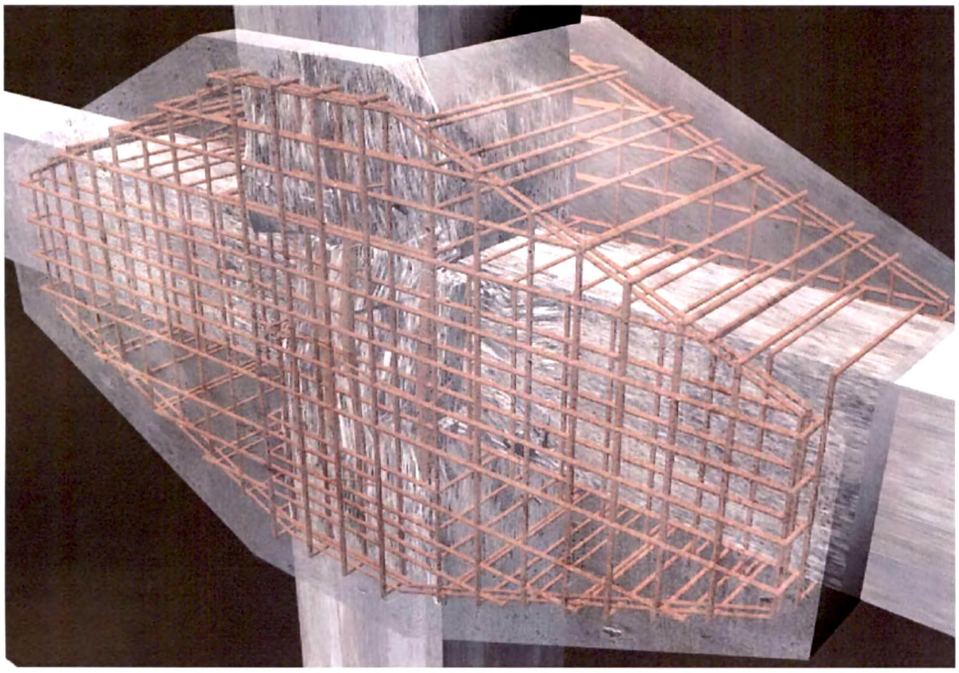


**Fig. 5.77 (b) Beam-Column Joint Retrofit by R.C Jacket**

### 5.9.6 Slab Retrofitting

Slab damage mainly appears in the form of cracks in the middle of large spans, near discontinuities such as corners of large openings and at the connections of stairs to the slabs. Depending on the extent and type of damage, a different degree of intervention can be applied. Repairing methods include repairs with epoxy resins when small cracks exhibit without crushing of the concrete or bond deterioration, increase the slab thickness or to add new reinforcement when check indicates that the slab resistance is insufficient. Effective force transfer between the old and new concrete is the key to success of the intervention. This can be accomplished by some other means besides roughening the old surface or resin coatings on the interface, such as anchors, dowels, welding of old and new reinforcement etc.

## 5.10 CLOSING REMARKS

Capability of computer program prepared in VC++, is described and virtual reality implementation has been demonstrated as a first step towards the automation of a comprehensive solution for earthquake forces. The program is self sufficient in evaluating existing member elements for their seismic capacity as well as designing new ones to withstand the same. Further, once deficiencies are identified, damage recovery is done either by increasing the stiffness with addition of new elements or increasing the strength by strengthening the existing elements or adding new ones to meet the global demands placed on the structure in the event of an earthquake. The entire database is then transferred into 3D Studio Max which is capable of very high quality 3 dimensional animation along with user interface options. Thus user gets a feel of the geometry, steel ductile detailing and displacements that will occur under the influence of the earthquake as also the damage occurring in various elements. Finally the virtual reality module also gives a brief review of local retrofitting options of beams, columns and slabs. The complete work is carried out in MAX environment for exploring the VR application. Geometry is built up from the Excel files through the first program "Pushover Analysis Application", where user can see the actual deformations of the members for any of the selected load case. Videos and MAX files can be viewed from the second program of "3DS MAX Application". Numerical calculations as per IS code criteria are provided in structural design section and in-depth explanation for failure patterns and retrofit methods have been discussed.