

**SEISMIC EVALUATION  
AND  
STRENGTHENING  
OF  
REINFORCED COCNETRE BUILDINGS**

by:-

**Dr.Anand S. Arya, FNA, FNAE, FIE  
(Professor Emeritus, Dept. of Eq. Engineering, IIT Roorkee)  
National Seismic Advisor (EVR)  
Ministry of Home Affairs**

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**SEISMIC EVALUATION AND STRENGTHENING OF EXISTING REINFORCED  
CONCRETE BUILDINGS - Guidelines**

**1. FOREWORD**

1.1 The seismic retrofitting consists in upgrading the strength of an existing structure with the aim to increase its capacity to withstand future earthquakes. The need for seismic retrofitting in existing building can arise due to any of the following reasons

- (i) buildings not designed to codes
- (ii) upgrading of code based seismic design forces
- (iii) upgrading of seismic zone
- (iv) deterioration of strength on aging of the structure
- (v) modification of the existing structures affecting its strength adversely
- (vi) change in use of the building increasing the floor loads

1.2 This document is intended to reduce the risk of death and injury that may result from the damaging effects of earthquakes on building which predate the current seismic codes [IS:1893-2002 (Part-1), IS:4326-1976 and IS:13920-1993] or have not been designed for earthquake forces.

1.3 This document describes a set of key steps and procedures for the assessment of the expected seismic performance of existing building in the event of a design level earthquake and where found necessary, strengthening of existing structural systems and elements for improved seismic performance.

1.4 Seismic forces for evaluation criteria of existing buildings are different from those meant for the design of new buildings. Appropriate modifications are made to address the issues of reduced serviceable life and acceptable risk for buildings with large occupancy and higher importance. Further, to account for uncertainty in the reliability of available information about the existing structure and the condition of structural components, strength calculations need to be suitably modified.

1.5 For deficient buildings, a broad outline for the design seismic strengthening measures has been developed and the interface with current design codes in general terms has been identified.

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1.6 In developing this document, assistance has been derived from the following publications:

- (1) ATC 33.03 - Guidelines for Seismic Evaluation of Existing Buildings,(75% complete draft), Applied Technology Council, CA.
- (2) Eurocode 8-Design Provisions for Earthquake Resistance of Structures-Part 3, CEN, Brussels, 2001
- (3) FEMA 178 -NEHRP Handbook for the seismic evaluation of existing buildings, Building Seismic Safety Council, Washington, D. C, 1992.
- (4) FEMA 154 -Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook, Federal Emergency Management Agency, Washington DC, USA, 1988.
- (5) FEMA 310 -Handbook for the Seismic Evaluation of Buildings – A Prestandard, Federal Emergency Management Agency, Washington DC, USA, 1998.
- (6) FEMA 356 - Prestandard and commentary for the Seismic Rehabilitation of Building, Federal Emergency Management Agency, Washington DC, USA, 20C
- (7) The Assessment and Improvement of the Structural Performance of Earthquake Risk Buildings - Draft for General Release, New Zealand National Society for Earthquake Engineering for Building Industry Authority, *New Zealand*, 1996T
- (8) Post-Earthquake Damage Evaluation and Strength Assessment of Buildings under Seismic Conditions, Volume 4, UNDP/UNIDO,
- (9) Seismic Evaluation of Existing Buildings, American Society of Civil Engineers, USA, 2003.
- (10) Uniform Code for Building Conservation, International Conference of Building Offices, Whittier, CA, USA, 1991.

## 2. **SCOPE**

2.1. This document is particularly concerned with the seismic evaluation and strengthening of the existing reinforced concrete buildings and it is intended to be used as a guideline.

2.2. This document provides a method to assess the ability of an existing building to reach an adequate level of performance related to life safety of occupants. Therefore, the emphasis is on identification of unfavorable characteristics of the building that could damage either part of the building or the entire structure.

## 3. **TERMINOLOGY**

3.1. For the purpose of this guideline, the definitions given in IS:1893 (Part I) -2002, IS:13920-1993 and the following shall apply.

### **3.2. Acceptance Criteria**

Limiting values of properties such as drift, strength demand, and inelastic deformation used to determine the acceptability of a component.

### **3.3. Action**

An internal moment, shear, torque, axial load, deformation, displacement, or rotation corresponding to a displacement due to a structural degree of freedom; designated as force- or deformation-controlled.

### **3.4. Capacity**

The permissible strength or deformation for a structural member or system.

### **3.5. Column (or Beam) Jacketing**

A method in which a concrete column or beam is covered with a steel or concrete *jacket* in order to strengthen and or repair the member by confining the concrete.

### **3.6. Components**

The basic structural members that constitute a building, including beams, columns, slabs, braces, walls, piers, coupling beams, and connections.

### **3.7. Deformation**

Relative displacement or rotation of the ends of a component or element or node.

### **3.8. Demand**

The amount of force or deformation imposed on an element or component.

### **3.9. Displacement**

The total movement, typically horizontal, of a component or element or node.

### **3.10. Flexible Diaphragm**

A floor diaphragm shall be considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.5 times the average displacement of the entire diaphragm.

### **3.11. Infill**

A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed *isolated infills*. Panels that are in tight contact with a frame around its full perimeter are termed *shear infills*.

### **3.12. Knowledge Factor**

A factor to represent the uncertainty about the reliability of the available information about the structural configuration and present condition of materials and components of the existing building.

### **3.13. Lateral Force Resisting System**

The collection of frames, shear walls, bearing walls, braced frames and interconnecting horizontal diaphragms that provide earthquake resistance to a building.

### **3.14. Life Safety Performance Level**

Building performance that includes significant damage to both structural and nonstructural components during a design earthquake, though at least some margin against either partial or total structural collapse remains. Injuries may occur, but the level of risk for life-threatening injury and entrapment is low.

### **3.15. Load Path**

A path that seismic forces take to the foundation of the structure and, finally, to soil. Typically, the load travels from the diaphragm through connections to the vertical lateral-force-resisting elements, and then proceed to the foundation.

### **3.16. Nonstructural Component**

Architectural, mechanical or electrical components of a building that are permanently installed in, or are an integral part of, a building system.

### **3.17. Occupancy Risk Factor**

A factor to represent acceptable risk of damage for existing buildings as a function of occupancy and importance.

### **3.18. Overturning**

Action resulting when the moment produced at the base of vertical lateral-force-resisting elements is larger than the resistance provided by the foundation's uplift resistance and building weight.

### **3.19. Plan Irregularity**

Horizontal irregularity in the layout of vertical lateral-force-resisting elements, producing a mismatch between the center of mass and center of rigidity that typically results in significant torsional demands on the structure.

### **3.20.Pounding**

Two adjacent buildings coming into contact during earthquake excitation because they are too close together and/or exhibit different dynamic deflection characteristics.

### **3.21.Primary Component**

Those components that are required as part of the building's lateral-force-resisting system.

### **3.22.Primary Element**

An element that is essential to the ability of the structure to resist earthquake-induced deformations.

### **3.23.Probable or Measured Nominal Strength**

The capacity of a structure or component to resist the effects of loads, as determined by (1) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics; or (2) field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

### **3.24.Redundancy**

Quality of having alternative paths in the structure by which the lateral forces are resisted, allowing the structure to remain stable following the failure of any single element.

### **3.25.Required Member Resistance (or Required Strength)**

Load effect acting on an element or connection, determined by structural analysis, resulting from the factored loads and the critical load combinations.

### **3.26.Rigid Diaphragm**

A floor diaphragm shall be considered to be rigid, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is less than 1.5 times the average displacement of the entire diaphragm. Reinforced concrete monolithic slab-beam floors or those consisting of prefabricated/precast elements with topping reinforced screed can be taken as rigid diaphragms.

### **3.27.Secondary Component**

Those components that are not required for lateral force resistance. They may or may not actually resist some lateral forces.

### **3.28.Secondary Element**

An element that does not affect the ability of the structure to resist earthquake-induced deformations.

### **3.29. Seismic Demand**

Seismic hazard level commonly expressed in the form of a ground shaking response spectrum. It may also include an estimate of permanent ground deformation.

### **3.30. Seismic Evaluation**

An approved process or methodology for evaluating deficiencies in a building which prevent the building from achieving life safety objective.

### **3.31. Short Column**

The reduced height of column due to surrounding parapet, infill wall, etc. is less than five times the dimension of the column in the direction of parapet, infill wall, etc. or 50% of the nominal height of the typical columns at that level.

### **3.32. Strength**

The maximum axial force, shear force, or moment that can be resisted by a component.

### **3.33. Strengthening Measures**

Modifications to existing components, or installation of new components, that correct deficiencies identified in a seismic evaluation as part of a strengthening scheme.

### **3.34. Strengthening Method**

A procedural methodology for the reduction of building vulnerability.

### **3.35. Strengthening Strategy**

A technical approach for developing strengthening measures for a building to reduce its earthquake vulnerability.

### **3.36. Strong Column-Weak Beam**

The capacity of the columns in any moment frame joint must be greater than that of the beams, to ensure inelastic action in the beams.

### **3.37. Vertical Irregularity**

A discontinuity of strength, stiffness, geometry, or mass in one story with respect to adjacent stories.

## **4. LIST OF SYMBOLS**

The symbols and notations given below apply to the provisions of this document:

$A_c$  Total cross-sectional area of columns

$A_{hm}$  Modified seismic coefficient

$A_n$  Area of net mortared/grouted section

$A_w$	Total area of shear walls in the direction of loading
$C_p$	Horizontal force factor
$D$	In-plane width dimension of masonry
$f_{ck}$	Characteristic strength of concrete
$F_o$	Axial force due to overturning
$F_{wx}$	Force applied to a wall at level $x$
$H$	Total height of the building
$H'$	Least clear height of opening on either side of $p_{ij}$
$L$	Length of the building
$m_l$	Factor for reduced useable life
$m_o$	Occupancy risk factor
$m_k$	Knowledge factor
$n_c$	Total number of columns
$n_f$	Total number of frames
$t$	Thickness of wall
$v_u$	Unit shear strength of the diaphragm
$V_B$	Base shear
$V_c$	Column shear force
$V_{ca}$	Total shear capacity of cross walls in the direction of analysis immediately above the diaphragm level being investigated
$V_{cb}$	Total shear capacity of cross walls in the direction of analysis immediately below the diaphragm level being investigated
$V_d$	Diaphragm shear
$V_j$	Story shear at level;
$V_{wx}$	Total shear force resisted by a shear wall at the level under consideration
$W_d$	Total dead load tributary to a diaphragm
$\tau_{col}$	Average shear stress in concrete columns
$\tau_{wall}$	Average shear stress in walls

## 5. - Evaluation Criteria

### 5.1 - Main Causes of damage of RC buildings observed in earthquake

1. *Lack of good design* in planning lateral load resisting system such as moment resistant frames, frames with shear walls or with infill walls, and the joints.
2. *Poor quality* of construction materials and technology.
3. *Inadequate detailing* of reinforcement in beams, columns, beam-column joints, particularly from ductility considerations.
4. *Inadequate diaphragm* action of roofs and floors.
5. *Inadequate treatment* of non-structural components like infill masonry walls, staircases, water tanks on roofs, etc.

The evaluation criteria are aimed at determining the weakness/deficiencies in design and construction, which will make the buildings susceptible to the above mentioned damage.

## 5.2

The seismic performance of existing buildings is evaluated in relation to the performance criteria in use for new buildings. This section defines the minimum evaluation criteria for the expected performance of life- safety of existing buildings with appropriate modification to IS: 1893 seismic force, which is applicable for the seismic design of new buildings. Since the provisions of this code are strongly correlated with the design criteria of new buildings contained in IS: 1893, reference shall always be made to the current edition of IS: 1893. All existing structural elements must be able to carry full other non-seismic loads in accordance with the current applicable codes related to loading and material strengths.

## 5.3 -

Basic inputs for determination of seismic forces such as seismic zone, building type, response reduction factor are to be taken directly from IS: 1893. Alternatively, a site-specific seismic design criteria developed along the principles described in IS: 1893 may be used. Modification to seismic forces as given in IS: 1893 and to material strengths will be applicable to both preliminary and detailed assessments described in this document.

## 5.4 - Lateral Load Modification Factor

The lateral force determined for strength related checks needs to be modified for reduced useable life. The useable life factor  $U$ , is to be multiplied to the lateral force (base shear) for new building as specified in IS: 1893-2002 (Part – 1).  $U$  will be determined as

$$U = (T_{\text{rem}}/T_{\text{des}})^{0.5}$$

where

$T_{\text{rem}}$  = Remaining useful life of the building

$T_{\text{des}}$  = Design useful life of the building.

$U$  will not be taken less than 0.7 in any case.

### **Note:-**

1. *By comparing the requirements of the versions of IS:1893 of 2002 with 1984, 1975, 1966 & 1962 versions, it is seen that buildings designed accordingly from time to time, will be found deficient to some extent.*
2. *It may be mentioned that buildings designed as per 1984 version will in general not need retrofitting except those on stilts (soft first storey) and those using 230 mm or thinner columns will need retrofitting.*
3. *Buildings designed to 1975 version of the code may be found deficient to a small extent. Engineer incharge may use his discretion in regard to retrofitting decision.*

4. *Building designed to earlier code versions, unless over designed and those not designed for earthquake forces will generally need retrofitting.*
5. *Factor U may be applied in all cases (except in a building of critical safety, if desired U may be taken as 1.0).*
6. *Stability of infill walls under out of plane lateral earthquake must be ensured in each case.*

### **5.5 - Modified Material Factor**

Strength capacities of existing building components should be based on the probable material strengths in the building. Probable or measured nominal strengths are best indicator of the actual strength and can only be obtained by field or lab tests on a series of samples. This document recommends that probable strengths are either based on actual tests or the default values given in the subsequent sections of this document. These can also be assessed from the values given in the original building documents. However, they all need to be further modified for the uncertainty regarding the reliability of available information, and present condition of the component. The probable material strengths need to be multiplied with a Knowledge Factor, K as defined in Table 1.

**Table 1: Knowledge factor, K**

<b>S. No</b>	<b>Description of Building</b>	<b>K</b>
1	Original construction documents available, including post-construction activities, such as modification to structure or materials testing undertaken of existing	1.00
2	Documentation as above in (1) but no testing of materials, i.e., using originally specified values for materials.	0.90
3	Documentation as above in (1) but no testing of materials, i.e., originally specified values for materials and minor deterioration of original condition	0.85
4	Incomplete but useable original construction documents and no testing.	0.80
5	Documentation as in (4) and limited inspection, and verification of structural members, or materials test results with large variation	0.75
6	Little knowledge of details of a component	0.70

### **5.6 - Evaluation Process**

Existing buildings that were not designed in accordance with the principles and philosophies of earthquake codes and buildings that predate the current seismic code as described in the following sections would be assessed.

#### **5.6.1 -**

A preliminary evaluation of building is carried out which involves broad assessment of apparent physical condition, robustness, structural integrity and strength of structure, including simple calculations.

### 5.6.2 -

If the results of preliminary evaluation for strength, overall stability and integrity are acceptable, no further action is required. Else a detailed evaluation is required unless exempted.

### 5.6.3 -

A detailed evaluation when required will include numerical checks on stability and integrity of the whole structure as well as the strength of each member. Conventional design calculations for these checks will use modified demands and strengths. A flow diagram summarizing various steps of the evaluation process is shown in Figure 1.

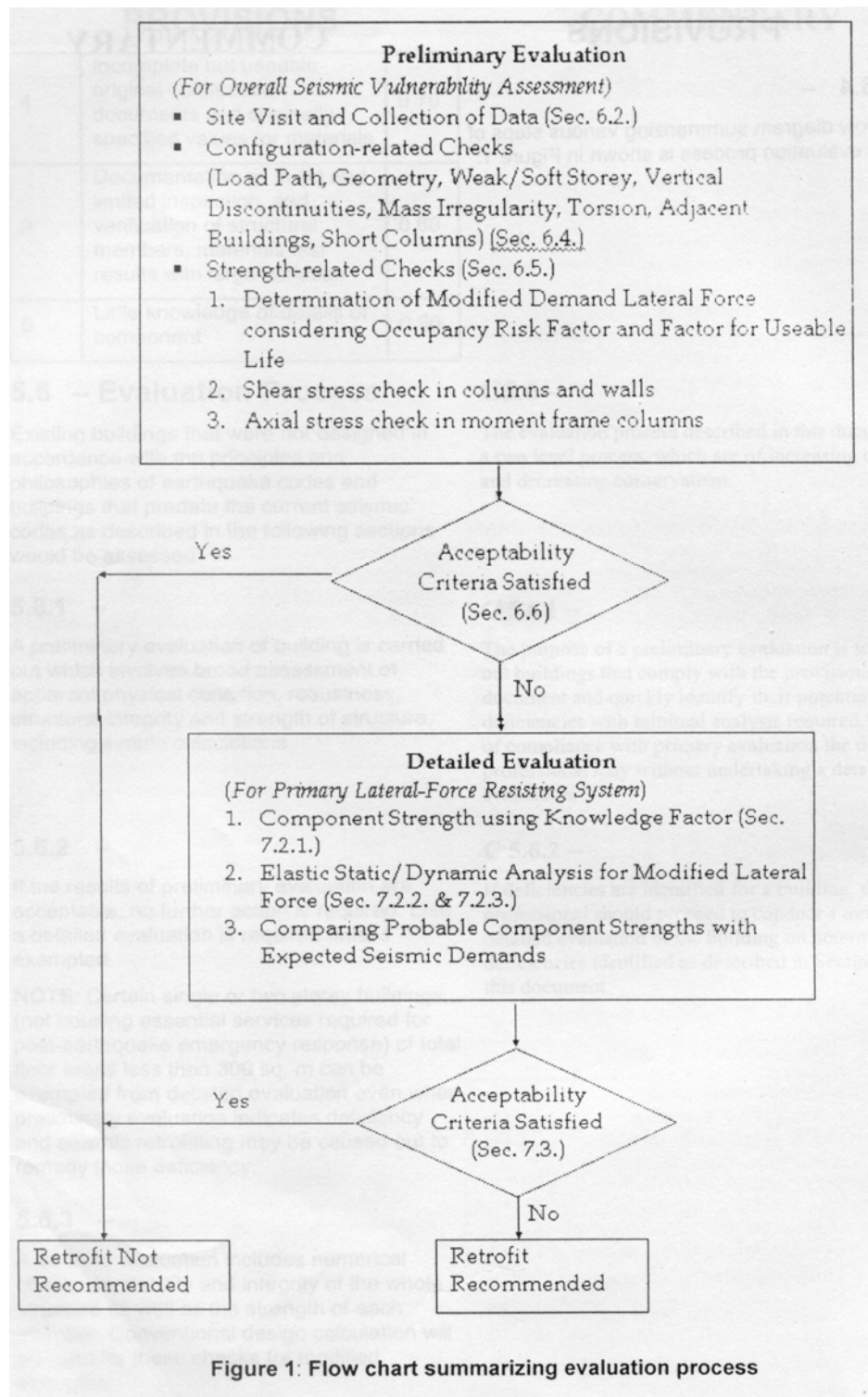


Figure 1: Flow chart summarizing evaluation process

## 6. -PRELIMINARY EVALUATION

### 6.1 - General

Preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that can affect its seismic vulnerability. It is a very approximate procedure based on conservative parameters to identify the potential earthquake risk of a building and can be used

to screen buildings for detailed evaluation. Method is primarily based on observed damage characteristics in previous earthquakes coupled with some simple calculations.

## **6.2 - Site Visit**

A site visit will be conducted by the design professional to verify available existing building data or collect additional data, and to determine the condition of the building and its components. The following information either needs to be confirmed or collected during the visit:

- (a) General information: Number of storeys and dimensions, year of construction
- (b) Structural system description: Framing vertical lateral force-resisting system, floor and roof diaphragm connection to frames, basement and foundation system
- (c) Building type as in IS 1893 (Part1)
- (d) Site soil classification as in IS 1893 (Part 1)
- (e) Building use and nature of occupancy
- (f) Adjacent buildings and potential for pounding and falling hazards
- (g) General conditions: Deterioration of materials, damage from past earthquakes, alterations and additions that could affect earthquake performance
- (h) Architectural features that may affect earthquake performance, especially location of masonry infill walls.

## **6.3 - Acceptability Criteria**

A building is said to be acceptable if it meets ALL the configuration-related checks as well as global level checks on axial and shear stress as outlined in the following sections.

## **6.4 -Configuration-Related Checks**

### **6.4.1 - Load Path**

The structure shall contain at least one rational and complete load path for seismic forces from any horizontal direction so that they can transfer all inertial forces in the building to the foundation.

### **6.4.2 -Geometry**

No change *should be made* in the horizontal dimension of lateral force resisting system of more than 50% in a storey relative to adjacent stories, excluding penthouses and mezzanine floors.

### **6.4.3 - Weak Storey**

The strength of the lateral force resisting system in any storey shall not be less than 80% of the strength in the storey above.

#### **6.4.4 -Soft Storey**

The stiffness of lateral load resisting system in any storey shall not be less than 70% of the stiffness in the storey above or less than 80% of the average stiffness of the three storeys above.

#### **6.4.5 - Vertical Discontinuities**

All vertical elements in the lateral force resisting system shall be continuous from the roof to the foundation.

#### **6.4.6 - Mass**

There shall be no change in effective mass more than 100% from one storey to the next. Light roofs, penthouses, and mezzanine floors need not be considered in mass irregularity.

#### **6.4.7 - Torsion**

The estimated distance between a storey center of mass and the storey centre of stiffness shall be less than 30% of the building dimension at right angles to the direction of loading considered.

#### **6.4.8 - Adjacent Buildings**

The clear horizontal distance between the building under consideration and any adjacent building shall be greater than the gap width specified in Cl.7.11.3 of IS:1893 (Part-1) – 2002.

#### **6.4.9 - Short Columns**

The reduced height of a column due to surrounding parapet, infill wall, etc. shall not be less than five times the dimension of the column in the direction of parapet, infill wall, etc. or 50% of the nominal height of the typical columns in that storey.

#### **6.4.10 - Redundancy**

The number of lines of moment frames in each principal direction shall be greater than or equal to 2. The number of bays of moment frames in each line shall be greater than or equal to 2.

#### **6.4.11 - Deterioration of Concrete**

There should be no visible deterioration of the concrete or reinforcing steel in any of the vertical or lateral force resisting elements. In case it is there, proper restoration will need to be taken before or along with retrofitting.

#### **6.4.12 - Mezzanine/loft/sub-floor –**

Interior mezzanine/loft/sub-floor levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure.

## 6.5 -Strength-Related Checks

Approximate and quick checks shall be used to compute the strength and stiffness of building components. The seismic base shear and storey shears for the building shall be computed in accordance with IS 1893-2002 (Part-1) and the modification factor U as per cl.5.4 of this document.

### 6.5.1 - Shear Stress in RC Frame Columns

The average shear stress in concrete columns,  $\tau_{col}$ , computed in accordance with the following equation shall be limited to  $0.1 \sqrt{f_{ck}}$  where,  $f_{ck}$  is characteristic cube strength of concrete:

$$\tau_{col} = \left[ \frac{V_j}{n_c - n_f} \right] \left[ \frac{1}{A_c} \right] n_c \dots \dots \dots (6.1)$$

where,

$n_c$  = total no. of columns,

$n_f$  = total no. of frames in the direction of loading,

$V_j$  = storey shear at level  $j$  and

$A_c$  = total cross-sectional area of columns.

### 6.5.3 - Axial Stress in Moment Frames

Axial stress in the columns of moment frames at base due to overturning forces alone ( $F_o$ ) as calculated using the following equation shall be less than  $0.25f_{ck}$ .

$$F_o = \frac{3}{4} \left[ \frac{V_B}{n_f} \right] \left[ \frac{H}{L} \right] \dots \dots \dots (6.3)$$

where,

$n_f$  = total no. of frames in the direction of loading

$V_B$  = base shear

$H$  = total height

$L$  = length of the building.

## 6.6 - Recommendation for Detailed Evaluation

A building is recommended to undergo a detailed evaluation as described in Section 6, if any of the following conditions are met:

- a) The building fails to comply with the requirements of the Preliminary Evaluation.
- b) A building is 6 storeys and higher.
- c) Buildings located on incompetent or liquefiable soils and/or located near (less than 12 km) active fault trace and/or with inadequate foundation details.
- d) Buildings with inadequate connections between primary structural members, such as poorly designed and/or constructed joints of pre-cast elements.

## 7. - DETAILED EVALUATION

### 7.1 - Checking Original Design Details

The following details shall be checked in the original design. Any deficiency should be considered in choosing the response reduction factor  $R$  in detailed evaluation and in the retrofit design.

- (a) **No Shear Failures** — 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 for shear design of beams and columns.
- (b) **Strong Column/Weak Beam** – The sum of the moment of resistance of the columns at any joint 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.
- (c) **Beam Bars** - At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars located at the joints for either positive or negative moment shall be continuous throughout the length of the members.
- (d) **Column-Bar Splices** - Lap splices shall be located only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be located over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall preferably be spliced at one section. If more than 50 percent of the bars are spliced at one section, the lap length shall be  $1.3 L_d$  where  $L_d$  is the development length of bar in tension as per IS 456: 2000.
- (e) **Beam- bar Splices** - Longitudinal bars shall be spliced only if hoops are located over the entire splice length, at a spacing not exceeding 150 mm. The lap length shall not be less than the bar development length in tension. Lap splices shall not be located (a) within a joint, (b) within a distance of  $2d$  from joint face, and (c) within a quarter length of the member where flexural yielding may occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.
- (f) **Column-Tie Spacing** - The parallel legs of rectangular hoop shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, the provision of a crosstie should be there. Alternatively, a pair of overlapping hoops may be located within the column. The hooks shall engage peripheral longitudinal bars.
- (g) **Stirrup Spacing**—The spacing of stirrups over a length of  $2d$  at either end of a beam shall not exceed (a)  $d/4$ , or (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50 mm from the joint face. In case of beams vertical hoops at the same spacing as above shall also be located over a length equal to  $2d$  on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding  $d/2$ .
- (h) **Joint Reinforcing**— Beam-column joints shall have ties spaced at or less than 150 mm.

- (j) **Stirrup and Tie Hooks** - The beam stirrups and column ties shall preferably be anchored into the member cores with hooks of 135° and 6d extension.

## 7.2 - RC Frames with Masonry Infill Wall

The infill walls should be checked for the following criteria:

- (a) **Masonry Units** — There shall be no visible deterioration of masonry units.
- (b) **Masonry Joints** — The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.
- (c) **Cracks in Infill Walls** — There shall be no existing diagonal cracks in infill walls.
- (d) **Cracks in Boundary Columns** — There shall be no existing diagonal cracks in concrete columns that encase masonry infills.
- Note:- In case (a) to (d) exist, restoration will be needed before or during retrofitting*
- (e) **Wall Connections** — All infill walls shall have a positive connection to the frame to resist out-of-plane forces. If not, positive connection will need to be provided during retrofitting.

## 7.3 - Detailed Evaluation Procedure

That is, the Available Seismic Resistance should at least be equal to the Minimum Seismic Resistance required. These are defined below:

a) *Minimum seismic resistance*

The required minimum seismic resistance (MSR) is expressed quantitatively by the design seismic coefficient as per IS: 1893-2002, which includes (i) the basic seismic coefficient for the zone (ii) the fundamental period of the building, and (iii) the importance factor of the building. The overall lateral force (or Base shear) on the building is obtained by multiplying the design seismic coefficient and the total weight of the building. Seismic demand on critical individual components shall be determined using seismic analysis methods described in IS 1893-2002 (part-1) for lateral forces prescribed therein with modification for reduced Useable life (i.e., factor U), described in Section 5.4 of the document.

b) *Available seismic resistance*

The available seismic resistance (ASR) of a building is expressed quantitatively by the earthquake force under which the first of the columns of any building storey will reach its ultimate limit strength, when the remaining structure remains in the undamaged state. The probable strengths determined from conventional methods and applicable codes shall be modified with appropriate knowledge factor  $K$  given in table 1.

### **7.3.1 -**

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 as per respective codes for various building types and the strength modified with knowledge factor (K).

### **7.3.2 -**

Calculate the total lateral force (design base shear) in accordance with IS: 1893 and multiply it with U the factor for reduced useable life (see 5.4).

### **7.3.3 -**

Perform a linear equivalent static or dynamic analysis of lateral load resisting system of the building for the base shear calculated as in 7.2.2 and determine the resulting member actions for critical components.

### **7.3.4 -**

Evaluate acceptability of each component by comparing its probable strength with the seismic actions.

### **7.3.5 -**

Estimate the storey drifts and decide whether it is acceptable in terms of the requirements of IS 1893-2002 (Part-1).

## **7.4 - Acceptability Criteria**

A building is said to be acceptable if either of the following two conditions are satisfied:

- a) All critical elements of lateral force resisting elements have strengths greater than computed actions and drift checks are satisfied.
- b) Except a few elements, all critical elements of the lateral force resisting elements have strengths greater than computed actions and drift checks are satisfied. The engineer has to ensure that the failure of these few elements will not lead to loss of stability or initiate progressive collapse. This needs to be verified by a non-linear analysis such as pushover analysis, carried out upto the collapse load.

## **8. - SEISMIC STRENGTHENING**

### **8.1 - General**

This section outlines seismic strengthening options and strategies at a general level, and describes a methodology for the design of the strengthening measures as modifications to correct/ reduce seismic deficiency identifying during the evaluation procedure discussed in Section 7.

## **8.2 - Seismic Strengthening Options and Strategies**

Seismic strengthening for improved performance in the future earthquakes can be achieved by one of several options discussed in this section. The chosen seismic strengthening scheme should increase the redundancy of lateral load resisting elements to avoid collapse and overall instability.

### **8.2.1 - Strengthening at Member Level**

#### **8.2.1.1 -**

Existing buildings with a sufficient level of strength and stiffness at the global level may have some members (or components), which lack adequate strength, stiffness or ductility. If such deficient members are small in number, an economical and appropriate strategy is to modify these deficient members alone while retaining the existing lateral-force resisting system.

#### **8.2.1.2-**

Member level modification can be undertaken to improve strength, stiffness and/or ductility of deficient members and their connections. Strengthening measures could include such as jacketing columns or beams.

#### **8.2.1.3-**

Member level strengthening measures that enhance ductility of the member without significantly increasing its strength/stiffness are often useful when analysis indicates that a few members of the lateral-load resisting system are deficient. One such measure is jacketing of RC columns, which improves the member level ductility by increased confinement.

### **8.2.2 - Eliminating or Reducing Structural Irregularities**

#### **8.2.2.1 -**

Irregularities related to distribution of strength, stiffness and mass result in poor seismic performance. Often these irregularities exist because of discontinuity of structural members. Simple removal of such discontinuities may reduce seismic demand on other structural components to acceptable levels.

#### **8.2.2.2 -**

An effective measure to correct vertical irregularities such as weak and/or soft storey is the addition of shear walls and braced frames within the weak/soft storey. Braced frames and shear walls can also be effectively used to balance stiffness and mass distribution within a storey to reduce torsional irregularities.

### **8.2.2.3 -**

Seismic gaps (or movement joints) can be created wherever possible between various parts of a building with irregular plan geometry to separate it into a number of regular independent structures. However, care should be exercised to provide sufficiently wide gaps to avoid the problem of pounding.

## **8.2.3 - Strengthening At Structural Level**

### **8.2.3.1 -**

In structures where more than a few critical members and components do not have adequate strength and ductility, an effective way is to strengthen the structure so that the overall displacement demands can be reduced. It may enhance force demands on some other elements, which may require further strengthening. Braced frames and shear walls are an effective means of adding stiffness and strength.

## **8.3 - Alternative Strengthening Options**

Often existing buildings do not have sufficient lateral resistance against earthquake forces and resulting large ductility demands for members are too difficult to be met by them. Provision of supplemental strength in form of additional lateral force-resisting elements such as shear walls, braced or moment frames help reduce inelastic demands on the existing gravity load resisting elements.

### **8.3.1 - Supplemental Damping and Isolation**

Seismic isolation and supplemental damping are rapidly evolving unconventional strategies for improving seismic performance of structures.

An overall response of base isolation is reduction in demands on the elements of the structure. This technique is most effective for relatively stiff buildings with low profiles and large mass compared to light, flexible structures. However, base isolation is technically complex and costly to implement.

Energy dissipation helps in overall reduction in displacements of the structure. This technique is most effective in structures that are relatively flexible and have some inelastic deformation capacity. Again, this strategy is technically complex, but less costly compared to base isolation.

## **8.4 - Methods of Analysis and Design for Strengthening**

### **8.4.1 - Design Criteria**

The performance criteria for the design of strengthening measures shall be same as for evaluation process as defined in Section 5.

## 8.4.2 - Member Capacities

Member capacities of existing elements shall be based on the probable strengths as defined in Section 5 and also used for Detailed Evaluation.

## 8.4.3 - Analysis Options

The engineer may choose to perform the same analysis as performed during the evaluation process.

## 8.5 - Strengthening Options for RC Framed Structures

### 8.5.1 -

The, deficient frame members and joints are identified during detailed evaluation of building. Members requiring strengthening or enhanced ductility can be jacketed by reinforced concrete jacketing, steel profile jacketing, steel encasement or wrapping with FRPs. Where possible, the deficient members should first be stress-relieved by propping.

- (a) RC jacketing involves placement of new longitudinal reinforcement and transverse reinforcement bars in the new concrete overlay around existing member.
- (b) Steel profile jacketing can be done through steel angle profiles placed at each corner of the existing reinforced concrete member and connected together as a skeleton with transverse steel straps. Another way is by providing steel encasement. Steel encasement is the complete covering of the existing member with thin plates.
- (c) Retrofitting using FRPs involves placement of composite material made of continuous fibers with resin impregnation on the outer surface of the RC member.

#### 8.5.1.1 - RC Jacketing of Columns

Reinforced concrete jacketing improves column flexural strength and ductility. Closely spaced transverse reinforcement provided in the jacket improves the shear strength and ductility of the column. The procedure for reinforced concrete jacketing is:

- (a) The seismic demand on the columns, in terms of axial load ( $P$ ) and moment ( $M$ ) is obtained.
- (b) The column size and section details are estimated for  $P$  and  $M$  as determined above.
- (c) The existing column size and amount of reinforcement is deducted from the values obtained considering the demand.
- (d) The extra size of column cross-section and reinforcement is provided in the jacket.
- (e) The actual concrete and steel provided in the jacket is as given below:

$$A_c = (3/2) A_c' \text{ and } A_s = (4/3) A_s'$$

where

$A_c$  and  $A_s$  = Actual concrete and steel to be provided in the jacket

$A_c'$  and  $A_s'$  = Concrete and steel values obtained for the jacket after deducting the existing concrete and steel from their respective required amount.

(f) The spacing of ties to be provided in the jacket in order to avoid flexural shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = \frac{f_y}{\sqrt{f_{ck}}} \frac{d_h^2}{t_j}$$

where

$f_y$  = yield strength of steel

$f_{ck}$  = cube strength of concrete

$d_h$  = diameter of stirrup

$t_j$  = thickness of jacket

(g) In order to transfer the additional axial load from the old to the new longitudinal reinforcement, bent down bars are provided which are intermittent lap welded to bars of jacket and longitudinal bars in the existing column exposed for the purpose. Moreover, bent-down bars help in good anchorage between existing and new concrete.

(h) The number of bent-down bars required is given as,

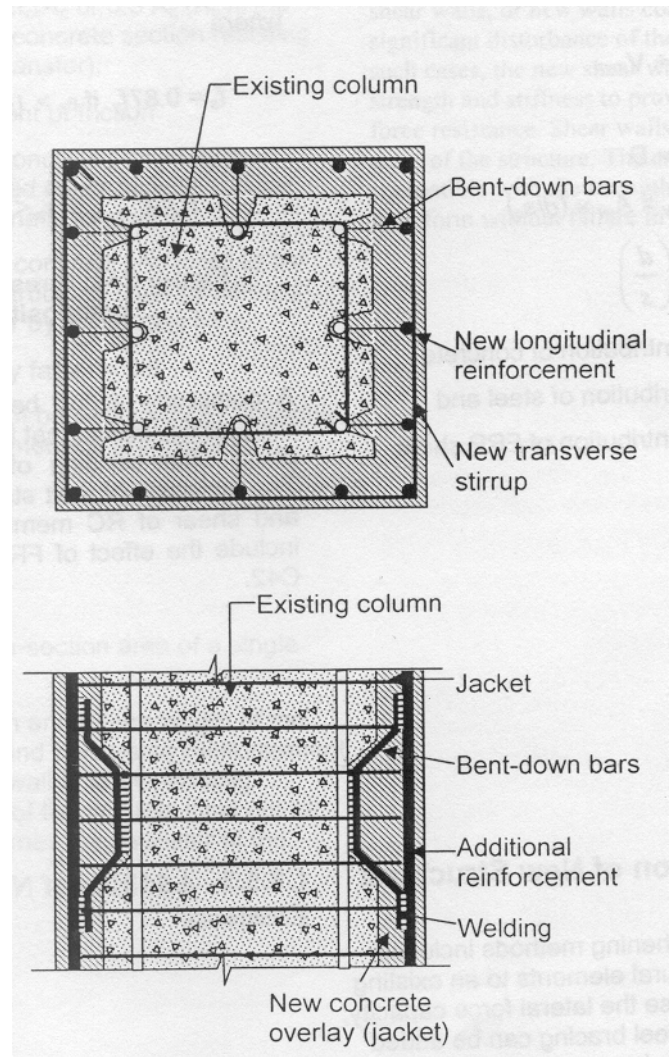
$$n_a = \left[ \frac{\Delta P}{20 A_{sb} + 10 h_s} \right]$$

where

$\Delta P$  = additional axial load to be transferred to the jacket reinforcement.

$A_{sb}$  = total cross-section of the bent down bars.

$h_s$  = width of bent-down bars.



**Figure 3: Reinforced Concrete Jacketing**

- (i) If the transfer of axial load to new longitudinal steel is not critical then friction present at the interface can be relied on for the shear transfer, which can be enhanced by roughening the old surface.
- (j) Dowels which are epoxy grouted and bent into 90° hook can also be employed to improve the anchorage of new concrete jacket.

The minimum specifications for jacketing of columns are:

- (a) Strength of the new materials must be equal or greater than those of the existing column. Concrete strength should be at least 5 MPa greater than the strength of the existing concrete.

- (b) For columns where extra longitudinal reinforcement is not required, a minimum of  $12\phi$  bars in the four corners and ties of  $8\phi @ 100$  c/c should be provided with  $135^\circ$  bends and  $10\phi$  leg lengths.
- (c) Minimum jacket thickness should be 100 mm.
- (d) Lateral support to all the longitudinal bars should be provided by ties with an included angle of not more than  $135^\circ$ .
- (e) Minimum diameter of ties should be 8 mm and not less than  $1/3$  of the longitudinal bar diameter.
- (f) Vertical spacing of ties shall not exceed 200 mm, whereas the spacing close to the joints within a length of  $1/4$  of the clear height should not exceed 100 mm. Preferably, the spacing of ties should not exceed the thickness of the jacket or 200 mm whichever is less.

### 8.5.1.2- Fiber Jacketing of a beam

Dimensions of FRP jacket is determined assuming composite action between fiber and existing concrete. The rupture strength of FRP is used as its limiting strength.

Limit state moment capacity of FRP retrofitted member is given by:

Ultimate flexural strength is determined based on the assumption that compressive concrete reaches a strain of 0.0035 and FRP reaches its maximum strain.

Shear strength of a beam after strengthening:

$$V = V_{con} + V_S + V_{FRP}$$

where,

$$V_{con} = t_c \times b \times D$$

$$V_S = 0.87 \times f_y \times A_{sv} \times (d/S_v)$$

$$V_{FRP} = A_f f_f (d/s)$$

$V_{con}$  is shear contribution of concrete

$V_S$  is shear contribution of steel and

$V_{FRP}$  is shear contribution of FRP sheet

### 8.5.2 - Addition of New Structural Elements

One of the strengthening methods includes adding new structural elements to an existing structure to increase the lateral force capacity. Shear walls and steel bracing can be added as new elements to increase the strength and stiffness of the structure.

#### 8.5.2.1 -

Addition of new reinforced concrete shear walls provides a better option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures. The design of shear walls shall be done as per IS: 13920.

- (a) The shear transfer reinforcement (dowel bars), perpendicular to the shear plane, is given as,

$$A_{vf} = \frac{V_u}{f_y \mu} \eta$$

where,

$V_u$  = Allowable shear force not greater than  $0.2f_{ck}A_c$  or  $5.5 A_c$  ( $A_c$  is the area of concrete section resisting shear transfer).

$\mu$  = Coefficient of friction

= 1.0 for concrete placed against hardened concrete with surface intentionally roughened.

= 0.75 for concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars.

$\eta$  = Efficiency factor = 0.5

(b) The number of bars required for resisting shear at the interface are given as,

$$n = \frac{A_{vf}}{A_{vf}'}$$

where,  $A_{vf}'$  = cross-section area of a single bar.

(c) The minimum anchorage length of the grouted-in longitudinal and transverse reinforcement of the shear wall in to the existing components of the building shall not be less than 6 times the diameter of the bars.

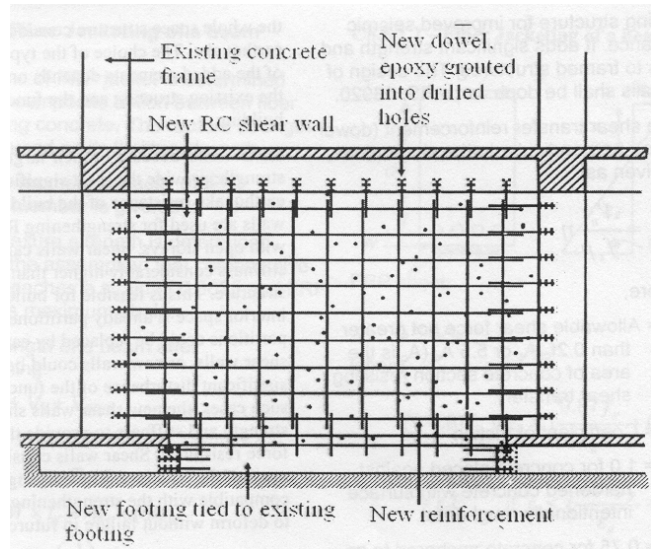
#### 8.5.2.2 -

Steel diagonal braces can be added to existing concrete frames. Braces should be arranged so that their center line passes through the centers of the beam-column joints. Angle or channel steel profiles can be used. Some of the design criteria for braces are given below:

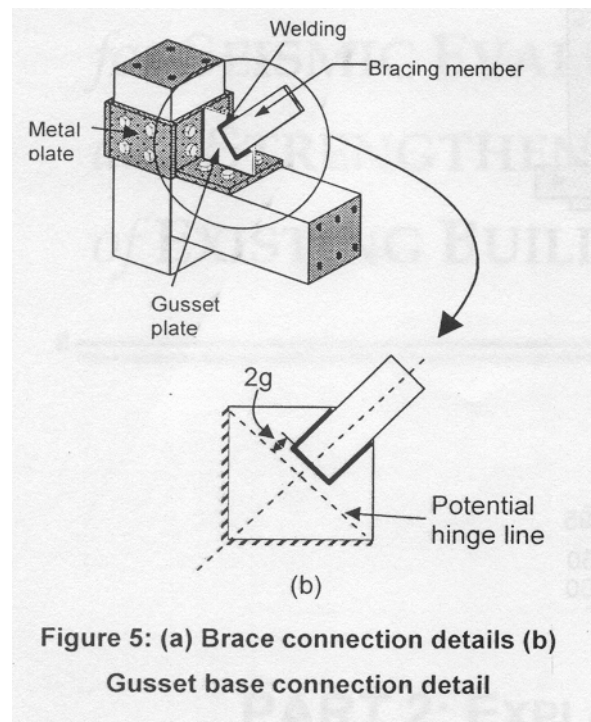
(a) Slenderness of bracing member shall be less or equal to  $2500/\sqrt{f_y}$ .

(b) The width-thickness ratio of angle sections for braces shall not exceed  $136/\sqrt{f_y}$ . For circular sections the outside diameter to wall thickness ratio shall not exceed  $8960/f_y$ , and rectangular tubes shall have an out-to-out width to wall thickness ratio not exceeding  $288/\sqrt{f_y}$ .

(c) In case of Chevron (V) braces, the beam intersected by braces shall have adequate



**Figure 4: Adding new shear walls**



**Figure 5: (a) Brace connection details (b) Gusset base connection detail**

strength to resist effects of the maximum unbalanced vertical load applied to the beam by braces. This load shall be calculated using a minimum of yield strength  $P_y$  for the brace in tension and a maximum of 0.3 times of load capacity for the brace in compression  $P_{ac}$ .

- (d) The top and bottom flanges of the beam at the point of intersection of V braces shall be designed to support a lateral force equal to 2% of the beam flange strength  $f_y b_f t_f$ .
- (e) The brace connection should be adequate against out-of-plane failure and brittle fracture. Typical connection detail is shown in Figure 5.

### 8.5.2.3 -

Prefabricated steel bracing subassemblages as shown in Figure 6 can be used, for ease of construction, Braces in X-, V- and inverted V- can be arranged inside a heavy rectangular steel rim, which is then placed in frame bay and firmly connected.

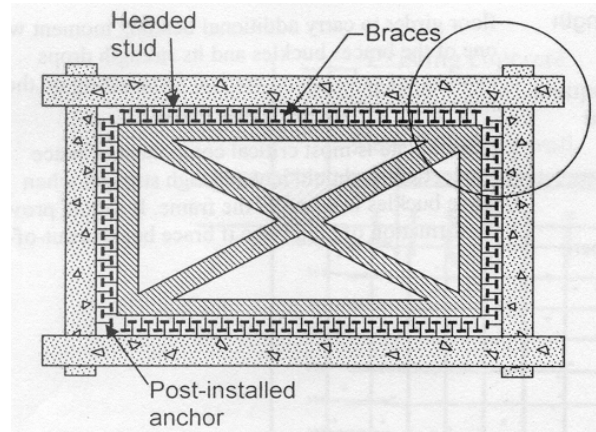


Figure 6(a): Prefabricated steel bracing

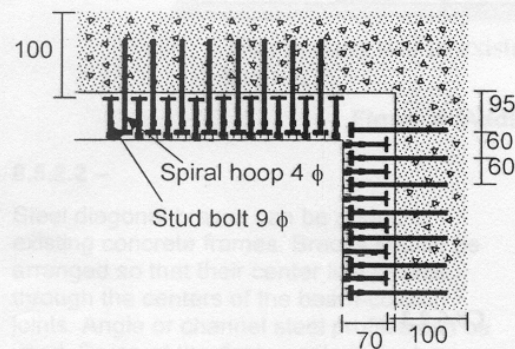


Figure 6(b): Detailing of corner view of figure 6(a)

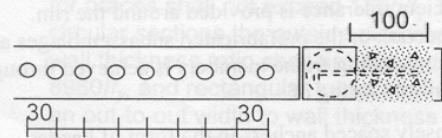


Figure 6(c): Detailing of top view of figure 6(a)

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Prepared by:

***Professor Anand S. Arya and Ankush Agarwal***

*under the GoI-UNDP Disaster Risk Management Programme*

Email: [anand.s.arya@undp.org](mailto:anand.s.arya@undp.org), [ankush.agarwal@undp.org](mailto:ankush.agarwal@undp.org)

**NATIONAL DISASTER MANAGEMENT DIVISION**

Ministry of Home Affairs, North Block, New Delhi

Tel: 91-11-23093178; Tele/fax: 23094019, Email: [ndmindia@nic.in](mailto:ndmindia@nic.in); Website: [www.ndmindia.nic.in](http://www.ndmindia.nic.in)