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IS 15988 (2013): Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings - Guidelines [CED 39: Earthquake Engineering]

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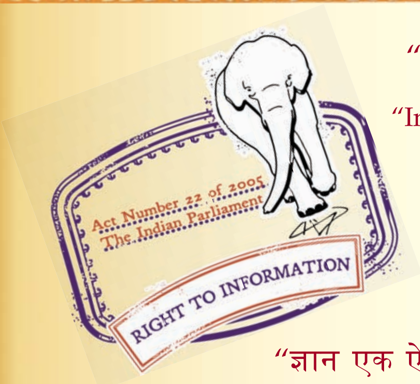

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“Knowledge is such a treasure which cannot be stolen”

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प्रबलित कंक्रीट के बने भवनों के भूकम्पीय मूल्यांकन और सुद ढ़ीकरण — दिशा निर्देश

Indian Standard

SEISMIC EVALUATION AND STRENGTHENING OF EXISTING REINFORCED CONCRETE BUILDINGS — GUIDELINES

ICS 91.120.25

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Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

This standard is intended to reduce the risk of death and injury that may result from the damaging effects of earthquake on building which predate the current seismic codes [IS 1893 (Part 1) : 2002 'Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings', IS 4326 : 1993 'Code of practice for earthquake resistant design and construction of buildings' and IS 13920 : 1993 'Ductile detailing of reinforced concrete structures subjected to seismic forces — Code of practice'] or have not been designed for earthquake forces.

This standard 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.

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 higher importance. Further, to account for uncertainty in the reliability of available information about the existing structure and the condition of structure components, strength calculations need to be suitably modified.

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.

In the formulation of this standard, assistance has been derived from the following publications:

ATC 33.03 Guidelines for seismic evaluation of existing buildings, Applied Technology Council, CA.

Eurocode 8 Design provisions for earthquake resistance of structures : Part 3, CEN, Brussels, 2001.

FEMA 178 NEHRP Handbook for the seismic evaluation of existing buildings, Building Seismic Safety Council, Washington, D.C., 1992.

FEMA 154 Rapid visual screening of buildings for potential seismic hazards: A Handbook, Fedral Emergency Management Agency, Washington DC, USA, 1998.

FEMA 310 Handbook for the seismic evaluation of buildings: A Prestandard, Federal Emergency Management Agency, Washington DC, USA, 20C.

FEMA 356 Prestandard and commentary for the seismic rehabilitation of building, Federal Emergency Management Agency, Washington DC, USA, 20C.

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.

ASCE 31-03 Seismic evaluation of existing buildings, American Society of Civil Engineers, Reston, VA, 2003.

ASCE 41-06 Seismic rehabilitation of existing buildings, American Society of Civil Engineers, Reston, VA,2006.

Seismic assessment and retrofit of reinforced concrete buildings, International Federation of structural Concrete (Fib), Laussance, Switzerland 2003.

Uniform code for building conservation, International Conference of Building Officials, Whittier, CA, USA, 1991.

Post-earthquake damage evaluation and strength assessment of buildings, under seismic conditions, Volume 4, UNDP/UNIDO, Vienna, 1985.

International existing building code (IBC), International Code Council, Illinois, 2006.

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Indian Standard

SEISMIC EVALUATION AND STRENGTHENING OF EXISTING REINFORCED CONCRETE BUILDINGS — GUIDELINES

1 SCOPE

1.1 This standard is particularly concerned with the seismic evaluation and strengthening of existing buildings and it is intended to be used as a guideline.

1.2 This standard 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 unfavourable characteristics of the building that could result in damage to either part of a building or the entire structure.

2 REFERENCES

The following standards contain provisions, which through reference in this text, constitute provisions of the standard. At the time of publication, the editions indicated were valid. All standards are subject to revision and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
1893 (Part 1) : 2002	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings
13920 : 1993	Ductile detailing of reinforced concrete structures subjected to seismic forces — Code of practice

3 TERMINOLOGY

For the purpose of this standard, the definitions given in IS 1893 (Part 1), IS 13920 and the following shall apply.

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

3.2 Action — An internal moment, shear, torque, axial load, developed in a member due to externally applied load/displacement on the structure.

3.3 Capacity — The permissible strength or deformation of a structural member or system.

3.4 Column (or Beam) Jacketing — A method in which a concrete column or beam is covered with a steel or reinforced concrete *jacket* in order to strengthen and/or repair the member by confining the concrete.

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

3.6 Deformation — Relative displacement or rotation at the ends of a component or element or node.

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

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

3.9 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. Diaphragms of wood construction and of similar material or elements which are not connected together for seismic loading are considered as flexible diaphragms. Cast-*in-situ* RC floor systems are usually not flexible diaphragms.

3.10 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. A panel in tight contact with a frame around its full perimeter is termed a shear infill.

3.11 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.

3.12 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.

3.13 Life Safety Performance Level — Building performance that includes significant damage to both structural and non-structural components during a design earthquake, where 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.

3.14 Load-Bearing Wall — A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load.

3.15 Load Path — The path that seismic forces acting anywhere in the building, take to the foundation of the structure and, finally, to the soil. Typically, the load travels from the diaphragm through connections to the vertical lateral-force-resisting elements, and then proceeds to the foundation.

3.16 Masonry — The assemblage of masonry units, mortar, and possibly grout and/or reinforcement. Types of masonry are classified herein with respect to the type of the masonry units, such as brick/clay-unit masonry or concrete masonry.

3.17 Non-structural Component — Architectural, mechanical or electrical components of a building that are permanently installed in, or are an integral part of a building.

3.18 Out-of-Plane Wall — A wall that resists lateral forces applied normal to its plane.

3.19 Overturning — An action resulting when the moment produced at the base of a vertical lateral-force-resisting element is larger than the resistance provided by the foundation's uplift resistance and building weight.

3.20 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.

3.21 Pounding — Two adjacent buildings impacting during earthquake excitation because they are too close together.

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

3.24 Redundancy — Provision of alternative load paths in a 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 pre-fabricated/pre-cast elements with adequate topping reinforced screed can be taken as rigid diaphragms.

3.27 Secondary Element — An element that does not affect the ability of the structure to resist earthquake-induced deformations. They may or may not actually resist any lateral force.

3.28 Seismic Demand — Seismic hazard level and commonly expressed in the form of a ground shaking response spectrum. Structural actions (force)/ deformation in members of the building are computed due to design earthquake.

3.29 Seismic Evaluation — An approved process or methodology of evaluating deficiencies in a building which prevent the building from achieving life safety objective.

3.30 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 (a) the direction of parapet, infill wall, etc, or (b) 50 percent of the nominal height of the typical columns at that level.

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

3.32 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.

3.33 Strengthening Method — A procedural methodology for the reduction of earthquake vulnerability of the building.

3.34 Strengthening Strategy — A technical approach for developing strengthening measures for a building to reduce its earthquake vulnerability.

3.35 Strong Column-Weak Beam — The capacity of the column in any moment frame joint must be greater than that of the beams, to ensure inelastic action in the beams.

3.36 Vertical Irregularity — A discontinuity of strength, stiffness, geometry, or mass in one storey with respect to adjacent stories.

4 SYMBOLS

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

A_c = total cross-sectional area of columns
A_g = gross area of the reinforced concrete section
A_s = steel to be provided in the jacket
A_{vf} = area of shear transfer reinforcement
A_{vf} = cross-section area of a single bar
A_w = area of shear wall
A_{wall} = total area of shear walls in the direction of loading
b_f = width of flange
d_h = diameter of stirrup
E_c = modulus of concrete
f_{ck} = characteristic strength of concrete
F_o = axial force due to overturning
f_y = yield strength of steel
H = total height
I_g = gross moment of intertia of reinforced concrete section
K = knowledge factor
L = length of the building
L_d = development length of bar in tension
M = moment
n_c = total number of columns
n_f = total number of frames in the direction of loading
P = axial load
P_{ac} = strength in axial compression
P_y = minimum yield strength in tension for the braces
t_f = thickness of flange
T_{rem} = remaining useful life of the building
T_{des} = design useful life of the building
τ_{col} = average shear stress in concrete columns
τ_{wall} = average shear stress in walls
t_j = thickness of jacket
U = useable life factor
μ = coefficient of friction
η = efficiency factor
V = total shear capacity of reinforced concrete beam
V_{con} = shear contribution of concrete
V_{FRP} = shear contribution of FRP sheet
V_B = base shear
V_s = shear force contribution of steel in a reinforced concrete beam
V_j = storey shear at level j
V_u = allowable shear force

5 EVALUATION CRITERIA

5.1 General

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 (Part 1) seismic force which is applicable for the seismic design of new buildings.

5.2 Since the provisions of this standard are strongly correlated with the design criteria of new buildings contained in IS 1893 (Part 1), reference shall always be made to the current edition of IS 1893 (Part 1). All existing structural elements must be able to carry full other non-seismic loads in accordance with the current applicable standards 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 (Part 1). Alternatively, a site-specific seismic design criteria developed along the principles described in IS 1893 (Part 1) may be used. Modification to seismic forces as given in IS 1893 (Part 1) and to material strengths will be applicable to both preliminary and detailed assessments described in this standard.

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 (Part 1). *U* will be determined as

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

where

T_{rem} = remaining useful life of the building; and
T_{des} = design useful life of the building.

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

NOTES

1 By comparing the requirements of the revisions of IS 1893 of 2002 with 1984, 1975, 1966 and 1962 revisions, 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 IS 1893 will in general not need retrofitting except those on stilts (soft first story) and those using 230 mm or thinner columns will need retrofitting.

3 Buildings designed to earlier code revisions of IS 1893 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 revisions of IS 1893, 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).

5.5 Modified Material Factor

Strength capacities of existing building components shall be based on the probable material strengths in the building. Probable or measured nominal strengths are best indicator of the actual strength and may only be obtained by field or lab tests on a series of samples. It is recommended that probable strengths are either based on actual tests or the default values given in the subsequent clauses. These may 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*

Sl No. (1)	Description of Building (2)	*K* (3)
i)	Original construction documents available, including post-construction activities, such as modification to structure or materials testing undertaken of existing structure	1.00
ii)	Documentation as in Sl No. (i) but no testing of materials, that is using originally specified values for materials	0.90
iii)	Documentation as in Sl No. (i) no testing of, that is originally specified values for materials and minor deterioration of original condition	0.80
iv)	Incomplete but useable original construction documents and no testing	0.70
v)	Incomplete or no documents available but extensive testing and inspection done to establish current strength of load resisting members	0.70
vi)	Documentation as in Sl No. (iv) and limited inspection, and verification of structural members, or materials test results with large variation	0.60
vii)	Little knowledge of details of a component	0.50

5.6 Evaluation Process

Existing buildings not designed in accordance with the principles and philosophies and requirements of current seismic standards as described in the following clauses shall be assessed.

5.6.1 A preliminary evaluation of building is carried out. This involves broad assessment of its 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.

NOTE — Single or two storey buildings (not housing essential services required for post-earthquake emergency response) of total floor areas less than 300 sq. m may be exempted from detailed evaluation even when a preliminary evaluation indicates deficiencies and where seismic retrofitting is carried out to remedy those deficiencies.

5.6.3 A detailed evaluation includes numerical checks on stability and integrity of the whole structure as well as the strength of each member. Conventional design calculations for these checks shall use modified demands and strengths**.** A flow diagram summarizing various steps of the evaluation process is shown in Fig. 1.

6 PRELIMINARY EVALUATION

6.1 General

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that may affect its seismic vulnerability. It is a very approximate procedure based on conservative parameters to identify the potential earthquake risk of a building and may 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 shall 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 walls, basement and foundation system.
c) Building type and site soil classification as in IS 1893 (Part 1).
d) Building use and nature of occupancy.
e) Adjacent buildings and potential for pounding and falling hazards.
f) *General conditions* — Deterioration of

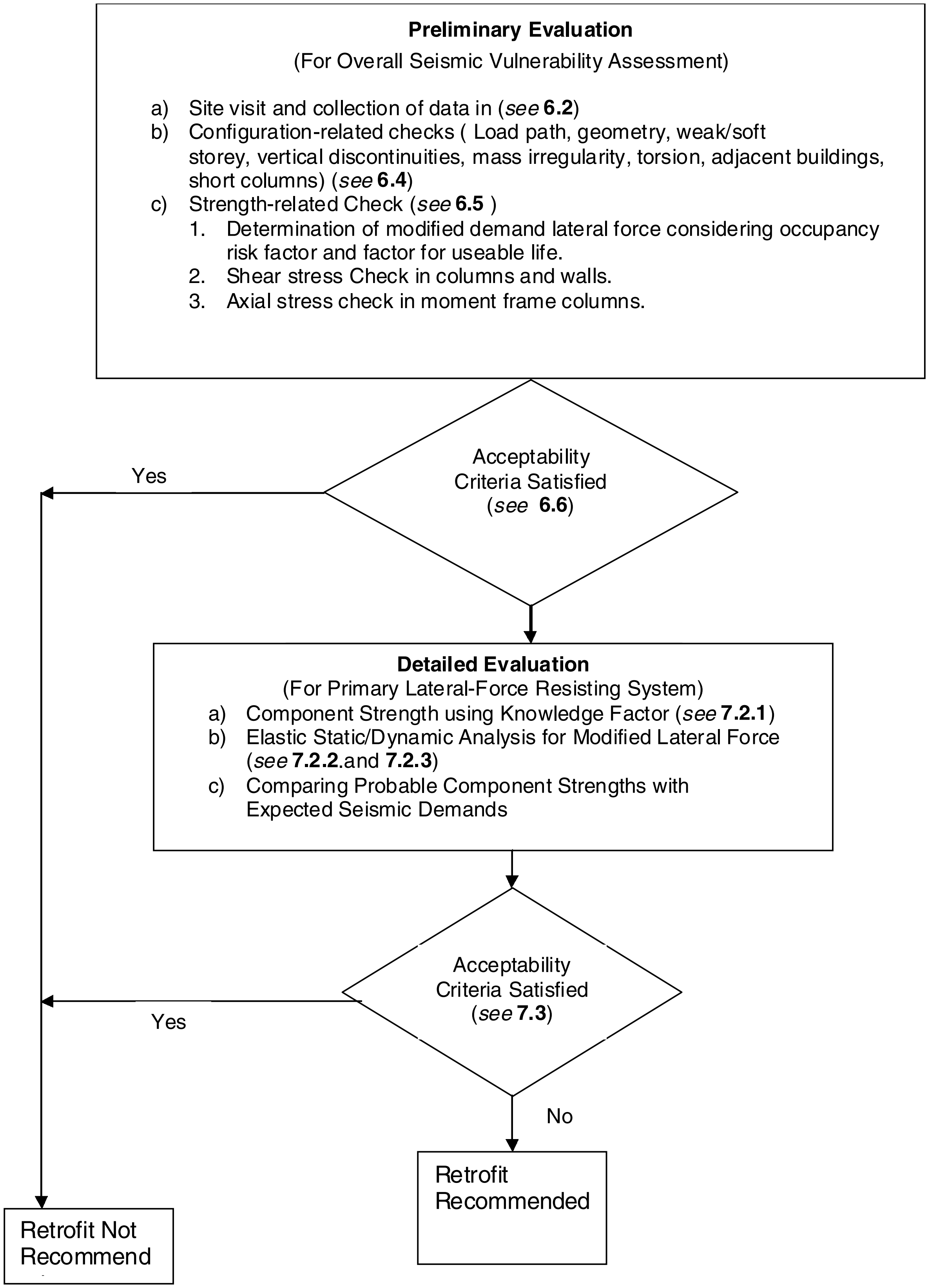


FIG. 1 FLOW CHART SUMMARIZING EVALUATION PROCESS

materials, damage from past earthquakes, alterations and additions that could affect earthquake performance.

g) Architectural features that may affect earthquake performance, especially location of masonry infill walls.

h) Geological site hazards and foundation conditions: Susceptibility for liquefaction and conditions for slope failure and surface fault rupture.

j) Special construction anomalies and conditions.

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 clauses.

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 may transfer all inertial forces in the building to the foundation.

6.4.2 *Redundancy*

The number of lines of vertical lateral load resisting elements in each principal direction shall be greater than or equal to 2. In the case of moment frames, the number of bays in each line shall be greater than or equal to 2. Similarly, the number of lines of shear walls in each direction shall be greater than or equal to 2.

6.4.3 *Geometry*

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

6.4.4 *Weak Storey*

The strength of the vertical lateral force resisting system in any storey shall not be less than 70 percent of the strength in an adjacent storey.

6.4.5 *Soft Storey*

The stiffness of vertical lateral load resisting system in any storey shall not be less than 60 percent of the stiffness in an adjacent storey or less than 70 percent of the average stiffness of the three storeys above.

6.4.6 *Vertical Discontinuities*

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

6.4.7 *Mass*

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

6.4.8 *Torsion*

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

6.4.9 *Adjacent Buildings*

The clear horizontal distance between the building under consideration and any adjacent building shall be greater than 4 percent of the height of the shorter building, except for buildings that are of the same height with floors located at the same levels. The gap width specified in **7.11.3** of IS 1893 (Part 1).

6.4.10 *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 percent of the nominal height of the typical columns in that storey.

6.4.11 *Mezzanines/Loft/Sub-floors*

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 (Part 1) and the requirements of **5**.

6.5.1 *Shear Stress in Reinforced Concrete Frame Columns*

The average shear stress in concrete columns, $\tau_{col,}$ computed in accordance with the following equation shall be lesser of,

a) 0.4 MPa; and

b) $0.10\sqrt{f_{ck}}$, f_{ck} is characteristic cube strength of concrete:

$$\tau_{col} = \left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right)$$

where

n_c = total number of columns;

n_f = total number of frames in the direction of loading;

V_j = storey shear at level j; and

A_c = total cross-sectional area of columns.

6.5.2 *Shear Stress in Shear Walls*

Average shear stress in concrete and masonry shear walls, $\tau_{wall,}$ shall be calculated as per the following equation:

$$\tau_{wall} = \left(\frac{V_j}{A_{wall}} \right)$$

where

V_j = storey shear at level j; and

A_{wall} = total area of shear walls in the direction of the loading.

NOTES

1 For concrete shear walls, τ_{wall} shall be less than 0.40 MPa.

2 For unreinforced masonry load bearing wall buildings, the average shear stress, τ_{wall} shall be less than 0.10 MPa.

6.5.3 *Shear Stress Check for Reinforced Concrete Masonry Infill Walls*

The shear stress in the reinforced masonry shear walls shall be less than 0.30 MPa and the shear stress in the unreinforced masonry shear walls shall be less than 0.10 MPa.

6.5.4 *Axial Stress in Moment Frames*

The maximum compressive axial stress in the columns of moment frames at base due to overturning forces alone (F_0) as calculated using the following equation shall be less than $0.25f_{ck.}$

$$F_0 = \frac{2}{3}\left(\frac{V_B}{n_f} \right)\left(\frac{H}{L} \right)$$

where

n_f = total number of frames in the direction of loading,

V_B = base shear,

H = total height, and

L = length of the building.

6.6 Recommendation for Detailed Evaluation

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

a) 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 15 km) active faults and/or with inadequate foundation details; and
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 General

The detailed evaluation procedure is based on determining the probable strength of lateral load resisting elements 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 K given in **5**. An assessment of the building for its present condition of its components and strength of materials is required. Further, seismic demand on critical individual components shall be determined using seismic analysis methods described in IS 1893 (Part 1) for lateral forces prescribed therein with modification for (reduced) useable life factor, described in **5**.

7.1.1 *Condition of the Building Components*

The building shall be checked for the existence of some of the following common indicators of deficiency:

a) *Deterioration of concrete* — There shall be no visible deterioration of the concrete or reinforcing steel in any of the vertical or lateral force resisting elements.
b) *Cracks in boundary columns* — There shall be no existing diagonal cracks wider than 3 mm in concrete columns that encase masonry infills.
c) *Masonry units* — There shall be no visible deterioration of masonry units.
d) *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.
e) *Cracks in infill walls* — There shall be no existing diagonal cracks in infill walls that extend throughout a panel, are greater than 3 mm, or have out-of-plane offsets in the bed joint greater than 3 mm.

7.1.2 *Condition of the Building Materials*

An evaluation of the present day strength of materials shall be performed using on-site non-destructive testing and laboratory analysis of samples taken from the building. Field tests are usually indicative tests and therefore shall be supplemented with proper laboratory facilities for accurate quantitative results.

7.2 Evaluation Procedure

The key steps of this evaluation procedure are as follows:

7.2.1 *Probable Flexure and Shear Demand and Capacity*

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 modified with knowledge factor K.

7.2.2 *Design Base Shear*

Calculate the total lateral force (design base shear) in accordance with [IS 1893 (Part 1)] and multiply it with U, a factor for the reduced useable life (equal to 0.70).

7.2.3 *Analysis Procedure*

Perform a linear equivalent static or a dynamic analysis of the lateral load resisting system of the building in accordance with IS 1893 (Part 1) for the modified base shear determined in the previous step and determine resulting member actions for critical components.

a) *Mathematical model* — Mathematical model of the physical structure shall be such as to represent the spatial distribution of mass and stiffness of the structure to an extent that is adequate for the calculation of significant features of its distribution of lateral forces. All concrete as well as masonry elements shall be included in the model.
b) *Component stiffness* — Component stiffness shall be determined based on some rational procedure. Some standard values are given in Table 2.

7.2.4 *Demand-Capacity Ratio*

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

7.2.5 *Inter-storey Drift*

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

7.3 Acceptability Criteria

A building is said to be acceptable if either of the following two conditions are satisfied along with supplemental criteria for a particular building type described in **7.4**:

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 shall 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.

7.4 Ductility and Detailing Related Evaluation

In addition to the general evaluation (*see* **7.2**) for buildings which addresses only strength issues more criteria need to be considered which relate to ductility and detailing of structural components. These criteria address certain special features affecting the lateral load-behaviour which are specific to each building type.

7.4.1 *Moment Resisting Reinforced Concrete Frame Buildings*

For RC moment frame buildings designed using response reduction factor R [*see* IS 1893 (Part 1)] equal to 5 the following supplemental criteria need to be satisfied. Any deficiency should be considered in suitably reducing the value of R.

Table 2 Some Effective Stiffness Values
(*Clause* 7.2.3)

Sl No. (1)	Component (2)	Flexural Rigidity (3)	Shear Rigidity (4)	Axial Rigidity (5)
i)	Beam, non pre-stressed	$0.5\ E_c\ I_g$	—	—
ii)	Beam, pre-stressed	$1.0\ E_c\ I_g$	—	$E_c\ A_g$
iii)	Column in compression($P > 0.5f_c'A_g$)	$0.7\ E_c\ I_g$	$0.4\ E_c\ A_w$	$E_c\ A_g$
iv)	Column in compression ($P \geq 0.5f_c'A_g$)	$0.5\ E_c\ I_g$	—	$E_c\ A_g$
v)	Walls — Uncracked	$0.8\ E_c\ I_g$	—	$E_c\ A_g$
vi)	Walls — Cracked	$0.5\ E_c\ I_g$	—	$E_c\ A_g$
vii)	Flat slab	To be determined based on rational procedure		

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) *Concrete columns* — All concrete columns shall be adequately anchored into the foundation from top face of pedestal of base slab.
c) *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 at each frame joint.
d) *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 percent of the longitudinal bars located at the joints for either positive or negative moment shall be continuous throughout the length of the members.
e) *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.
f) *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 (1) within a joint; (2) within a distance of *2d* from joint face; and (3) within a quarter length of the member near supports 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.
g) *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 cross tie should be there. Alternatively, a pair of overlapping hoops may be located within the column. The hooks shall engage peripheral longitudinal bars.
h) *Stirrup spacing*—The spacing of stirrups over a length of *2d* at either end of a beam shall not exceed (1) *d/4,* or (2) 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*.
j) *Joint reinforcing*— Beam-column joints shall have ties spaced at or less than 150 mm.
k) *Stirrup and tie hooks* — The beam stirrups and column ties shall preferably be anchored into the member cores with hooks of 135°.

7.4.2 *Concrete Shear Wall Buildings*

Concrete shear wall buildings can be either the ordinary reinforced type or ductile shear wall type. Some of the provisions mentioned below are applicable to both types of shear walls while some are applicable only for ductile shear walls. Applicable provisions shall indicate the suitable choice for the response reduction factor *R*.

7.4.2.1 *Thickness*

The thickness of any part of an ordinary shear wall shall preferably, not be less than 100 mm while for ductile shear wall it shall not be less than 150 mm. In case of coupled shear walls, the thickness of the walls shall be at least 200 mm.

7.4.2.2 *Overturning*

All shear walls shall have aspect ratio less than 4 to 1, else the foundation system shall be investigated for its adequacy to resist overturning moments. Wall piers need not be considered.

7.4.2.3 *Reinforcement*

a) Shear walls shall be provided with reinforcement in the longitudinal and transverse directions in the plane of the wall to resist bending moment and to prevent premature shear failure. The minimum reinforcement ratio for ordinary shear walls shall be 0.001 5 of the gross area in each direction. For ductile shear walls this value is increased to 0.002 5 in the horizontal direction. This reinforcement shall be distributed uniformly across the cross-section of the wall.
b) The stirrups in all coupling beams over openings for doors, passages, staircases, etc, shall be spaced at or less than *d/2* and shall

be anchored into the core with hooks of 135° or more. The shear and flexural demand on coupling beams which are non-compliant are calculated using analysis procedure of **7.2** and their adequacy is checked. If they are found inadequate then their adequacy is checked as if they were independent.

7.4.2.4 *Opening in walls*

Total length of openings shall not be greater than 75 percent of the length of any perimeter wall.

The adequacy of remaining wall for shear and overturning resistances shall be evaluated according to **7.2**. Shear transfer connection between the diaphragm and walls shall also be evaluated and checked for adequacy.

7.4.3 *Reinforced Concrete Frames with Masonry Infill Walls*

The provisions of **7.4.1** also apply to reinforced concrete frames with masonry infill walls. In addition, the infill walls shall be checked for the following additional criteria:

a) *Wall connections* — All infill walls shall have a positive connection to the frame to resist out-of-plane forces.
b) *Out of plane stability* — The unreinforced masonry wall height-to-thickness ratios shall be less than as given in Table 3. The frame element beams are assumed to provide necessary lateral support for the unreinforced masonry wall in out-of-plane direction.

Table 3 Allowable Height-to-Thickness Ratios of Unreinforced Masonry Walls

Sl No.	Wall Type	Zone II and III	Zone IV	Zone V
(1)	(2)	(3)	(4)	(5)
i)	Top storey of multi-storey building	14	14	9
ii)	First storey of multi-storey building	18	16	15
iii)	All other conditions	16	16	13

c) *Unreinforced masonry parapets* — The maximum height of an unsupported unreinforced masonry parapet shall not exceed the height-to-thickness ratio as shown in Table 4. If the required parapet height exceeds this maximum height, a bracing system designed for the forces determined as per non-structural elements specified in **8.5.2.2**, shall support the top of the parapet. The minimum height of a parapet above any wall anchor shall be 300 mm. If a reinforced concrete beam is provided at the top of the wall, the minimum height above the wall anchor may be 150 mm.

Table 4 Maximum Allowable *h/t* Ratio for Parapets

Unreinforced Masonry Parapets	Zone V	All Other Zones
Maximum allowable height-to-thickness ratio	1.5	2.5

8 SEISMIC STRENGTHENING

8.1 General

This clause 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 given in **7**.

8.2 Seismic Strengthening Options and Strategies

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

8.2.1 *Strengthening at Member Level*

a) 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.
b) Member level modification shall be undertaken to improve strength, stiffness and/or ductility of deficient members and their connections strengthening measures shall include such as jacketing columns or beams.
c) 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 reinforced concrete columns, which improves the member level ductility by increased confinement.

8.2.2 *Eliminating or Reducing Structural Irregularities*

a) 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.
b) 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 may also be effectively used to balance stiffness and mass distribution within a storey to reduce torsional irregularities. Shear wall shall be placed such that it forms an integral part of load flow path for lateral loads. Minimum two shear wall shall be constructed in each orthogonal direction in opposite side of shear centre away from centre as far as possible to add better torsional resistance to the entire structure. The stiffness centre of the complete structure at a floor level after adding shear wall shall be such that eccentricity with respect to centre of gravity of mass is reduced to a minimum.
c) Seismic gaps (or movement joints) shall be created between various parts of a building with irregular plan geometry to separate it into a number of regular independent structures. However, care shall be exercised to provide sufficiently wide gaps to avoid the problem of pounding.

8.2.3 *Strengthening at Structural Level*

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 shall 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

8.3.1 *Supplemental Damping and Isolation*

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

Energy dissipation helps in the overall reduction in displacements of the structure.

This technique is most effective in structures that are relatively flexible and have some inelastic deformation capacity.

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 **5**.

8.4.2 *Member Capacities*

Member capacities of existing elements shall be based on the probable strengths as defined in **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 Reinforced Concrete Framed Structures

8.5.1 *Jacketing*

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

NOTES

1 Reinforced concrete jacketing involves placement of new longitudinal reinforcement and transverse reinforcement bars in the new concrete overlay around existing member.

2 Steel profile jacketing shall 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.

3 Retrofitting using FRPs involves placement of composite material made of continuous fibres with resin impregnation on the outer surface of the reinforced concrete member.

8.5.1.1 *Reinforced concrete 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 as follows:

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 to obtain the amount of concrete and steel to be provided in the jacket.

d) The extra size of column cross-section and reinforcement is provided in the jacket.

e) Increase the amount of concrete and steel actually to be provided as follows to account for losses. $A_c = (3/2)A_c'$ and $A_s = (4/3)A_s'$

where

A_c and A_s = actual concrete and steel to be provided in the jacket; and

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, and

t_j = thickness of jacket.

g) If the transfer of axial load to new longitudinal steel is not critical then friction present at the interface shall be relied on for the shear transfer, which shall be enhanced by roughening the old surface.

h) Dowels which are epoxy grouted and bent into 90° hook shall also be employed to improve the anchorage of new concrete jacket.

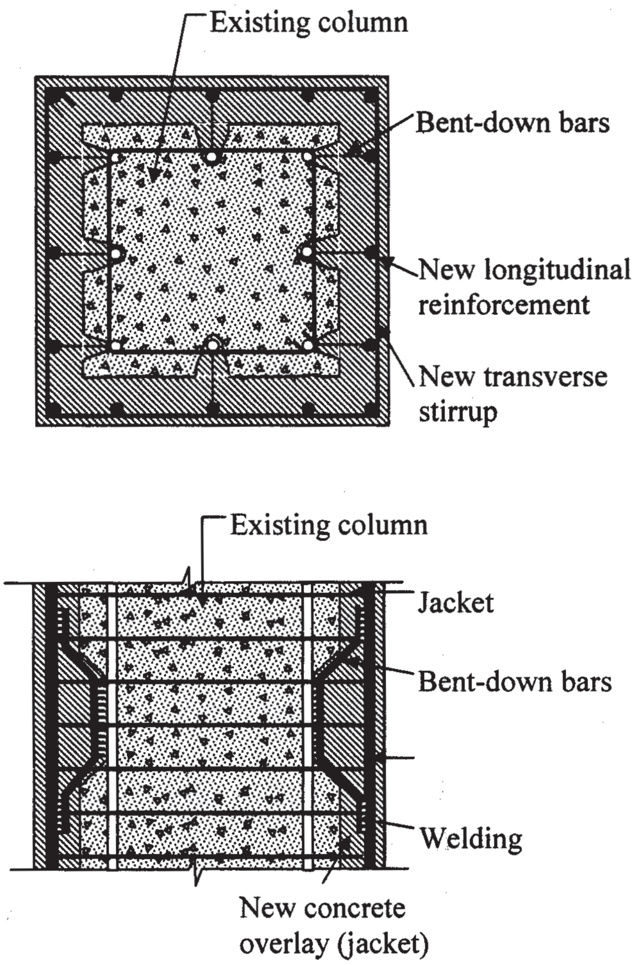


FIG. 2 REINFORCED CONCRETE JACKETING

8.5.1.2 The minimum specifications for jacketing columns are:

a) Strength of the new materials shall be equal or greater than those of the existing column. Concrete strength shall 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 12ϕ bars in the four corners and ties of 8ϕ @ 100 c/c should be provided with 135° bends and 10ϕ leg lengths.
c) Minimum jacket thickness shall be 100 mm.
d) Lateral support to all the longitudinal bars shall be provided by ties with an included angle of not more than 135°.
e) Minimum diameter of ties shall be 8 mm and not less than one-third 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 ¼ of the clear height shall not exceed 100 mm. Preferably, the spacing of ties shall not exceed the thickness of the jacket or 200 mm whichever is less.

8.5.1.3 *Fibre 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 flexure strength is determined based on the assumption that compressive concrete reaches a strain of 0.003 5 and FRP reaches its maximum strain.

Shear strength of a beam after strengthening:

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

where

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

$V_s = 0.87 \times f_y \times A_{sv} \times (d/s_v)$

$V_{FRP} = A_f f_f \left(\frac{d}{s}\right)$

V_{con} = shear contribution of concrete;

V_s = shear contribution of steel; and

V_{FRP} = 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 its lateral force capacity. Shear walls and steel bracing shall be added as new elements to increase the strength and stiffness of the structure.

8.5.2.1 *Addition of reinforced concrete shear wall*

Addition of new reinforced concrete shear walls provides the best 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) Where vertical shear walls are inserted between existing columns shear transfer reinforcement (dowel bars), perpendicular to the shear plane, is given by:

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

where

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

μ = 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; and

η = efficiency factor = 0.5

b) The number of bars required for resisting shear at the interface are given by:

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

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 (*see* Fig. 3).
d) Wherever thickness of column is 250 mm or less, shear wall shall encase the column by wrapping shear wall reinforcement around column after roughening reinforced concrete column surface. In case where shear wall spans perpendicular to the larger dimension of column, the transverse reinforcement of shear wall shall be anchored and wrapped around the column surface as shown in the sketch.

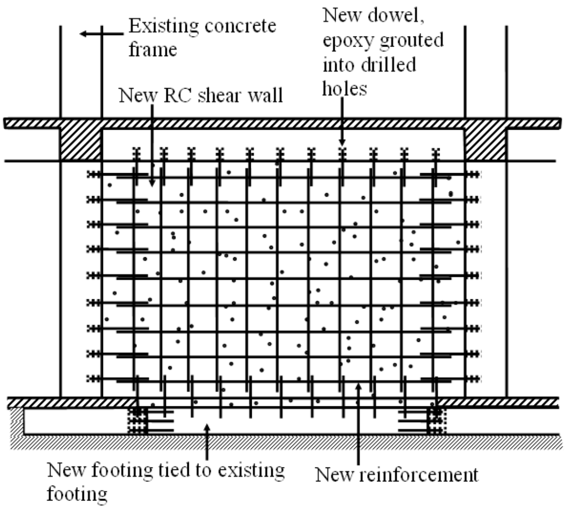


FIG. 3 ADDING NEW SHEAR WALLS

8.5.2.2 *Addition of steel bracing*

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

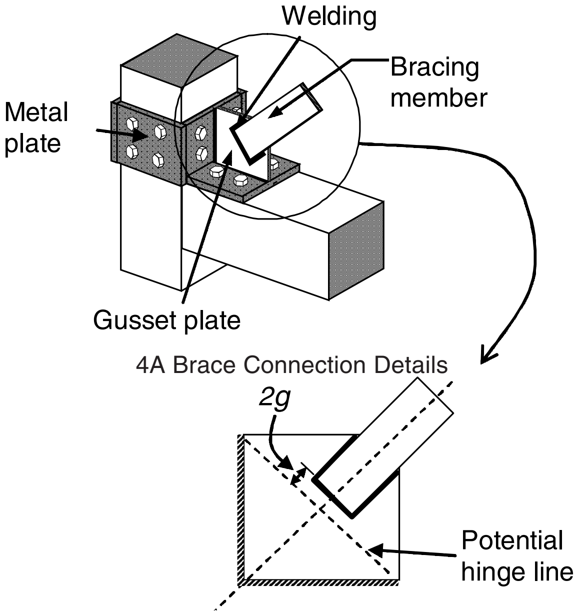
a) Slenderness of bracing member shall be less or equal to 2 500/$\sqrt{f_y}$.

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 8 960/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 (inverted-V) braces, the beam intersected by braces shall have adequate 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 percent of the beam flange strength $f_y b_f t_f$.

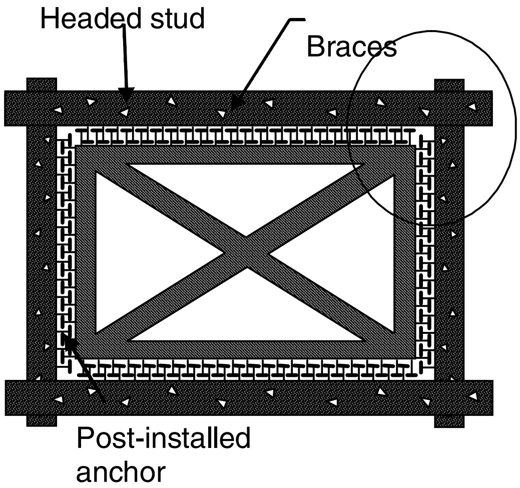
e) The brace connection shall be adequate against out-of-plane failure and brittle fracture. Typical connection detail is shown in Fig. 4.



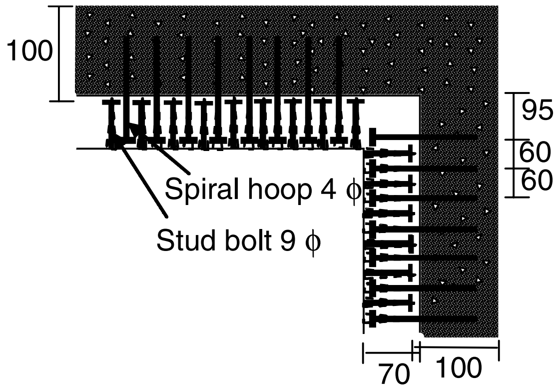
4B Gusset Base Connection Detail

FIG. 4 BRACE CONNECTIONS

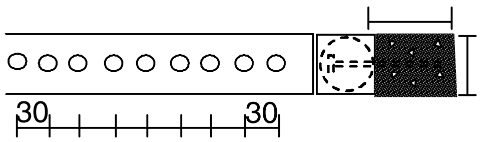
8.5.2.3 Pre-fabricated steel bracing sub assemblages as shown in Fig. 5 may be used, for ease of construction, Braces in X-, V-and inverted V-shall be arranged inside a heavy rectangular steel frame, which is then placed in frame bay and firmly connected.



5A Pre-fabricated Steel Bracing



5B Detailing of Corner View of Fig. 5A



5C Detailing of Corner View of Fig. 5A

FIG. 5 DETAILING OF PRE-FABRICATED STEEL BRACING

ANNEX A
(*Foreword*)

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(*Continued from second cover*)

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The composition of the Committee responsible for the formulation of this standard is given in Annex A.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

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Amendments Issued Since Publication

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