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Draft Final Report: A - Earthquake Codes
IITK-GSDMA Project on Building Codes

Seismic Evaluation and Strengthening of Existing Buildings

by

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IITK-GSDMA GUIDELINES

for SEISMIC EVALUATION AND STRENGTHENING OF BUILDINGS

Provisions with Commentary and Explanatory Examples

Indian Institute of Technology Kanpur

Gujarat State Disaster Mitigation Authority

August 2005

Other IITK-GSDMA Guidelines:

- IITK-GSDMA Guidelines for Seismic Design of Liquid Storage Tanks
- IITK-GSDMA Guidelines for Structural Use of Reinforced Masonry
- IITK-GSDMA Guidelines for Seismic Design of Earth Dams and Embankments

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FOREWORD
(Jointly by IITK and GSDMA)

PREFACE

Current Indian codes do not address the evaluation of seismic resistance of existing building stock, which may not have been designed for earthquake forces. Further, adequate codal provisions are lacking for strengthening of the structural systems of seismically deficient buildings. An appropriate level of safety needs to be ensured for the occupants of these buildings. The present guidelines are intended to provide a systematic procedure for the seismic evaluation of buildings which can be applied consistently to a rather wide range of buildings. Though not applicable to all building types, the document also discusses some cost-effective strengthening schemes for existing older buildings identified as seismically deficient during the evaluation process.

These guidelines have been developed based on the experience gained over many years of practice of seismic strengthening of buildings in various seismically active countries of the world. Their codes of practices, standards and guidelines have been extensively referred in developing these guidelines. The seismic evaluation procedure presented in these guidelines is derived from such documents as ATC 40, FEMA 310, FEMA 273 (now FEMA 356), UCBC (now GSREB) of ICBO, New Zealand Draft Code, and Eurocode.

Special emphasis is placed on two classes of most hazardous buildings: (a) unreinforced masonry (URM) bearing wall buildings and (b) non-ductile reinforced concrete (RC) frame buildings. The ABK (Agbabian-Barnes-Kariotis) methodology has been especially included for potentially hazardous URM buildings with flexible diaphragms for effective and cost-efficient strengthening options.

It is hoped that these guidelines which reflect a prescriptive engineering approach will lead to more performance oriented procedures with the experience gained from their usage. Further, these may include additional techniques and options for strengthening a wider variety of building stock.

The document has been reviewed by Prof. A Meher Prasad (IIT Madras), Prof. Sudhir K Jain (IIT Kanpur), Mr. Jitendra Bothara (NSET-Nepal) and Prof. Andrew Charleson (Victoria University of Wellington). Prof. Charleson provided most extensive and thorough review and offered valuable suggestions for improvement. Ms Ami Patwa and Mr Ankur Singh assisted in research and analysis of a large number of documents available on the subject. Also, they, along with Mr Samaresh Paikara and Mr Kamlesh Kumar, assisted in developing explanatory examples for the guidelines.

An earlier version of this document has been discussed in an e-conference hosted by Structural Engineers Forum of India (www.sefindia.org) during August 9-21, 2004, where in many practicing engineers and professionals from academia and industry expressed their views and comments. The material contained in these documents was also presented in workshops held in Ahmedabad and New Delhi for practicing structural engineers. The feedback received on these occasions has been appropriately addressed while revising the document.

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INDIAN INSTITUTE OF TECHNOLOGY KANPUR
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PART 2: EXPLANATORY EXAMPLES

Example No.	Title	Description	Page No.
1.	*Seismic evaluation of a RC moment resisting frame building*	A five-storey RC building with lateral load moment-resisting frame at the perimeter is checked for seismic adequacy. Preliminary evaluation steps including configuration and strength related checks are illustrated. Detailed evaluation of one frame is illustrated, including calculation of earthquake forces, lateral load analysis of frames, and demand/capacity calculation of members.	81-91
2.	*Seismic retrofit with limited replacement/modification of frame members*	A seismic retrofit of a five-storey RC building with deficient perimeter lateral load resisting frames (of Example 1) is illustrated. The retrofit strategy involves the modification/limited replacement of perimeter frame members of required strength and ductility.	92-100
3.	*Seismic retrofit with the addition of new shear walls to an existing frame*	Design of new RC shear walls on the perimeter of a seismically deficient five-storey building (of Example 1) is illustrated as a retrofit option.	101-105
4.	*Seismic evaluation and retrofit of unreinforced masonry building with flexible diaphragms*	Seismic evaluation of a four-storey unreinforced masonry building with flexible diaphragms is illustrated. Only a detailed evaluation using the new *special procedure* is presented. It accounts for the flexibility of diaphragms in the lateral load analysis. Two retrofit options are illustrated. They involve the provision of new cross-walls to control deformation of diaphragm and the addition of new braced-frame to increase the in-plane shear resistance of end-walls weakened by large openings.	106-120

IITK-GSDMA GUIDELINES
for SEISMIC EVALUATION
and STRENGTHENING
of EXISTING BUILDINGS

PART 1: PROVISIONS AND COMMENTARY

PROVISIONS

1. General

1.1 Purpose

1.1.1

This document is developed as part of project entitled "Review of Building Codes and Preparation of Commentary and Handbooks" awarded to Indian Institute of Technology Kanpur by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank finances.

1.2 Scope

1.2.1 –

This document is particularly concerned with the seismic evaluation and strengthening of existing buildings and it is intended to be used as a guide. It is not expected to be definitive in any situation.

1.2.2 –

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 result in damage to either part of a building or the entire structure.

1.2.3 –

Evaluation of the seismic performance of an existing building is done in relation to the provisions of IS 1893 (Part1), which is applicable to a new building. Modifications for load, materials and other features of the existing building are incorporated into the provisions of this code for the evaluation purpose.

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C1.2.1 –

These guidelines are based on international experience primarily of countries, like New Zealand, USA and Japan etc. The subject is undergoing more refinement and further development as more experienced is continuously gained with real life applications.

C1.2.2 –

The objective of the provisions of this document to reduce risk of loss life and injury. This is accomplished by limiting the likelihood of damage and controlling the extent of damage. Emphasis is on identifying the most vulnerable elements of the buildings at risk, and strengthening them with priority given to reducing the risk of partial or complete collapse. However, these provisions do not necessarily prevent earthquake damage in strengthened buildings; rather they improve structural performance in earthquakes. For simplicity the word 'strengthening' which means same as 'retrofitting' or improving the seismic performance, is used in the document.

C1.2.3 -

The relationship between the design of a new building and the evaluation of an existing building is depicted in Figure C1. The key concept lies in the fact that the loadings standard and materials standard are common to both of them, however these standards are suitably modified to account for and the present strength of materials and the remaining useful life.

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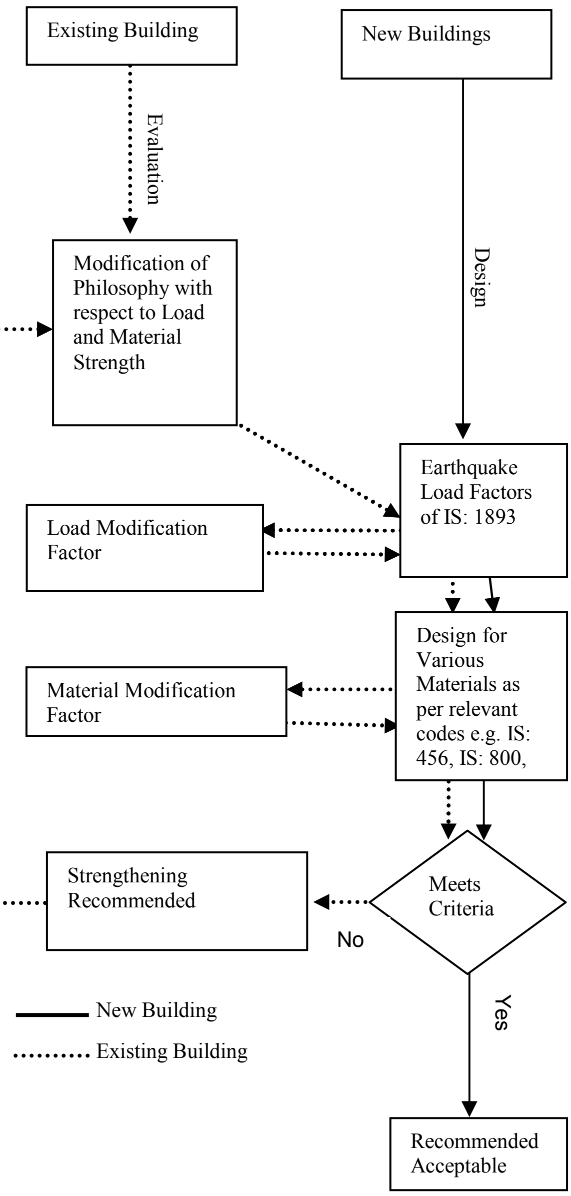

Figure C1. The relationship between the procedure for the design of new buildings and the evaluation of existing buildings

PROVISIONS **COMMENTARY**

2. – References

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

1. ABK – Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: The Methodology, Topical Report 08, National Science Foundation, Applied Science and Research Applications, Washington, D.C., 1984.
2. ASCE 31-03 – Seismic Evaluation of Existing Buildings, American Society of Civil Engineers, 2003.
3. ATC 14 – Evaluating the Seismic Resistance of Existing Buildings, Applied Technology Council, CA, 1987.
4. ATC 33.03 – Guidelines for Seismic Evaluation of Existing Buildings, (75% complete draft), Applied Technology Council, CA.
5. ATC 40 – Seismic Evaluation and Retrofit of Concrete Buildings, Applied Technology Council, CA, 1986.
6. Eurocode 8 – Design Provisions for Earthquake Resistance of Structures-Part 3, CEN, Brussels, 2001.
7. FEMA 172 – NEHRP Handbook of Techniques for Seismic Rehabilitation of Existing Buildings, Building Seismic Safety Council, Washington, D. C, 1992
8. FEMA 178 – NEHRP Handbook for the seismic Evaluation of Existing Buildings, Building Seismic Safety Council, Washington, D. C, 1992.
9. FEMA 154 – Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook, Federal Emergency Management Agency, Washington DC, USA, 1988.
10. FEMA 310 – Handbook for the Seismic Evaluation of Buildings – A Prestandard, Federal Emergency Management Agency, Washington DC, USA, 1998.
11. FEMA 356 – Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington DC, USA, 2000.

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12. The Assessment and Improvement of the Structural Performance of Earthquake Risk Buildings – Draft for General Release, New Zealand National Society for Earthquake Engineering, for the Building Industry Authority, New Zealand, 1996.
13. Post-Earthquake Damage Evaluation and Strength Assessment of Buildings under Seismic Conditions, Volume 4, UNDP/UNIDO, Vienna, 1985.
14. Seismic Evaluation of Existing Buildings, American Society of Civil Engineers, USA, 2003.
15. Uniform Code for Building Conservation, International Conference of Building Officials, Whittier, CA, USA, 1991.
16. Guidelines for Seismic Retrofit of Existing Buildings, International Council of Building Officials, USA, 2003.
17. International Existing Building Code, ICC, Illinois, 2003.
18. Seismic Assessment and Retrofit of Reinforced Concrete Buildings, International Federation of Structural Concrete (fib), Laussane, Switzerland 2003

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3. – Terminology

For the purpose of this document, the definitions given in IS 1893(Part I)[1] 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– Chord

Chord is a line of edge beams that is connected to the floor, or reinforcing in the edge of a slab or in a spandrel which is assumed to withstand flexural stress.

3.5– Collector

A structural element used to transfer the shears or drag the forces from the diaphragm into the shear walls or bracing members. It is also referred as drag tie.

3.6– Column (or Beam) Jacketing

A method in which a concrete column or beam is encased in a steel or reinforced concrete *jacket* in order to strengthen and/or repair the member.

3.7– Components

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

[1] *Criteria for Earthquake Resistant Design of Structures – Part I General Provisions and Buildings*

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3.8– Concrete Masonry

Masonry constructed with solid or hollow units made of concrete. Hollow concrete units may be ungrouted, or grouted.

3.9– Deformation

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

3.10– Demand

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

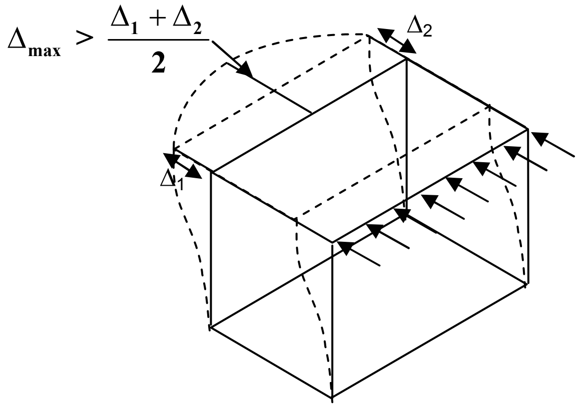
3.11– Displacement

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

3.12– 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.



Deflections are somewhat complex to calculate and at the best only an approximation can be made. Refer to the commentary in 3.30 for rigid diaphragms.

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3.13– Drag strut/tie

Also referred to as collectors, see 3.6.

3.14– 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*.

3.15– 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.

3.16 – 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.

3.17 – Load-bearing Wall

A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load.

3.18 – 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, load travel from the diaphragms through connections to the vertical lateral-force-resisting elements, and then proceeds to the foundation.

3.19 – 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.

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3.20 – Non-structural Component

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

3.21 – Out-of-plane Wall

A wall that resists lateral forces applied normal to its plane.

3.22 – 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.

3.23 – 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.

3.24 – Pounding

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

3.25 – Primary Element

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

3.26 – Probable or Measured Nominal Strength

The strength of a structure or a 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) strength field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

3.27 – Redundancy

Provision of alternative load paths in a

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

3.28 – Reinforced Masonry (RM) Shear Wall

A masonry wall that is reinforced in both the vertical and horizontal directions with minimum reinforcement as defined in *IITK-GSDMA Guidelines of Structural Use of Reinforced Masonry*. Reinforced walls are assumed to resist loads through resistance of the masonry in compression and the reinforcing steel in tension and compression. Reinforced masonry is partially grouted or fully grouted.

3.29 – 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.

3.30 – Rigid Diaphragm

Refer to flexible diaphragm .A floor diaphragm shall be considered to be rigid, if it is not flexible. Reinforced concrete monolithic slab-beam floors or those consisting of prefabricated/precast elements with adequate topping reinforced screed are usually rigid diaphragms. Also refer to the commentary for flexible diaphragm in 3.12.

3.31 – 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.

3.32 – 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 is computed due to design earthquake.

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3.33 – Seismic Evaluation

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

3.34 – 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% of the nominal height of the typical columns at that level.

3.35 – Strength

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

3.36 – 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.

3.37 – Strengthening Method

A procedural methodology for the reduction of building earthquake vulnerability.

3.38 – Strengthening Strategy

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

3.39 – 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.

3.40 – Vertical Irregularity

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

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4. – List of Notations

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_f	Total cross-sectional area of fiber sheet
A_n	Area of net mortared/grouted section
A_{st}	Total cross-sectional area of longitudinal reinforcement in a beam
A_w	Total area of shear walls in the direction of loading
C_p	Horizontal force factor
D	In-plane depth dimension of masonry pier;
D_d	Depth of the diaphragm parallel to the direction of applied forces being considered.
E_m	Elastic modulus of masonry
E_d	Elastic modulus of diaphragm material
F_F	Total force in fiber sheet wrapped around a RC member
F_o	Axial force due to overturning
F_R	Rupture strength of FRP
F_{px}	Force applied on a diaphragm
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 pier
k	Stiffness
$M_{u,lim}$	Design yield strength of RC beam
P_{CE}	Expected gravity compressive force applied to a wall or pier.
P_D	Superimposed dead load at the top of the pier under consideration
P_W	Weight of wall
S	Section modulus of base of a masonry wall
f_{ck}	Characteristic strength of concrete in MPa
f_f	Maximum stress in fiber sheet at rupture in MPa
f_t	Permissible tensile stress in masonry wall MPa
f_b	Permissible compressive stress in masonry in MPa
H	Unsupported height of unreinforced masonry wall
K	Knowledge factor
U	(Reduced) Useable life factor
m	Modular ratio
n_c	Total number of columns
n_f	Total number of frames
ε_s	Strain in longitudinal steel
ε_f	Strain in fiber sheet wrapped around RC member
t	Thickness of wall
t_f	Thickness of FRP sheet
τ_{col}	Average shear stress in concrete columns
τ_{wall}	Average shear stress in walls
v_a	Expected masonry shear strength
v_{te}	Average bed-joint shear strength
v_u	Unit shear strength of the diaphragm
w	Uniform lateral load on a diaphragm
w_f	Width of FRP sheet used for flexure
w_s	Width of FRP sheet for shear strength
w_{px}	Weight of a roof at level x
Δ_d	Deflection of a diaphragm
ΣQ_i	Lateral load at the *i*-th floor
Σw_i	Seismic weight at the *i*-th floor
$\Sigma v_u D$	Sum of the diaphragm shear capacities of both ends of the diaphragm

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T_s	Total force in longitudinal steel
V	Total shear capacity of RC beam
V_s	Shear force contribution of steel in a RC beam
V_{con}	Shear force contribution of concrete in FRP retrofitted beam
V_{FRP}	Shear force contribution of fiber sheet in a FRP retrofitted beam
V_a	Shear strength of an unreinforced masonry pier
V_B	Base shear
V_c	Column shear force
V_{ca}	Total shear capacity of walls in the direction of analysis immediately above the diaphragm level being investigated
V_{cb}	Total shear capacity of walls in the direction of analysis immediately below the diaphragm level being investigated
V_d	Diaphragm shear
V_j	Storey shear at level j
V_p	Shear force on an unreinforced masonry wall pier
V_r	Rocking shear capacity of an unreinforced masonry wall or wall pier
V_{wx}	Total shear force resisted by a shear wall at the level under consideration
W_d	Total dead load tributary to a diaphragm
W_w	Total dead load of an unreinforced masonry wall above the level under consideration or above an open front of a building
W_{wx}	Dead load of an unreinforced masonry wall assigned to level x halfway above and below the level under consideration
$\Sigma\Sigma v_u D$	For diaphragms coupled with cross walls, $v_u D$ includes the sum of the

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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, which is applicable for the seismic design of new buildings. Reference shall always be made to the current edition of IS: 1893All 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.

5.2 –

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.

5.3 – Load Modifying Factor

The lateral force determined for strength related checks needs to be modified for reduced useable life. The useable life factor *U,* is taken as 0.67, is to be multiplied to the lateral force (base shear) for new building as specified in IS: 1893.

COMMENTARY

C5. – Evaluation Criteria

C5.1 – General

The main purpose of these guidelines is to reduce loss of life and injury to inhabituating buildings which are constructed without earthquake resistant features. Such buildings contain unfavorable configuration, inadequate strength and/or poor detailing; therefore it becomes necessary to undertake seismic rehabilitation. These guidelines are also expected to serve as a standard procedure for instructing design professionals on how to determine the adequacy of existing buildings to resist seismic forces.

The fundamental consideration behind correlating provisions of this document (on seismic evaluation and strengthening) with IS 1893 (Part 1) is based on the relationship between risk of life loss in existing and new building. With the development of the modern Indian seismic codes, IS 1893 (Part 1), it is intended to develop building designs with features important for proper seismic performance. An existing building may not have seismic design features for seismic resistance but it is expected that to meet requirements of other codes of structural safety for loads such as gravity, wind, snow, etc.

C5.3– Load Modifying Factor

The modified load factor accounts for a reduction in the lateral force for an existing building under consideration for factors such as useable life of the building.

It is widely accepted that given the scale of economic and social impact of seismic strengthening deficient buildings, a higher level of risk of life loss should be adopted for existing buildings than for new buildings.

The general relationship between an earthquake return period, risk over a given period and the seismic coefficient suggest that for higher assumed risk seismic coefficient will be smaller. A higher level of risk can be accepted for an existing building than for

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a new building, considering the reduced useable life of the structure. Consequently, design forces can be reduced, resulting in reduction of retrofitting costs. This reduction in design force is effected through a multiplying factor equal to 0.67 applied to design base shear of existing structures.

Other codes also follow similar approach of reducing design base shear for the existing structures; some use constant reduction factors which range from 0.5 to 0.75 whereas others use reduction factors which are functions of remaining useable life of the structure in relation to its design life.

5.4 –

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 2.

Table 1: Knowledge factor, K

S. No	Description of Building	K
1	Original construction documents available, including post-construction activities, such as modification to structure or materials testing undertaken of existing structure.	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) no testing of , i.e., originally specified values for materials and minor deterioration of original condition	0.80

C5.4–

Knowledge factor K accounts for the confidence and reliability of the configuration and the condition of members of the lateral force-resisting system of the existing building. It can be established from study of the original documents of the buildings or nondestructive testing of representative members. Using established field tests for materials in the building, present day strength can be estimated and used for evaluation purposes even when it is higher than the design strength. Foundations are examples of members for which a lower K value can de adopted.

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4	Incomplete but useable original construction documents and no testing.	0.70
5	Documentation as in (4) and limited inspection, and verification of structural members, or materials test results with large variation	0.60
6	Little knowledge of details of a component	0.50

5.5 – Evaluation Process

Existing buildings not designed in accordance with the principles and philosophies and requirements of current seismic code as described in the following sections would be assessed.

5.5.1 –

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.

5.5.2 –

A detailed evaluation is required unless results of preliminary evaluation are acceptable.

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 can be exempted from detailed evaluation even when a preliminary evaluation indicates deficiencies and where seismic retrofitting is carried out to remedy those deficiencies.

5.5.3 –

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 will use modified demands and strengths.

5.5.4 –

A flow diagram summarizing various steps of the evaluation process is shown in Figure 1.

C5.4 – Evaluation Process

The evaluation process described in this document is a two level process comprising of increasing detailing and decreasing conservatism.

C 5.5.1 –

The purpose of a preliminary evaluation is to sieve out buildings that comply with the provisions of this document and quickly identify the potential deficiencies of all others with minimal analysis required. In case of compliance with the preliminary evaluation a detailed evaluation is not required.

C 5.5.2 –

If deficiencies are identified for a building, the design professional should proceed to conduct a more detailed evaluation of the building as described in Section 7 of this document.

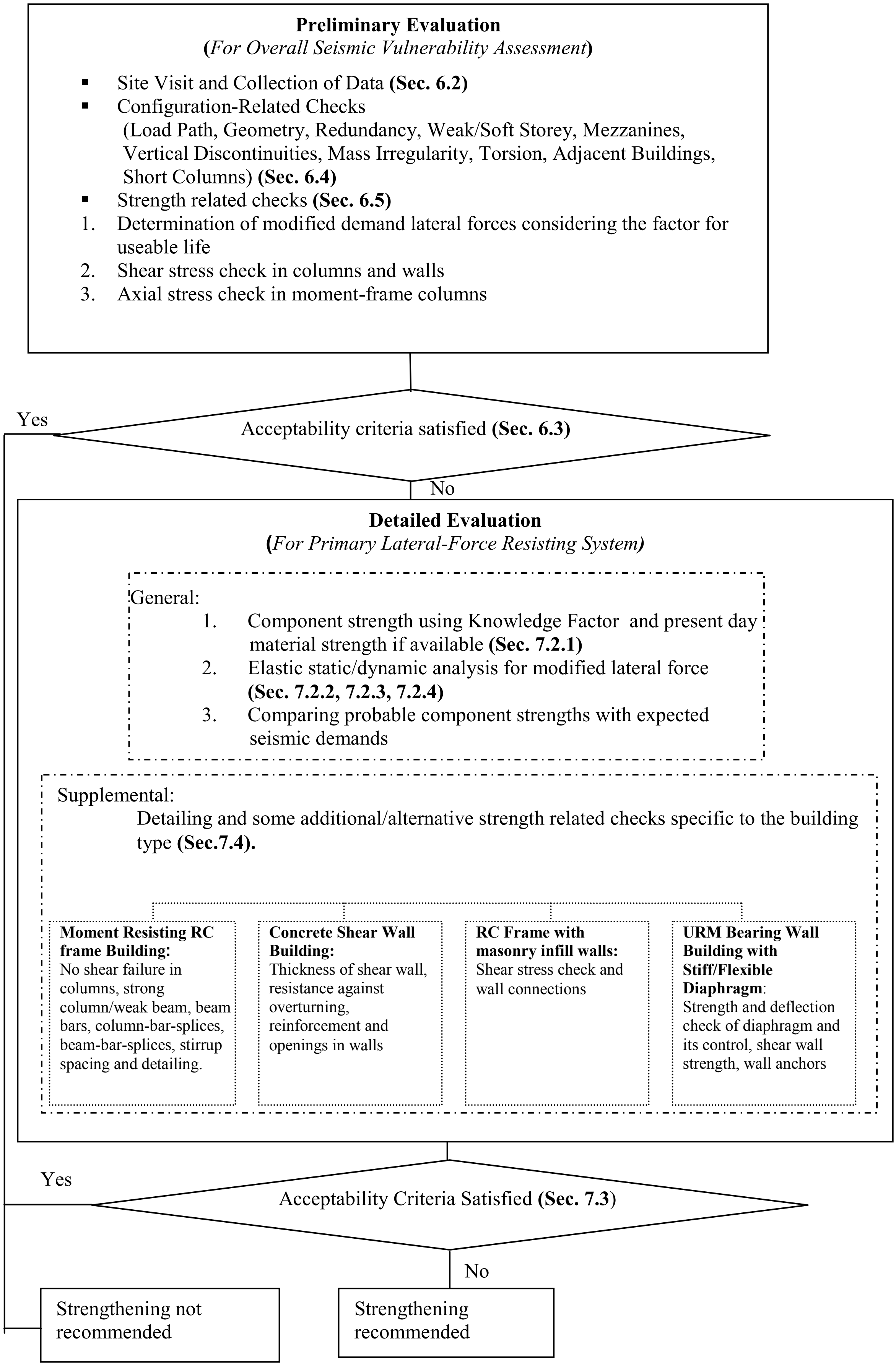

Figure 1: **Flow chart summarizing the evaluation process**

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6. –Preliminary Evaluation

6.1 – General

The 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 'back of the envelope' 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 walls, basement and foundation system

(c) Building type as in IS 1893 (Part1) 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 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.

Look out for special anomalies and

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C6. – Preliminary Evaluation

C6.1– General

The main purpose of the preliminary evaluation process is to quickly identify buildings that comply with the provisions of this document. Also it helps the design professional to become familiarize with the building, its potential deficiencies and its potential behavior.

C6.2 – Site Visit

A site visit is essential to get familiarized with the building system and take note of conditions which are usually not conveyed in designs/drawings and other documents. For reliable information on present day strength of materials a revisit should be undertaken with the objective to test/collect sample for laboratory investigations for the detailed evaluation phase.

Special emphasis should be given to study the effect of certain architectural features that may significantly affect the seismic performance. These include features like water tanks, parapets, infill walls and RC staircase.

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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 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.

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C6.3 – Acceptability Criteria

Configuration related checks ensure presence of coherent load path for lateral loads, and identification of structural features or conditions which have shown to result in poor performance of structures in past earthquakes. A few strength related checks are necessary to ensure the structural adequacy of the members which will transfer seismic forces from structure to the ground.

C6.4 – Configuration-Related Checks

C6.4.1 – Load Path

One of the fundamental attributes required for the proper seismic response of a building during earthquake motions is that its lateral load resisting members should be tied together to act as a single unit. This provision is intended to provide continuous lateral load system that ties all parts of the structures together. It also provides for proper connection between the members of the system to transmit additional seismic forces safely.

A vertical lateral force-resisting system should be continuous and should run from the foundation to the top of the building. The flow of seismic forces in the structure should be such that these forces are delivered through structural connections to horizontal diaphragms; the diaphragms then distribute these forces to the vertical lateral force resisting elements such as shear walls or frames; these vertical elements transfer the forces into foundation; and foundation transfers the forces into the soil.

The presence of discontinuity in a load path makes a building inadequate of carrying seismic forces. Therefore the design professional should identify any gaps in the load paths and then take necessary mitigation measures to complete the load path.

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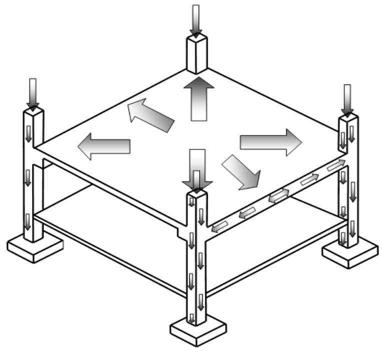


Figure C2: Load path

6.4.2 – Redundancy

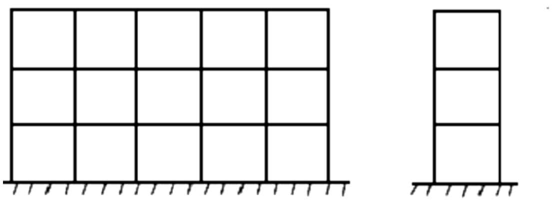
The number of lines of vertical lateral load resisting elements in each principal direction shall be greater than or equal to 2. In the case of moment/braced frames, the number of bays in each line shall be greater than or equal to 2. Similarly, the number of lines of shear walls in each direction shall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

C6.4.2 – Redundancy

The provision of redundancy is recommended because of the uncertainties involved in the magnitude of both seismic loads and member capacities. If any member of a lateral force resisting system fails, the redundancy of the structure will help ensure that there is another member present in the lateral force resisting system that will contribute lateral resistance to the structure.

Redundancy also provides multiple locations for potential yielding, possibly distributing inelastic activity within the structure and improving the ductility and energy dissipation. Typical characteristics of redundancy include multiple lines of resistance to distribute the lateral forces uniformly throughout a structure, and multiple bays in each line of resistance to reduce the shear and axial demands on any one element. Refer to Figure C3 (a) and figure C3 (b).

If enough redundancy is not present in the structure, an analysis is required to demonstrate the adequacy of the lateral force elements. A distinction should be made between redundancy and adequacy. Simple meaning of redundancy is “more than one”. One line of moment frame can be adequate to carry the entire design lateral load, but is not redundant.



Redundant **Non redundant**

Figure C3(a): Redundancy of a moment frame

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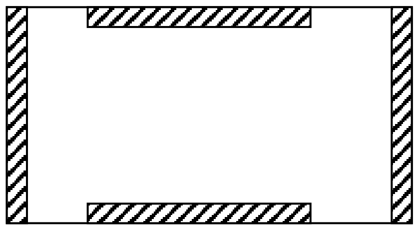


FigureC3 (b) : Redundancy of shear walls

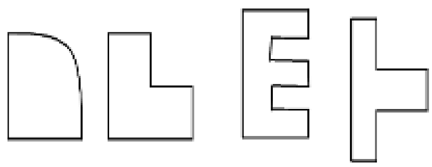
6.4.3 – Geometry

No change 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, should be made.

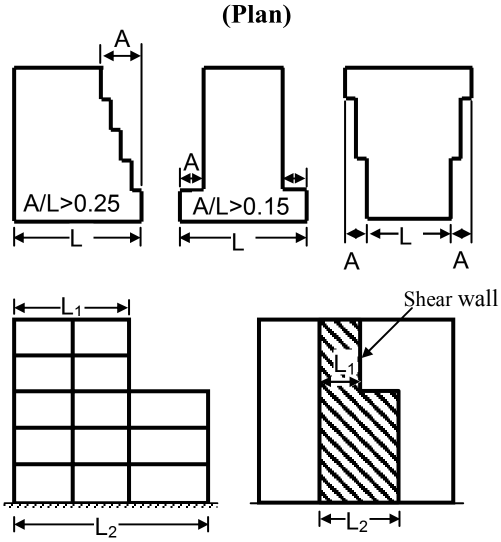
C6.4.3 – Geometry

Geometric irregularity of overall building shape in plan and elevation affects the seismic response of the structure by increasing ductility demands at a few locations. Also an irregular shape indicates an irregular mass distribution, and it may also happen that certain parts of building may respond dynamically independent to the rest of the building.

The plan configuration should always be symmetrical with respect to two orthogonal directions. It is also recommended that the plan should be compact and should not represent complex shapes, e.g. H, I, X, etc. Geometrical irregularity concerned with here is the dimension of the lateral-force-resisting system, not the dimensions of the building envelope.



(Plan)



(Elevation)

Figure C4: Irregularity in plan and elevation
(IS 1893)

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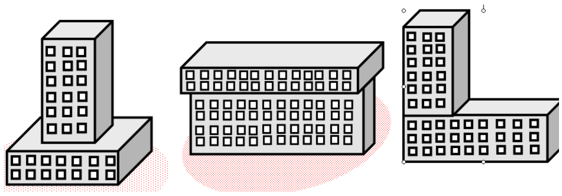

Figure C5: Buildings with offsets

6.4.4 – 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.

C6.4.4

Mezzanines/lofts/sub-floors can be defined as low or partial storey between the two storeys which are often added on later. They often lack a lateral-force-resistant system. Unbraced mezzanines pose a potential collapse hazard, and should be checked for stability.

When mezzanines are added on to the main structure, the supporting elements of the main structure should be evaluated considering both the magnitude and location of the additional forces imparted by the mezzanine. Also the lateral-force-resisting elements should be present in both directions to act as a bracing. If the load path is incomplete or non-existent, mitigation with elements or connections needed to complete the load path is necessary to achieve adequate performance level.

6.4.5 – Weak Storey

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

C6.4.5– Weak Storey

Weak storeys are usually found where vertical discontinuities exist, or where member size or reinforcement has been reduced. It is necessary to calculate the storey strengths and compare them. The result of a weak storey is a concentration of inelastic activity that may result in partial or total collapse of the storey.

The storey strength is the total strength of all the lateral force resisting elements in a given storey for the direction under consideration. It is the shear capacity of columns or shear walls, or the horizontal component of the capacity of diagonal braces. If the columns are flexural controlled, the shear strength is the shear corresponding to the flexural strength.

A dynamic analysis could be performed to determine if there are unexpectedly high seismic demands at locations of strength discontinuities. Compliance can be achieved if the elements of the weak storey can be shown to have adequate capacity near elastic levels.

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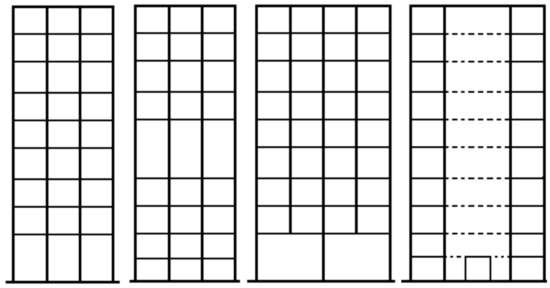

Figure C6: Cases where a weak storey can arise

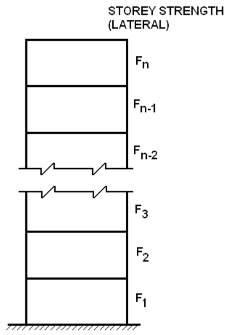

Figure C7: Storey strength variation *(IS 1893 (Part 1))*

6.4.6 – Soft Storey

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

C6.4.6 – Soft Storey

Soft storeys are present in buildings with open fronts on the ground floor or tall ground storeys. The most common examples are shopping malls, offices, hotels, etc. In such cases the tall columns should have enough stiffness and strength to resist seismic forces and special consideration should be given for inter-storey drifts. Soft storey buildings are well known for their poor performance during earthquakes. During Bhuj earthquake 2001 most of the multi-storey buildings collapses were due to soft storey.

A clear-cut distinction between soft and weak storey is usually not possible but there is a stark contrast between stiffness and strength. Stiffness is the force needed to cause a unit displacement and is given by the slope of the force-displacement relationship. Whereas, strength is the maximum force that a system can take. Soft storey refers to stiffness and weak storey refers to strength. Usually, a soft storey is also a weak storey. A column may be flexible but strong, or stiff but weak. A change in column size can affect strength and stiffness, and both need to be considered.

A soft storey can be detected by comparing the stiffness of adjacent storeys. It can also be revealed by an abrupt change in inter-storey drift. A dynamic analysis should be performed to

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determine if there are unexpectedly high seismic demands at locations of stiffness discontinuities.

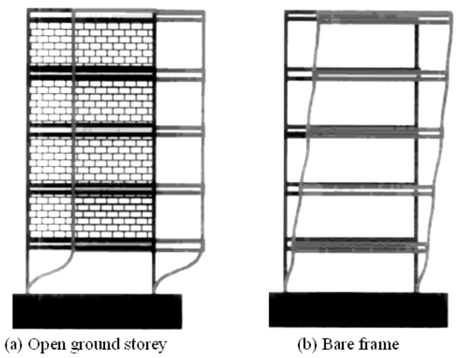

Figure C8: Soft-storey is subject to severe deformation demands during seismic shaking

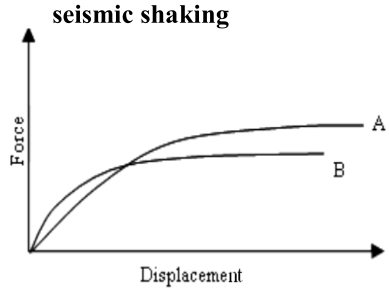

Figure C9: Structure A has higher strength & lower stiffness as compared to structure B

6.4.7 – Vertical Discontinuities

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

C6.4.7 – Vertical Discontinuities

Vertical discontinuities are usually found where elements are not continuous to the foundation but stop at an upper level and can be detected by visual observation. The most common example is a discontinuous shear wall or braced frame.

In the case where an element is not continuous to the foundation but stops at an upper level, the shear at this level might be able to be transferred through the diaphragm to other resisting elements below. This force transfer can be accomplished either through a horizontal strut/tie if the elements are in the same plane or otherwise through a connecting diaphragm (see Figure C10). In either case, the overturning forces that develop in the discontinuous element continue down through the supporting columns.

This issue is a local strength and ductility problem below the discontinuous element, not a global storey strength or stiffness irregularity. The concern is that the wall or braced frame may have more shear capacity than that considered in the design. These capacities impose overturning forces that could overwhelm the columns. While

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the strut or connecting diaphragm may be adequate to transfer the shear forces to adjacent elements, the columns which support vertical loads are the most critical. It should be noted that moment frames can have the same kind of discontinuity.

Compliance can be achieved if an adequate load path to transfer seismic forces exists, and the supporting columns can be demonstrated to have adequate capacity to resist the overturning forces generated by the shear capacity of the discontinuous elements.

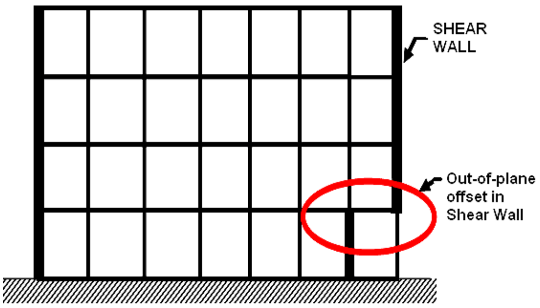

Figure C10: Vertical irregularity ***(IS 1893(Part 1))***

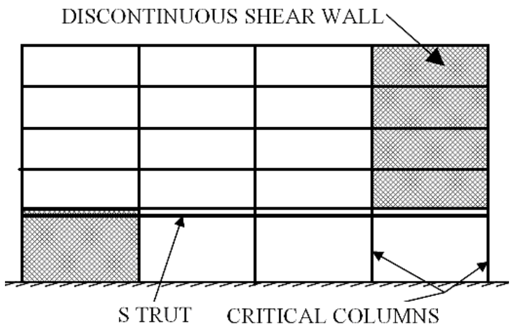

Figure C11: Discontinuous shear wall

6.4.8 – 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.

C6.4.8 – Mass

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

Mass irregularities affect the dynamic response of the structure by increasing ductility demands at a few locations and lead to unexpected higher mode effects.

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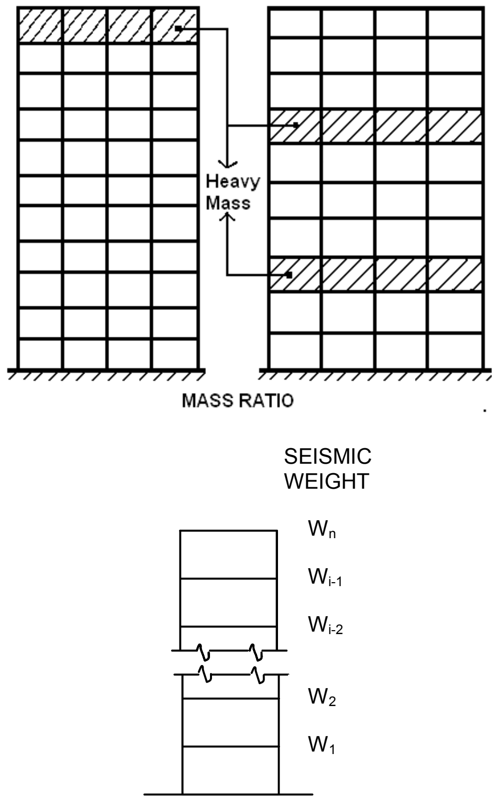

Mass Irregularity

WHEN $W_i > 2.0\, W_{i-1}$

OR $W_i > 2.0\, W_{i+1}$

Figure C12: Mass irregularity *(IS 1893 (Part 1))*

6.4.9 – 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.

C6.4.9– Torsion

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

During a seismic activity the eccentricity between the two centers will induce a twisting moment and additional horizontal forces in a building. As a result of these twisting forces on a building the rotation of diaphragm takes place imparting additional seismic loads and lateral drifts on to some vertical members. The worst effect of torsion during an earthquake is on the columns that support the diaphragm. They are forced to drift laterally with the diaphragm inducing lateral forces and p-delta effects. Such columns often have not been designed to resist these movements.

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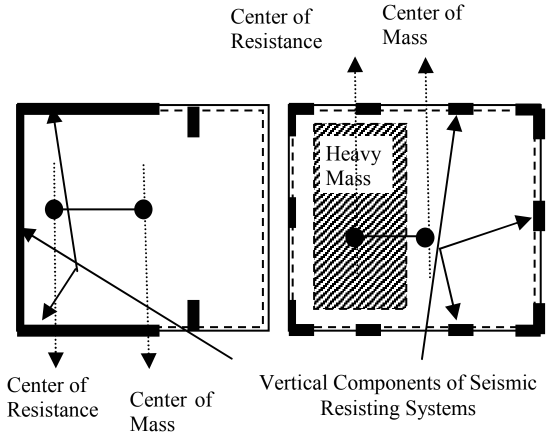

Figure C13: Torsional Irregularity

It is better to resist vertical load resisting elements such as a shear walls, braced frames etc apart in plan to increase torsion resisting lever arm as shown in Figure C15. Figure 14 shows a poor arrangement of such elements resulting in low torsional resistance.

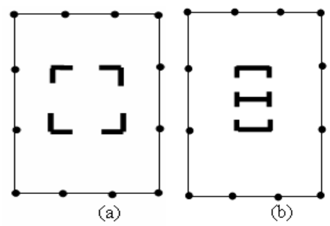

Figure C14: Arrangement of shear walls and braced frames-not recommended. Heavy lines indicate Shear walls and/or braced frames

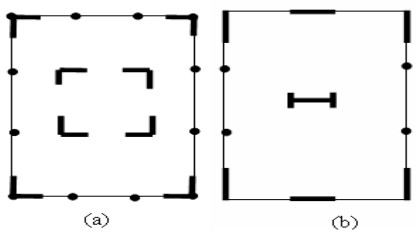

Figure C15: Recommended arrangement of shear walls and braced frames. Heavy lines indicate Shear walls and/or braced frames.

6.4.10 – Adjacent Buildings

The clear horizontal distance between the building under consideration and any adjacent building shall be greater than 4% of the height of the shorter building, except for buildings that are of the same height with floors located at the same levels.

C6.4.10– Adjacent Buildings

Buildings are often sited close to each other in order to make full use of the plot size. During seismic shaking two such adjacent buildings may hit each other due to lateral displacements. This is known as *pounding or hammering*. Building pounding affects the dynamic response of both buildings and additional inertia loads are induced

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on both structures.

Buildings with same height and matching storeys will show similar behavior and pounding damages will be limited to non-structural components. The pounding effect is much more serious if the floor levels of one building do not match those in the other. In this case, during seismic shaking the floors of one building will hit at the mid-height of columns in the other building causing them damage and possibly leading to their collapse. In case of buildings with different heights the shorter building may become prop for the taller building, leading to a major stiffness discontinuity altering its dynamic behavior, while pounding imparts additional loads on the shorter building.

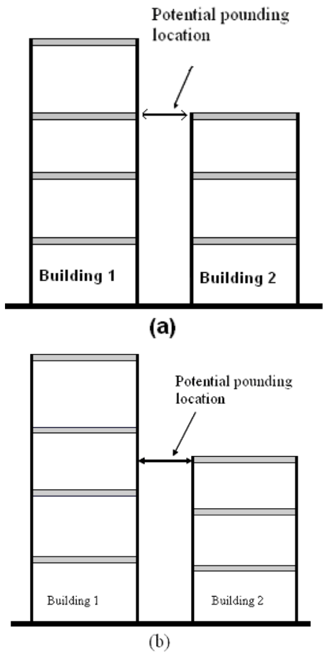

Figure C16: Pounding in situation (b) is far more damaging.

6.4.11 – 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.

C6.4.11– Short Columns

Short columns are relatively stiffer than other columns in a storey and tend to attract higher seismic forces because of their high stiffness relative to other columns. If not adequately detailed, such columns suffer a non-ductile shear failure which may result in partial collapse of the structure. A short column with the shear capacity to develop the flexural strength over the clear height will have some ductility to prevent sudden non-ductile failure of the vertical support system.

Short column behavior generally occurs in buildings with clerestory windows, or in buildings with partial height masonry infill panels.

Special confining reinforcement shall be provided

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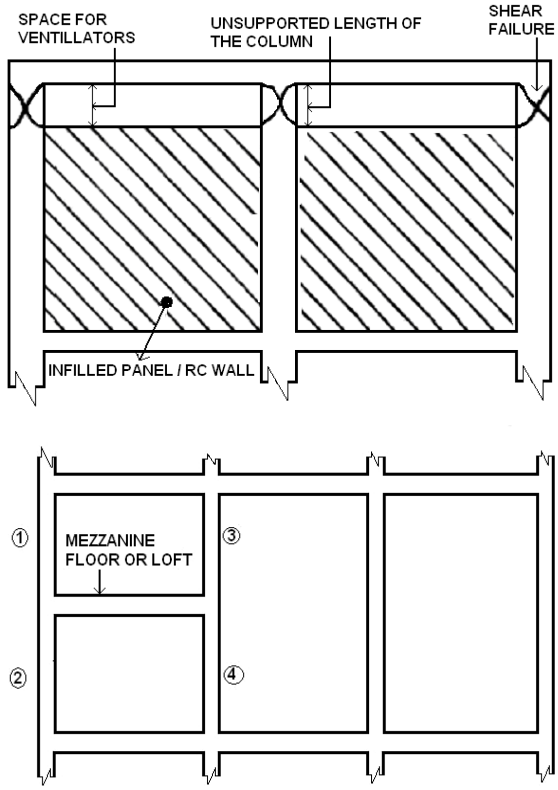
over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result due to the presence of bracing, a mezzanine floor/loft slab or a RC wall on either side of the column that extends only over a part of the column height (see Figure C17). This phenomenon is known as short column effect.



(1), (2), (3) and (4) are relatively stiff columns and attract large seismic forces.

Figure C17: Columns with variable stiffness

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 (Part1) and the requirements of Section 5 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 lesser of

C6.5.1 – Shear Stress in RC Frame Columns

Equation 6.1 assumes that all of the columns in the frame have similar stiffness. The term

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(a) 0.4 MPa and

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

$$\tau_{col} = \left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \quad ...... (6.1)$$

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.

6.5.2 – Shear Stress in Shear Walls

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

$$\tau_{wall} = \left(\frac{V_j}{A_w}\right) \quad ...... (6.2)$$

where,

V_j - storey shear at level j and

A_w - total area of shear walls in the direction

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$\left(\frac{n_c}{n_c - n_f}\right)$ is based on the assumption that shear force caused by columns at the end of RC frame are typically half of those carried by interior columns. However, this leads to a very conservative estimate (twice of the correct value) of shear for one-bay frame, but this discrepancy is not so serious for frames which are typically more redundant. . If a concrete column has a capacity in shear that is less than the shear associated with the flexural capacity of the column, brittle column shear failure may occur and result in collapse. The columns in these buildings often have ties at standard spacing equal to the depth of the column, whereas the current code requires the maximum spacing for shear reinforcing as *d/2.*

The following are the characteristics of concrete moment frames that have demonstrated acceptable seismic performance:

a. Brittle failure is prevented by providing a sufficient number of beam stirrups, column ties, and joint ties to prevent premature shear failure.

b. Concrete confinement is provided by beam stirrups and column ties in the form of closed hoops with 135-degree hooks.

c. Overall performance is enhanced by long lap splices that are restricted to favorable locations where moments are smaller and protected with additional transverse reinforcement.

d. The strong column/weak beam requirement is achieved by suitable proportioning of the members and their longitudinal reinforcing.

Older frame systems that are lightly reinforced, precast concrete frames, and flat slab frames generally do not meet the above requirements for ductile behavior.

C6.5.2 – Shear Stress in Shear Walls

The primary load resisting elements in URM buildings are load bearing walls. In highly redundant buildings with many long shear walls, stresses can be low but in less redundant buildings with large openings and slender walls, stresses can be high. Larger stresses indicate vulnerability in earthquakes, which need further investigation.

The in-plane shear strength of masonry walls is crucial factor for the survival and stability of the URM buildings particularly those buildings wherein the large size openings in masonry walls make them extremely weak and vulnerable.

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of the loading.

(a) For concrete shear walls, τ_{wall} shall be less than 0.4 MPa.

(b) 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 RC masonry infill walls

The shear stress in the reinforced masonry shear walls shall be less than 0.3 MPa and the shear stress in the unreinforced masonry shear walls shall be less than 0.1 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_o) as calculated using the following equation shall be less than $0.25f_{ck}$.

$$F_o = \frac{2}{3}\left(\frac{V_B}{n_f}\right)\left(\frac{H}{L}\right) \quad \text{...... (6.3)}$$

where,

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

V_B = base shear

H = total height

L = length of the building.

6.5.5 – 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 in RC and steel; and 3 storeys and higher in unreinforced masonry.

c) Buildings located on incompetent or liquefiable soils and/or located near (less than 12 km) active faults and/or with inadequate foundation

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C6.5.3 – Shear Stress Check for RC masonry infill walls

The shear stress check provides a quick assessment of the overall level of demand on the structure.

C6.5.4 – Axial Stress in Moment Frames

Response to earthquake ground motion results in a tendency for structures and individual vertical elements of structures to overturn about their bases. Although actual overturning failure is very rare, overturning effects can result in significant axial stresses.

Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. When axial forces due to seismic overturning moments are added, the columns may buckle in a non-ductile manner due to excessive axial compression.

The factor 2/3 in equation 6.3 is based on the assumption that floor forces due to earthquake are distributed in the inverted triangular pattern over the building height.

C6.5.5 – Recommendation for Detailed Evaluation

Detailed evaluations need to be carried out for buildings with greater consequence in terms of loss of life. Minor buildings which have passed preliminary checks on seismic robustness are exempted from detailed evaluation, which requires considerable effort in collecting information and performing the analyses. However, detailed evaluations are mandatory for important buildings and those buildings with problematic soils/foundations and structural systems which require detailed investigation.

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details.

d) Buildings with inadequate connections between primary structural members, such as poorly designed and/or constructed joints of pre-cast elements.

COMMENTARY

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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 Section 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 (Part1) for lateral forces prescribed therein with modification for (reduced) Useable life factor, described in Section 5 of this document.

7.1.1 - Condition of the Building components.

The building should be checked for the existence of some of the following common indicators of deficiency.

(a) **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

(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

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C7. – Detailed Evaluation

C7.1 – General

Under the detailed evaluation stage, a full building analysis is performed with respect to its present condition and adequacy of the lateral-load-resisting system. The evaluation requires a linear elastic analysis which could be linear static or dynamic as mentioned in IS 1893 (Part1)

. Essentially, the detailed evaluation procedure is based on the analysis and design philosophy of IS 1893 (Part1).This involves an equivalent static lateral force or a response spectrum analysis procedure, a working load with response reduction factors, allowable stress design or limit state design, and capacity over demand ratios for ductility as in IS 13920. However, forces for the analysis have been suitably modified to accept a higher risk of life loss in existing buildings.

C7.1.1- Condition of the Building the Buildings

A re-visit to the site should be made by the design professional to verify the existing data including those used in the preliminary analysis, and importantly to check the condition of various building components and perform suitable tests to assess the present day strength of materials for greater reliability. Deteriorated building components can jeopardize the capacity of a building to resist lateral forces.

Deterioration of Concrete -

When concrete is deteriorated the strength of concrete elements reduces significantly and along when water penetrates into concrete easily corrosion of reinforcing bars begins. This may lead to loss of cross-section of rebar and further strength loss. Concrete deterioration can also take the form of spalling which can lead to reduction in available surface area for bond between the concrete and steel. Further, corrosion can lead to a significant reduction in the cross section of a bar.

Cracks in Boundary Columns-

Large displacements or crushing of concrete results in the formation of cracks and signify a reduction in the strength of structural components. Crack width is a commonly used indicator of damage level in components. Small cracks in components have little effect on strength but are

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than 3 mm, or have out-of-plane offsets in the bed joint greater than 3 mm.

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matter of concern when they are large enough and not provide aggregate interlock or resistance against buckling of the reinforcement steel.

Columns may be required to resist diagonal compression strut forces that are developed in infill wall panels, axial forces induced by vertical components and the moment due to eccentricity between horizontal components and the beam. Columns having cracks spread over a large area may indicate locations of possible weakness and such columns may not be able to function in conjunction with the infill panel as expected.

Cracks in Infill Walls -

Diagonal wall cracks affect the interaction of the masonry units with surrounding frame and lead to a reduction in strength and stiffness. These cracks may also indicate distress in the wall from past seismic events, foundation settlement, or other causes.

Offsets in the bed joint along the masonry joints may affect the interaction of the masonry units in resisting out-of-plane forces. Recent studies list other factors, such as location, orientation, number, distribution and pattern of the cracks to be equally important in measuring the extent of damage present in the shear walls (ATC 43). All these factors should be considered when evaluating the reduced capacity of a cracked element.

7.1.2 – Condition of the Building Materials

An evaluation of the present day strength of materials can 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 should be supplemented with proper laboratory facilities for accurate quantitative results.

C7.1.2 – Condition of the Building Materials

Some standard test techniques are described below for easy reference. Further details of some of the techniques are given in Appendix C_1.

Table C1: Concrete Structures

Information Required	***Test Techniques***
Strength	Rebound hammer
Depth of carbonation	Petrographic examination
Permeability	Surface absorption test Water and gas permeability test Absorption test on intact cores
Rate of carbonation Rate of corrosion	Presence, position of, and cover to steel reinforcement
Extent of corrosion of reinforcement	Physical exposure Electrical potential

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Delamination	Ultrasonic pulse velocity Core Examination
Elongation of steel reinforcement	Laboratory tests (IS 10790:Part1:1984)

Table C2: Masonry structures

Information Required	***Test techniques***
Strength of clay units	Crushing of units
Strength of cement based units	Rebound hammer Internal fracture test Windsor test Crushing of units
Compressive strength of masonry	Crushing strength of units combined with mortar mixed proportions Split cylinder tests on horizontal cores with and without horizontal diametric bed joints Flatjack test
Moisture test	Direct moisture measurement
Shear resistance of masonry	Presence of voids, internal structure, thickness of retaining walls.

7.2 – Evaluation Procedure

The key steps of this evaluation procedure are described 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 (Part1) and multiply it with *U,* a factor for the reduced useable life (equal to 0.67).

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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 (Part1) for the modified base shear determined in the previous step and determine resulting member actions for critical components.

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-Story Drift

Calculate whether the inter-storey drifts and decide whether it is acceptable in terms of the requirements of IS 1893 (Part1).

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 section 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 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.

7.4 – Supplemental Evaluation

In addition to the general evaluation (Section 7.2) for buildings which addresses only strength issues more criteria need to

COMMENTARY

C7.3 – Acceptability Criteria

The evaluation procedure described in Section 7.2 is primarily a strength check and does not address many issues of ductility and energy dissipation capacity which are responsible for survival of buildings in severe earthquakes. Therefore, it is important that an evaluation of structural detailing which ensures ductile behavior should be performed for a complete assessment of the structures' ability to resist earthquakes. These detailing provisions are specific to each building type and therefore have to be addressed separately. Lists of these checks are specified under Supplemental Criteria in Section 7.4.

Pushover analysis is a simple method that performs a non-linear static analysis of building structures subjected to monotonically increasing horizontal loading. By a pushover analysis, the base shear versus roof displacement curve of the structure, usually called the capacity curve, is obtained. As the magnitude of the loading increases, weak links and the failure modes of the structure emerge. The ATC-40, ASCE 31-03 and FEMA-273 documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis that must be satisfied.

C7.4 – Supplemental Evaluation

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be considered which relate to ductility and detailing of structural components. These criteria address certain special features affecting the lateral load-behavior which are specific to each building type.

7.4.1 – Moment Resisting RC Frame Buildings

For RC moment frame buildings designed using response reduction factor R (IS 1893 (Part 1)) equal to 5 the following supplemental criteria need to be satisfied.

(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.

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No Shear Failures -

Shear failure is a brittle failure whereas flexural failure is ductile. When the column attains shear capacity before its moment capacity, it develops a non-ductile sudden failure, leading to collapse. Columns should therefore be designed in such a fashion to not reach their shear capacity before reaching its flexural capacity. The shear capacity of a column is affected by axial loads acting on it, so its shear capacity should be based on the most critical combination of axial load and shear.

The design shear force for columns shall be the maximum of:

a) calculated factored shear force as per analysis,

b)a factored shear force given by

$$V_u = 1.4\left[\frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}}\right]$$

Where $M_{u,lim}^{bL}$ and $M_{u,lim}^{bR}$ are moment of resistance, of opposite signs, of beams framing into the column from opposite faces (see Fig. C18); and h_{st} is the storey height. The beam moment capacity is to be calculated as per IS 456: 2000. The factor of 1.4 is based on the consideration that plastic moment capacity of a section is usually calculated by assuming the stress in flexural reinforcement as 1.25 f_y as against 0.87 f_y in the moment capacity calculation.

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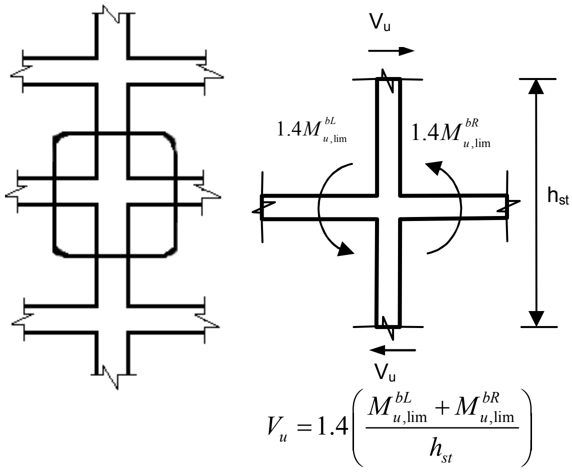

Figure C18: Calculation of design shear force for column

(b) Concrete Columns — All concrete columns shall be anchored into the foundation.

Concrete Columns -

When a concrete column is an integral part of a lateral load resisting system it may be subjected to additional tensile or compressive loads in addition to loads from dead and live load which can produce unfavorable situations of uplift and reduce its shear capacity. A proper connection is required to reduce the chances of uplift or sliding of the support.

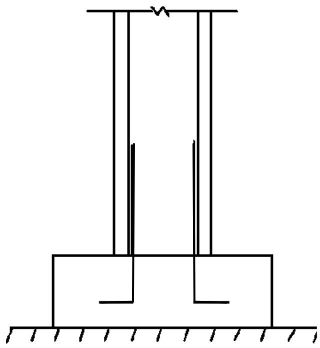

Figure C19: Column anchored into foundation

(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.

Strong Column/Weak Beam -

Failure of columns before failure of beams can lead to a storey mechanism. This may cause large displacements which also can lead to the instability of the whole structure. So for ductile behavior the failure of the beams are preferred than failure of columns. Therefore, capacity of columns should be larger than the capacity of the beams with due consideration of the over strength of the beams.

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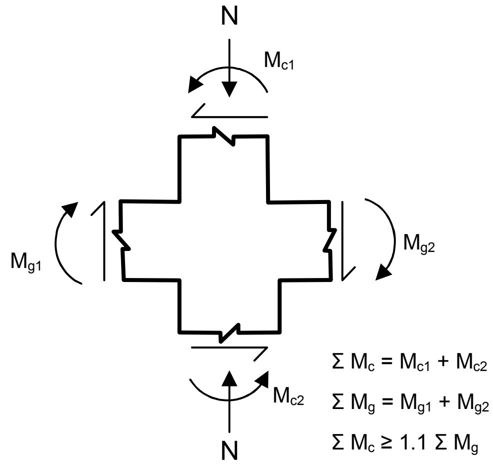

Figure C20: Concept of strong-column weak-beam

(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% of the longitudinal bars located at the joints for either positive or negative moment shall be continuous throughout the length of the members.

Beam Bars -

Two continuous bars are required to prevent total collapse in case of complete beam failure. The continuous bars will also prevent total collapse of the supported floors by holding the beam in place by catenary action. The current construction practices use bent up longitudinal bars as reinforcement which are transitioned from bottom to the top at gravity load inflection point. But during earthquakes the moments due to seismic forces can shift the location of inflection points; therefore it is desired to provide at least two continuous top and bottom reinforcement.

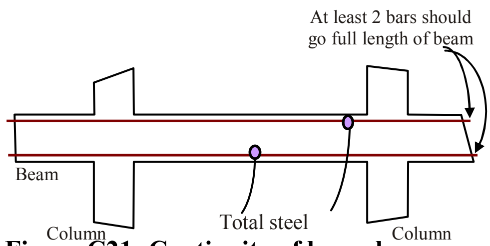

Figure C21: Continuity of beam bars

(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: 2000.

Column-Bar Splices -

Column bar splices are generally located just above the floor levels and in the vicinity of potential plastic hinge formation regions. Splice failures are non-ductile and sudden. Short splices are vulnerable to sudden loss of bond and widely spaced ties result in a spalling of the concrete cover and loss of bond. Splices are the locations of potential weakness. Therefore, it is preferable not to splice all bars in same location. When it is unavoidable it is only allowable with a penalty of increased bond length by increasing the lap length. Seismic moments are maximum in columns just above and just below the beam.

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Hence, reinforcement must not be changed at those locations. Also the seismic moments are minimum in the central half of the column height. Thus a good practice is to specify the column reinforcement from a mid-storey-height to next mid-storey-height. There is one very important implication of this clause, pertaining to the dowels to be left out for future extension. Inadequate projected length is a very serious seismic threat as this creates a very weak section at all columns at that location and all upper storeys are liable to collapse at that point.

All the lap splices should be proportioned as tension splices, as columns may develop substantial reversible moments (more than what we have designed the column for), when subjected to seismic forces. Hence all the bars are liable to go under tension. The provision of restricting the percentage of bars to be lapped at the same location means that in buildings of normal proportions, half the bars to be spliced in one storey and the other half in the next storey leading to construction difficulties. A good alternative can be of allowing all the bars to be lapped at the same location but with a penalty on the lap length.

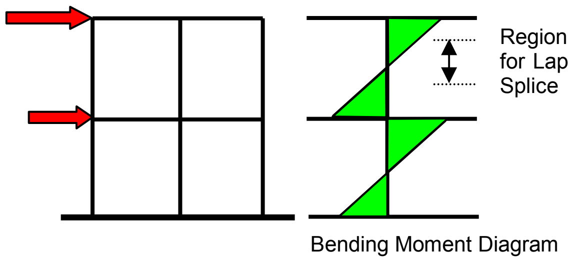

Figure C22: Lap splices

(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 (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.

Beam-Bar Splices -

End zones of beams are potential zones of plastic hinges. These are the locations where moment demand will reach the capacity first, so if splices are present there they may not achieve the full capacity and turn into potential zone of failure before reaching the design capacity.

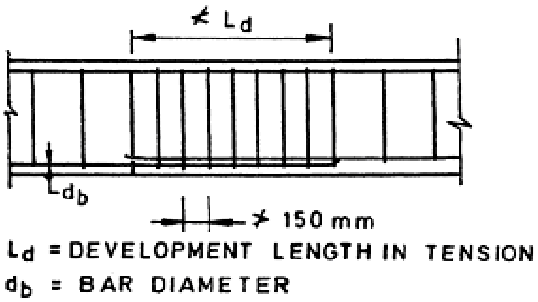

Figure C23: Lap splice in beam

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(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 crosstie 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 (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*.

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Column-Tie Spacing -

Column tie spacing is limited to a certain value to ensure ductile behaviour in the column. Better confinement improves the cyclic loading property of the column.

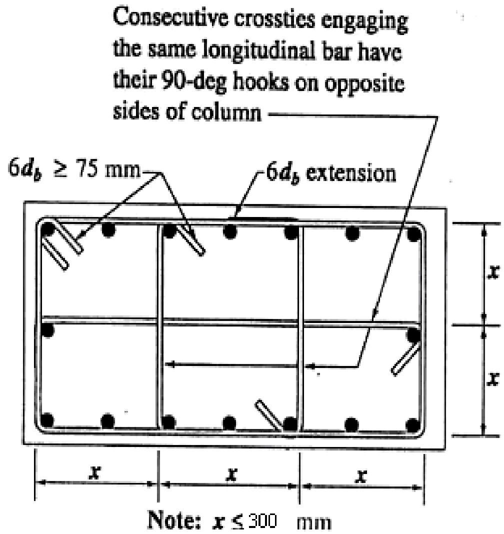

Figure C24: Transverse reinforcement in column

Stirrup Spacing -

Widely spaced ties are not good to carry shear in the sense that problem of local buckling may occur in the ties due to shear load which act axially on the ties. So effective length of tie bars should be reduced to carry more shear. And increase in ties also increases the shear capacity in the plane parallel to the ties, thereby increases the likelihood of ductile failure. To ensure space for needle vibrator, the minimum hoop spacing has been restricted to 100 mm. IS 456 allows ¾ *d* as against the requirement of *d/2* in this clause. One should bear in mind that the provisions of IS 13920 are over and above IS 456 subject to the applicability of IS 13920 in various seismic zones.

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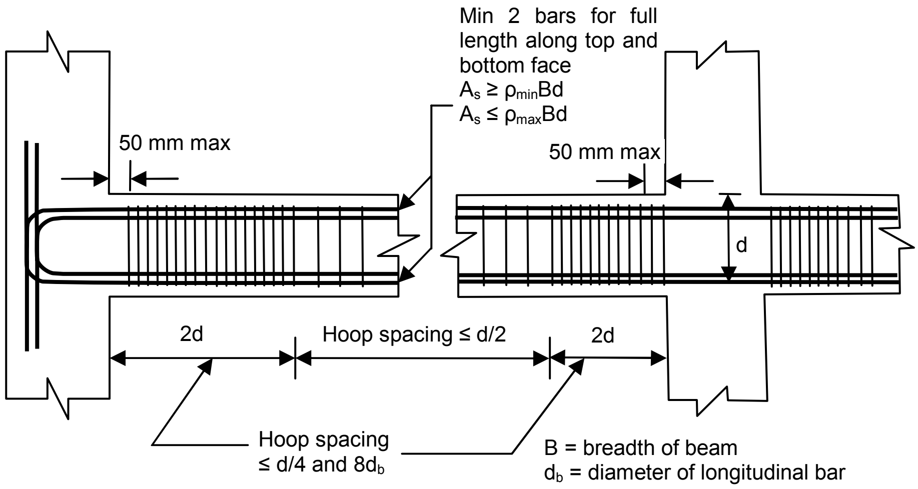

Figure C25: Beam reinforcement

(i) Joint Reinforcing— Beam-column joints shall have ties spaced at or less than 150 mm.

Joint Reinforcing -

Beam column joint requires shear reinforcement to develop the required strength of joint and connecting members. If the shear reinforcement is not present the joint will have a non ductile failure (Figure C26). Perimeter columns are especially vulnerable to this failure because the confinement of joint is limited to three sides (along the exterior) or two sides (at a corner).

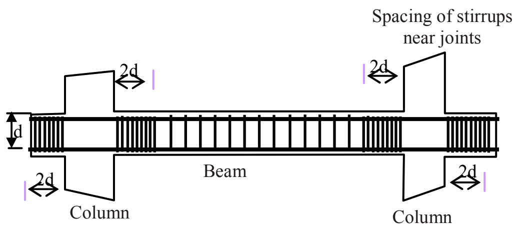

Figure C26:Location and amount of vertical stirrups in beams

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

Stirrup and Tie Hooks -

Stirrups and ties must be anchored into the confined core of the member to be fully effective, otherwise the shear resistance and confinement will be reduced. 90°hooks that are anchored within the concrete cover are not as effective as 135° hooks if the cover spalls during plastic hinging.

Though the shear cracks develop at 45^0 of the longitudinal direction, direction of shear forces may reverse during earthquake shaking and then the inclined hoops designed for shear in one direction will not be effective. Closed stirrup should always be used because open stirrups are not effective in confining the concrete. 135^0 hooks and 10 diameter extension (≥75 mm) provide good anchorage to stirrups.

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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.

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 200mm.

7.4.2.2 – Overturning –

All shear walls should have aspect ratio less than 4 to 1, else the foundation system should 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 0.0015 of the gross area in each direction. For ductile shear walls this value is increased to 0.0025 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^{o} or more. The shear and flexural demand on coupling beams which are non-compliant are calculated using analysis procedure of section 7.2 and their adequacy is checked. If they are found inadequate then their adequacy is checked as if they were independent.

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C7.4.2 – Concrete Shear Wall Buildings

Large lateral force may cause damage to a shear wall in several modes. The overturning moment can result in a buckling failure at the compression face of a thin wall or a non-ductile tension failure at the tension edge. Ductile tension failure may consist of slippage in bar splices, bar yield and fracture.

C7.4.2.1 – Thickness

The minimum thickness is specified for shear walls as 100 mm to avoid unusually thin section. Very thin sections are susceptible to lateral instability in zones where inelastic cyclic loading may have to be sustained.

The requirement of minimum thickness in case of coupled shear walls is introduced in view of constructional difficulties in providing diagonal bars with the other steel in the coupling beam.

C7.4.2.2 -Overturning

Walls having higher slender slenderness have limited overturning resistance. So, if overturning forces are not properly resisted then displacement at the top of the wall may be greater than allowable value causing overturning of the wall.

C7.4.2.3 –Reinforcement

Distribution of a minimum reinforcement uniformly across the height and width of the wall helps to control the width of inclined cracks that are caused due to shear.

Coupling beams if properly reinforced to provide them with sufficient strength and stiffness, can increase the lateral stiffness of the building significantly. Lateral deflection of walls induces large moments and shears in the coupling beams as they resist imposed deformations. Seismic forces may damage and degrade inadequately reinforced beams to such an extent that system degenerates into pair of independent walls. This results in distribution of overturning forces which may cause potential stability problems for

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independent walls. Reinforcement pattern in a coupling beam is shown in the Figure below.

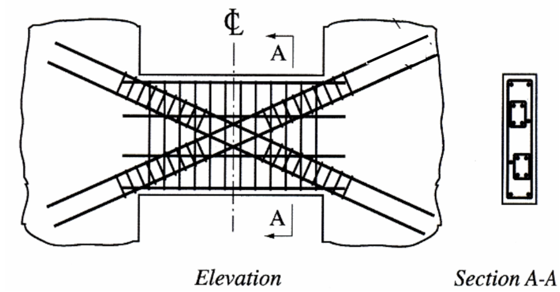

Figure C27: Schematic of reinforcement in coupling beams [from ACI318M, 2002]

7.4.2.4 – Opening in Walls

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

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

C7.4.2.4 – Opening in Walls

The perimeter walls that are considerably open, in some buildings with large open fronts, have limited wall length to resist seismic forces. These walls may be subjected to overturning or shear transfer problems during earthquake that were not accounted for in the original design, causing damage to these walls and the building.

7.4.3 – RC Frames with Masonry Infill Walls

The provisions of Section 7.4.1 also apply to RC frames with masonry infill walls. In addition, the infill walls should be checked for the following additional criteria:

Wall Connections — All infill walls shall have a positive connection to the frame to resist out-of-plane forces.

Wall Connections -

Performance of frame buildings with masonry infill walls is dependent upon the interaction between the frame and infill panels. The infill panel acts as a compression strut extending diagonally between corners of frame and provides resistance against in plane lateral forces. If a gap is present between frame and infill the strut action can not be developed between them. In case of panel walls separated from the frame due to out-of-plane forces, the properties of the bare frame determine the strength and stiffness of the system. If it is not detailed for earthquake forces severe damage may occur due to excessive drifts and *P*-delta effects.

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7.4.4 – Unreinforced Masonry Bearing Wall Buildings with Stiff Diaphragms

The following additional points must be considered for unreinforced masonry bearing wall buildings with stiff diaphragms during a detailed evaluation:

(a) **Height-to-thickness ratio—** The unreinforced masonry wall height-to-thickness ratios shall be less than as given in Table 3. The band beams (sill/lintel) are assumed to provide necessary lateral support for the unreinforced masonry wall in out-of-plane direction. The beams must be anchored into the return walls.

Table 3: Allowable height-to-thickness ratios of unreinforced masonry walls

Wall Type	Zone II & III	Zone IV	Zone V
Top storey of multi-storey building	14	14	9
First storey of multi-storey building	18	16	15
All other conditions	16	16	13

Height-to-thickness ratio -

Slender unreinforced masonry bearing walls with large height-to-thickness ratios (*h/t*) are more susceptible to damage from out-of-plane forces. Height `*h*` is unsupported height of unreinforced masonry wall, which is usually the storey height unless bands (at lintel/sill level) are present.

It is not necessary that unreinforced masonry walls are analyzed for the out-of-plane forces and in lieu of calculating bending stresses, as specified in IS 1905, permissible *h/t* ratios for the unreinforced masonry walls can be specified, which are dependent on the seismic zone and their location in the building.

Dynamic testing of full size walls has shown that walls meeting these *h/t* ratios are dynamically stable and their stability does not depend on the tension resistance of the masonry across bed joints. The permissible *h/t* ratio results high reliability of assessing wall stability after cracking. The walls at the uppermost storey level are most vulnerable because there is no overburden and horizontal acceleration are far greater than for lower levels. Walls at the first storey of the buildings have less vulnerability as the unamplified ground motion shakes the top of the wall.

In cases where *h/t* ratios of existing walls are exceed. The values given in Table 3, walls need to be braced to improve their stability conditions.

(b) **Masonry Lay-up —** Filled collar joints of multi wythe masonry walls shall have negligible voids.

Masonry Lay-up -

Walls having poor collar joints and an inadequate number of headers will have inner and outer wythes acting independently. This could make them inadequate to resist out-of-plane forces due to a lack of composite action between the inner and outer wythes. In this case of non-compliance a mitigation measure to provide out-of-plane stability and including anchorage of the wythes may be necessary.

(c) **Wall Anchorage –**

Walls shall be properly anchored to

Wall Anchorage -

Bearing walls should be positively anchored to

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diaphragms for out-of-plane forces with anchor spacing of 1.2 m or less.

(d) Connections – Diaphragms shall be reinforced and connected to transfer of loads to the shear walls.

(e) Openings In Diaphragms Near Shear Walls – Diaphragm openings immediately adjacent to the shear walls shall be less than 25% of the wall length.

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diaphragms in order to prevent walls from separating from the structure, which could lead to partial collapse of the floors and roof. Non-bearing walls which separate from the structure during an earthquake may represent a significant falling hazard and this hazard becomes more severe with the height above the building base.

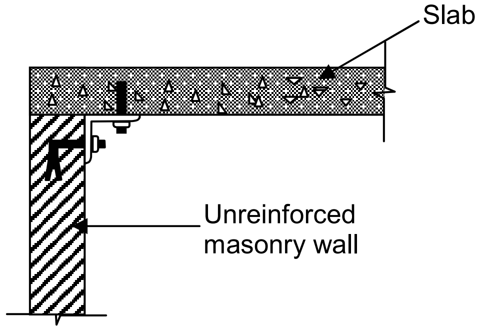

Figure C28: Wall anchorage

In order to prevent pull out failure of the anchor or local failure of the diaphragm, the anchorage forces must be fully developed into the diaphragm. In case of non-compliance and non-existence of anchorage, mitigation with elements or connections needed to anchor the walls to the diaphragms is necessary to achieve the selected performance level.

Connections -

The absence of proper connection between the diaphragms and supporting shear walls may cause the diaphragms/flow system to slide off of bearing supports during an earthquake. Suitable connections should be provided to prevent sliding and ensure load transfer.

Openings In Diaphragms Near Shear Walls-

Large openings in diaphragms near the shear walls significantly limit the ability of the diaphragm to transfer lateral forces to the wall. This can have a compounding effect if the opening is near one end of the wall and divides the diaphragm into small segments with limited strength that are ineffective in transferring shear to the wall. The presence of drag struts developed into the diaphragm beyond the wall will help mitigate this effect.

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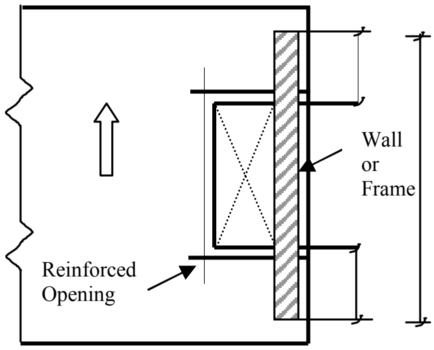

Figure C29 : Opening adjacent to shear wall

(f) Openings In Diaphragms Near Exterior Masonry Shear Walls – Diaphragm openings immediately adjacent to exterior masonry shear walls shall not be greater than 2.5 m.

(g) Plan Irregularities – There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities.

Plan Irregularities -

Diaphragms having plan irregularities such as extending wings, plan insets, or E-, T-, X-, L-, or C-shaped configurations have re-entrant corners where large tensile and compressive forces can develop (Figure C30). The diaphragm may not have sufficient strength at these re-entrant corners to resist these tensile forces. Local damage may occur.

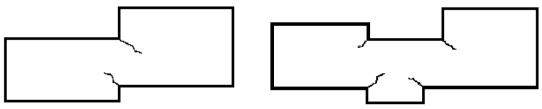

Figure C30: Re-entrant corners

(h) Diaphragm Reinforcement at Openings- There shall be reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.

Diaphragm Reinforcement at Openings-

Openings in diaphragms increase shear stresses and induce secondary moments in the diaphragm segments adjacent to the opening. Tension and compression forces are generated along the edges of these segments by the secondary moments, and must be resisted by chord elements in the sub-diaphragms around the openings. Openings that are small relative to the diaphragm dimensions may have only a negligible impact. Openings that are large relative to the diaphragm dimensions can substantially reduce the stiffness of the diaphragm and induce large forces around the openings.

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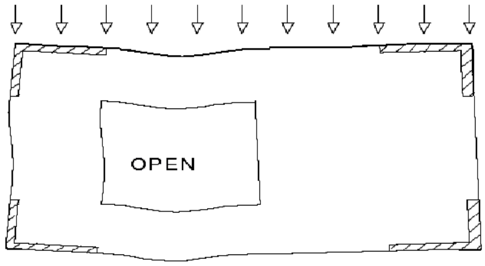

Figure C31: Diaphragm openings

7.4.5 – Unreinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

Unreinforced masonry buildings with flexible diaphragms can be analyzed by the general procedure described in sections 7.3 and 7.4.3. However, when applicable they may be analyzed by the *Special Procedure* described in Appendix A.

C7.4.5 – Unreinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

The distribution of horizontal shear in vertical elements depends on the rigidity of the diaphragm spanning across the vertical elements. Rigid diaphragms have adequate strength and stiffness to redistribute the forces and cause increased forces due to torsional effects. However, flexible diaphragms are incapable of redistributing horizontal shear. Moreover, shear forces transmitted to vertical elements will be limited by the non-linear behavior of the diaphragm. In other words, it is not necessary that the end-shear resistance of the diaphragm is same as the total lateral load calculated at that level, which is the product of weight tributary to that level and the base shear coefficient. As a result, lateral loads are distributed vertically in relation to strength and stiffness characteristics of the diaphragm. This behavior is considered in the *special procedure* along with non-linear force-deformation behavior of diaphragm.

7.4.5.1 - Horizontal load transfer for flexible diaphragm

Flexible diaphragms of unreinforced masonry buildings may deflect so much that the continuity is negligible and the distribution of load to the relatively stiff vertical elements is essentially on a load periphery basis.

C7.4.5.1 - Horizontal load transfer for flexible diaphragm

A flexible diaphragm can be considered to be analogous to a continuous beam or series of beams spanning between supports. The supports, usually walls, are considered non-yielding, and the relative stiffness of the walls in comparison to that of the diaphragm is very large. Thus, a flexible diaphragm is considered to distribute the lateral forces to the vertical resisting elements on a tributary area basis as shown in Figure C32 (a). On the other hand, for a rigid diaphragm, the distribution to vertical elements will be essentially in proportional to their relative stiffness with respect to each other as shown in Figure C32 (b).

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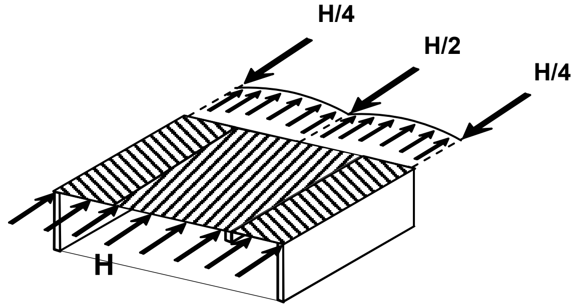
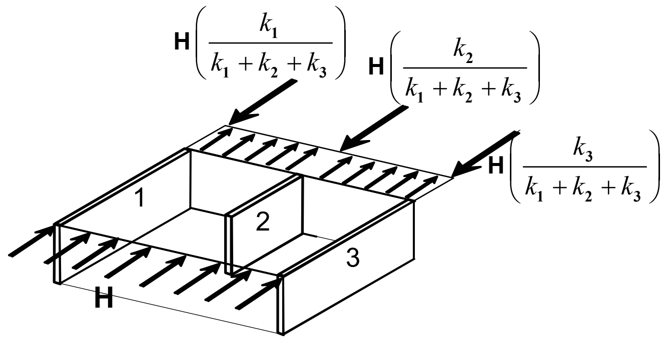


Figure C32: (a) Peripheral distribution of horizontal forces for a flexible diaphragm



k_1, k_2 and k_3 are lateral stiffness of walls 1, 2 and 3 respectively.

Figure C32: (b) Proportionate stiffness distribution of horizontal force for a rigid diaphragm

7.4.5.2 – Strength check of diaphragm

Floor and roof diaphragms should be able to resist diaphragm forces F_{px} as given below:

$$F_{px} = \frac{\sum_{i=x}^{n} Q_i}{\sum_{i=x}^{n} w_i} w_{px}$$

where, w_{px} is weight of roof or floor diaphragm at the level x and Q_i and w_i are lateral loads and seismic weights at the *i*-th floor as given in IS 1893 (Part 1) (Figure C33).

The force F_{px} determined from the above equation shall not be more than 0.75 ZIw_{px} and less than 0.35 ZIw_{px}.

C7.4.5.2 Strength check of diaphragm

The ability of floor and roof diaphragms to transfer lateral forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm needs to be checked.

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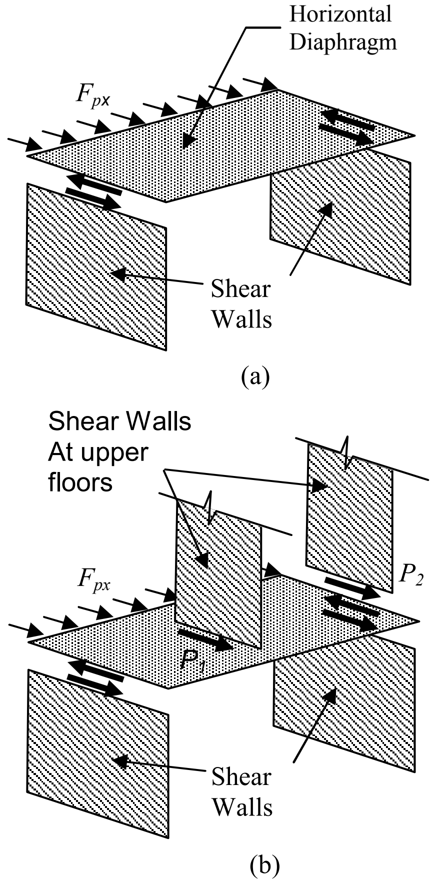

Figure C33: Forces acting on diaphragm system

7.4.5.3 – Deflection check of diaphragm

The deflection in plane of the diaphragm shall not exceed the permissible deflection of attached elements, e.g., walls. Permissible deflection of diaphragm shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the buildings. However, check for deflection is generally not required if span to width ratio of the diaphragm is less than 6.

C7.4.5.3 Deflection check of diaphragm

(a) Calculation of Diaphragm Deflection

The deflection of diaphragms as shown in figure C34 (a), should be determined by an adequate engineering analysis. However, it is realized that the calculation of diaphragm deflection is quite complex and also imprecise for various types of prevalent diaphragm construction. It is therefore necessary that a proper care is exercised in the choice of analysis method.

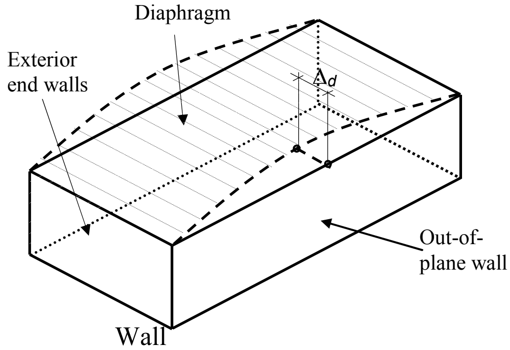

Figure C34(a): Diaphragm Deflection

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The behaviour of a flexible diaphragm under lateral load can be approximated as that of a wide flanged I beam with a large depth. The web of this I beam resists shear while the flange contributes in resisting moment generated by the uniform lateral load.

With this I beam approximation, deflection Δ_d of diaphragm of span, L, due to uniform lateral load as shown in figure C34 (b) is given by equation 7.1

$$\Delta_d = \frac{5wL^4}{384EI} \quad \text{.................................(7.1)}$$

For use in equation 7.1, the dimension of the diaphragm along the direction of lateral load is taken as the depth, D_d, of the I beam and the width of the flange is equal to six times the thickness of the supporting wall including depth of the diaphragm, measured equally above and below the centre of the diaphragm as shown in Figure C34(c).

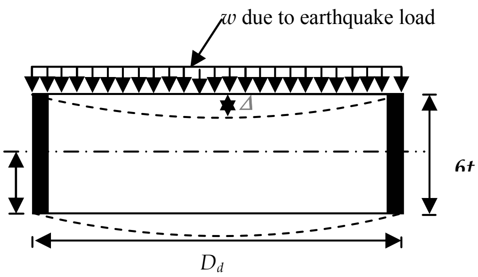

Figure C34(b): Diaphragm Deflection-I beam approximation

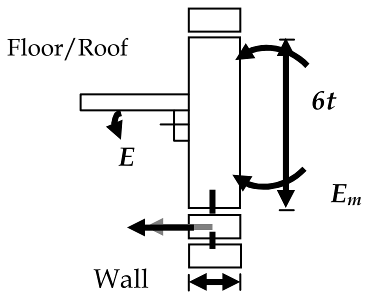

Figure C34(c): Diaphragm Deflection-I beam approximation

The moment of inertia of this beam about an axis perpendicular to the diaphragm is computed using equation 7.2.

$$I = n\frac{dD_d^{\,3}}{12} + \sum A\left(\frac{D_d}{2}\right)^2 \quad \text{...........................(7.2)}$$

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$$n = \frac{E_d}{E_m} \text{ ....(7.3)}$$

where Ed and Em are modulus of elasticity of diaphragm and masonry materials respectively

(b) Control of Diaphragm Deflection

As shown in Figure C34 (a) the in-plane deflection of a diaphragm due to seismic inertia forces causes out-of-plane deflection of some walls. The out-of-plane walls are thus subjected to flexural stresses in addition to stresses due to vertical loads; Excessive deflection of a diaphragm can seriously undermine the load carrying capacity of out-of-plane walls. The magnitude of diaphragm deflection should be limited so that walls are not subjected to extreme and damaging deflections.

One method to ensure that in-plane deflections of diaphragm are acceptable is by checking that flexural stresses so induced in the walls are within the permissible limits specified for the masonry as per IS 1905.

Top of the wall undergo lateral deflection Δ_d with the diaphragm which is the sum of the deflections due to bending moment (Δ_b) and deflections due to shear (Δ_v). For one storey building the deflection is caused primarily by bending. This deflection is caused by shear force at the level of diaphragm, which also generates a linearly varying bending moment up the height of the wall. To prevent masonry from developing tensile cracks and thereby making it unstable it is necessary that resultant stress remain within permissible limits.

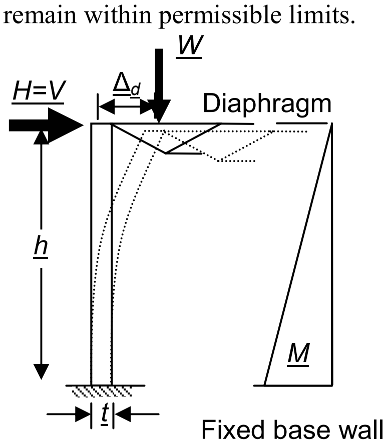

Figure C35: Deflection of the diaphragm

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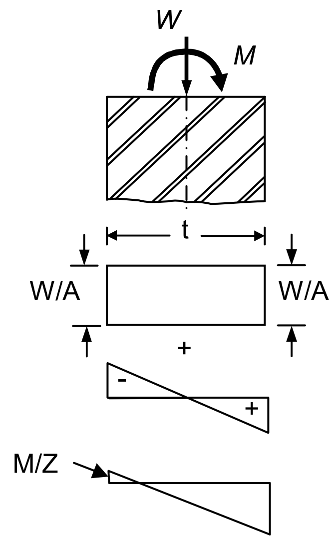
s

Moment at the base of wall is calculated using equation

$$M = Vh = \frac{3E_m I \Delta_d}{h^2} \text{ ....(7.4)}$$

and the resulting stresses are shown in Figure C39



Hence,

$$\frac{M}{Z} - \frac{W}{A} = f_t \qquad \text{(Tensile)}$$

$$-\left(\frac{M}{Z} + \frac{W}{A}\right) = fc \quad \text{(Compressive)}$$

$$\frac{M}{Z} = f_t + \frac{W}{A}$$

$$\frac{\mathbf{3}EI}{h^{\mathbf{2}}}\left(\frac{\Delta_d t}{\mathbf{2}I}\right) = \left(f_t + \frac{W}{A}\right)$$

$$\Delta_d = \frac{\mathbf{2}h^{\mathbf{2}}}{\mathbf{3}Et}\left(f_t + \frac{W}{A}\right)$$

Figure C36: Stresses at the base of wall

Taking allowable values of stresses in masonry, permissible deflection of diaphragm (wall) can be obtained as shown in figure C36. The net diaphragm deflection should be less than or equal to the permissible wall deflection.

Deflections can be controlled by controlling (h/t) ratio of the walls.

7.4.5.4 Shear Walls (in-plane loading)

(a) Shear wall Strengths

i. The shear wall strength shall be calculated as follows:

$$V_a = v_a Dt \text{ .... (7.1)}$$

where:

D = In-plane length of masonry wall (mm)

t = Thickness of wall (mm)

v_a = permissible masonry shear strength (MPa) given as shown below

$$v_a = 0.1 v_{te} + 0.15 \frac{P_{CE}}{A_n} \text{ ....(7.2)}$$

where:

C7.4.5.4 Shear Walls (in-plane loading)

(a) Shear Wall Strengths –

In-plane shear strength of the wall is allowable shear stress times the area of the wall. The shear strength is based on the average shear. The allowable shear stress formula is based on simple Mohr-Coulomb type formula for sliding failure along bed joints under low precompression.

The guideline prefers in-place shear push test (shove test) to determine the shear strength of masonry along bed joints. This test is discussed in detail in Appendix `A`.

The strength of piers between openings is determined by whether they are shear-critical or rocking-critical. Slender and lightly loaded piers are more likely to rock with flexural bed joint cracks at the top and the bottom of piers. The rocking capacity of a pier is based on the dynamic stability of pier where compression force provides

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v_{te} = Average bed-joint shear strength (MPa) determined from in-place shear test and not to exceed 0.6 MPa;

P_{CE} = Expected gravity compressive force applied to a wall or pier component stress;

A_n= Area of net mortared/grouted section (mm^2).

ii. The rocking shear strength shall be calculated as follows:

For walls without openings:

$$V_r = \left(0.50P_D + 0.25P_W\right)\frac{D}{H'} \quad ....(7.3)$$

For walls with openings:

$$V_r = 0.5P_D\frac{D}{H'} \quad ....(7.4)$$

(b) Shear Wall Acceptance Criteria – The acceptability of unreinforced masonry shear walls shall be determined as follows:

i. When $V_r < V_a$, [*Rocking controlled mode*: When the pier rocking shear capacity is less than the pier shear capacity]

$$V_{wx} < \sum V_r$$

ii. When $V_a < V_r$, [*Shear controlled mode*: Where the pier shear capacity is less than the pier rocking capacity] V_{wx} shall be distributed to the individual wall piers, V_p, in proportion to D/H and the following equations shall be met.

$$V_p < V_a$$

$$V_p < V_r$$

If $V_p < V_a$ and $V_p > V_r$ for any pier, the pier shall be omitted from the analysis and the procedure repeated using the remaining piers.

7.4.5.5 – Wall Anchorage

(a) Anchors shall be capable of developing the maximum of:

i. $2.5A_{hm}$ times the weight of the wall, or

ii. 3 kN per linear meter, acting normal to the wall at the level of the floor or

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the stabilizing force. Pier rocking is a storey phenomenon meaning that all piers in a storey should be rocking-critical.

In a given wall with a system of pier, spandrel and sill masonry, the stress in piers can be obtained by pier analysis of reinforced masonry. This method is explained in *IITK-GSDMA Guidelines on Reinforced Masonry.*

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roof.

Walls shall be anchored at the roof and all floor levels at a spacing of equal to or less than 1.8 m on center. At the roof and all floor levels, anchors shall be provided within 0.6 m horizontally from the inside corners of the wall. The connection between the walls and the diaphragm shall not induce cross-grain bending or tension in the wood ledgers.

(b) Collectors — Where walls do not extend the length of the diaphragm, collectors shall be provided. The collectors shall be able to transfer the diaphragm shears into the shear walls.

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Collectors –

It is essential that shear forces at the diaphragm edges must be 'collected' at the edges and then directly transferred to the masonry shear walls. When shear walls are present to the full depth of the diaphragm, there may be no need of collectors as shear load is uniformly distributed along the length of the shear wall. However, when the shear wall is present at one end of the diaphragm, collectors will be required to accumulate load from the diaphragm and will be designed as a tie and a strut member for the diaphragm shear force that it is transferring. Some typical collector elements are shown in Figure C37.

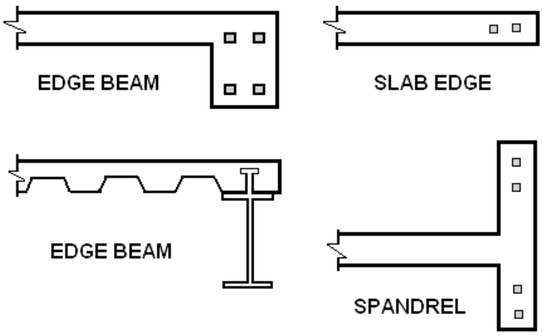

Figure C37: Collectors *(after FEMA 310)*

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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 improve strength, stiffness and/or ductility of deficient members and their connections.

8.2.1.3 –

Member level strengthening measures that enhance ductility of the member without significantly increasing its strength/stiffness

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C8 – Seismic Strengthening

C8.2 – Seismic Strengthening Options and Strategies

Increasing the redundancy of the lateral force resisting system under rehabilitation can be an effective and beneficial measure and should be considered while developing a rehabilitation strategy. The increased redundancy may prevent local failure or instability of one structure due to localized failure of a few elements of the system. It is preferable to have similar strength and stiffness for various elements of the redundant system.

C8.2.1 – Strengthening at Member Level

C8.2.1.1 –

If the deficient members are small in number an economical rehabilitation strategy could be improving member connectivity, member strength, and/or member deformation capacity.

C8.2.1.2 –

Here the modification measure will increase the capacity of member to resist more earthquake induced forces before getting damaged. Strengthening measures could include such as jacketing columns or beams and stiffening flexible diaphragms of masonry buildings.

C8.2.1.3 –

Member level strengthening measures to improve deformation capacity or ductility of a member will result in reduced damage without necessarily

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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.

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increasing strength of the member. Jacketing of RCC columns or beams is one such measure which improves the members' ability to deform without spalling or loss of anchorage or damage to the reinforcing bars. Another measure which can be adopted is reduction of cross section or weakening of selected members to increase the flexibility and deformation capacity.

C8.2.2 – Eliminating or Reducing Structural Irregularities

C8.2.2.1 –

Stiffness, mass, and strength irregularities may be detected either by reviewing the results of a linear analysis, examining the distribution of structural displacements and Demand Capacity Ratios (CDRs), or reviewing the results of a nonlinear analysis by examining the distribution of structural displacements and inelastic deformation demands. If the distribution of values of structural displacements, DCRs, or inelastic deformation demands predicted by the analysis is non uniform with disproportionately high values within one storey relative to the adjacent storey, or at one side of a building relative to the other, then an irregularity exists.

C8.2.2.2 –

Effective corrective measures for removal or reduction of irregularities, such as soft or weak stories, include the addition of braced frames or shear walls within the soft or weak storey. Torsional irregularities can be corrected by the addition of moment frames, braced frames, or shear walls to balance the distribution of stiffness and mass within a storey. Discontinuous components such as columns or walls can be extended through the zone of discontinuity. Such irregularities are often, but not always, caused by the presence of a discontinuity in the structure, as for example, termination of a perimeter shear wall above the first storey. Removal or lessening of existing irregularities may be an effective rehabilitation strategy if a seismic evaluation shows that the irregularities result in poor seismic performance. However, removal of discontinuities may be inappropriate in the case of historic buildings, and the effect of such alterations on important historic features should be considered carefully.

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8.2.2.3 –

Seismic gaps (or movement joints) can be created 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 Global 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.

8.2.4 – Supplemental Damping and Isolation

a) Seismic 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.

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C8.2.2.3 –

Expansion joints can be created to transform a single irregular building into multiple regular structures; however, care must be taken to avoid the potential problems associated with pounding.

C8.2.3 – Strengthening At Global Level

C8.2.3.1 –

Global strengthening of the structure may be an effective rehabilitation strategy where a seismic evaluation shows unacceptable performance due to overall structural strength. This can be identified when the onset of global inelastic behavior occurs at levels of ground shaking that are substantially less than code design levels. By providing supplemental strength to such a lateral-force-resisting system, it is possible to raise the threshold of ground motion at which the onset of damage occurs. Shear walls and braced frames are effective elements for this purpose, but they may be significantly stiffer than the structure to which they are added. This requires their design to provide nearly all of the structure's lateral resistance. This is desirable as the existing members will probably have very little inelastic strength capacity.

C8.2.4 – Supplemental Damping and Isolation

a) Seismic Isolation

Seismic isolation may be an effective rehabilitation strategy if the results of a seismic evaluation show deficiencies attributable to excessive seismic forces or deformation demands, or if it is desired to protect important contents and nonstructural components from damage. When a structure is seismically isolated, compliant bearings are inserted between the superstructure and its foundations. This produces a system (structure and isolation bearings) with a nearly rigid body translation of the structure above the bearings. Most of the deformation induced in the isolated system by the ground motion occurs within the compliant bearings, which are specifically designed to resist these concentrated displacements. Most bearings also have excellent energy dissipation characteristics (damping). Together, this results in greatly reduced demands on the existing elements of the structure, including contents and non-structural components. For this

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reason, seismic isolation is often an appropriate strategy to achieve enhanced rehabilitation objectives that include the protection of historic fabric, valuable contents, and equipment, or for buildings that contain important operations and functions. This technique is most effective for relatively stiff buildings with low profiles and large mass. It is less effective for light, flexible structures.

b) Supplemental Energy Dissipation

Installation of supplemental energy dissipation devices may be an effective rehabilitation strategy if the results of a seismic evaluation show deficiencies attributable to excessive deformations due to global structural flexibility in a building. Many available technologies allow the energy imparted to a structure by ground motion to be dissipated in a controlled manner through the action of special devices—fluid viscous dampers (hydraulic cylinders), yielding plates, or friction pads—resulting in an overall reduction in the displacements of the structure. The most commonly used devices dissipate energy through frictional, hysteretic, or viscoelastic processes. In order to dissipate substantial energy, dissipation devices must typically undergo significant deformation or stroke, which requires that the structure must experience substantial lateral displacements. Therefore, these systems are most effective in structures that are relatively flexible and have very little inelastic deformation capacity. Energy dissipaters are most commonly installed in structures as components of braced frames. Depending on the characteristics of the device, stiffness is added to the structure as well as energy dissipation capacity (damping). In some cases, although the structural displacements are reduced, the forces delivered to the structure can actually be increased. However, this strategy is technically complex but less costly compared to base isolation.

The designer must illustrate by detailed analysis that these devices provide sufficient protection to the buildings and equipment as described in IS 1893 (Part1). Performance of locally assembled isolation and energy absorbing devices should be evaluated experimentally before they are used in practice. Design of buildings and equipment using such device should be reviewed by competent authority.

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8.3 – Methods of Analysis and Design for Strengthening

8.3.1 – Design Criteria

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

8.3.2 – 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.

8.3.3 – Analysis Options

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

8.4 – Strengthening Options for RC Framed Structures

8.4.1 –

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.

(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

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C8.3 - Methods of Analysis and Design for Strengthening

C8.4 – Strengthening Options for RC Framed Structures

C8.4.1 –

Jacketing can be applied in cases of heavily damaged frame members or in cases of insufficient member strength. Member flexural strength increases with the enlargement of the concrete area and by adding new longitudinal reinforcement. Shear strength, and especially ductility, is improved by better confinement with close transverse reinforcement – ties or steel strips. The enlarged sections of repaired or strengthened members can result in considerable stiffness change of the different members, causing a redistribution of seismic moments and affecting the seismic forces in different parts of the building structure.

A steel jacket fitted around an RC member increases its shear strength, improves deficient lap splices and increases ductility through confinement. For the purpose of improving lap splices or increasing ductility. Steel jackets are applied over only plastic hinge or lap splice regions. To increase shear strength of member steel jackets extend over the full length of the member.

The shear strength of member, steel jackets must extend over the full length of the member. FRPs

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on the outer surface of the RC member.

8.4.1.1 – 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:

(i) The seismic demand on the columns, in terms of axial load (*P*) and moment (*M*) is obtained.

(ii) The column size and section details are estimated for *P* and *M* as determined above.

(iii) The existing column size and amount of reinforcement is deducted to obtain the amount of concrete and steel to be provided in the jacket.

(iv) .

Increase the amount of concrete and steel actually to be provided as follows to account for losses. $A_c = \frac{3}{2} A_c'$ and

$$A_s = \frac{4}{3} A_s'$$

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.

(v) 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:

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are mainly used for enhancement of shear capacity of deficient elements like column, beams and shear walls by providing externally bonded FRPs in the hoop direction. FRP jackets placed along beam or column perimeters increase the ductile behavior of flexural plastic hinges at beam or column ends through added confinement. They also prevent rapid strength degradation due to vertical cracks in cover concrete and cover spalling.

C8.4.1.1 – RC Jacketing of Columns

The amount of actual concrete and reinforcement provided in the RC jacket is increased in order to account for defects in workmanship, due to the small thickness of the jacket and the inherent difficulties in properly placing the new longitudinal reinforcement. Closely spaced stirrups help in avoiding tension cracking along the jacket. Provision of bent-down bars allows direct transfer of forces between longitudinal reinforcement. They also provide good anchorage between old and new concrete. If a good bond between the existing column and the new concrete of the jacket exists, the stiffness of the strengthened column should be determined as that of a monolithic member with an equivalent composite cross section.

The design method for concrete jacketing is based on provisions in Eurocode and UNIDO/UNDP documents.

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

where

f_y = yield strength of steel

f_{ck} = cube strength of concrete

d_h = diameter of stirrup

t_j = thickness of jacket

(vi) 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.

(vii) Dowels which are epoxy grouted and bent into 90^o hook can also be employed to improve the anchorage of new concrete jacket.

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This method of concrete jacketing which involves installation of longitudinal and bent down bars leads to corrosion of bars in the existing column because of exposure of old bars and different grades of old and new steel of different composition. Also, existing columns retain large part of the gravity load capacity even after undergoing heavy damage. Therefore, it can be argued that the transfer of axial force is not critical. It is better to rely only on friction force at the interface of old and new concrete overlay for transfer of shear forces. Friction is assisted by the compressive stress developing at the interface because old member restrains the shrinkage of new overlay in radial and circumferential direction (refer to FIB 2003).

(viii) In order to transfer the additional axial load from the old to the new longitudinal reinforcement, bent-down bars intermittent lap welded to bars of jacket and longitudinal bars in the existing column can be used. . Moreover, bent-down bars help in good anchorage between existing and new concrete Figure(x).

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

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

where

ΔP =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

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(k) The minimum specifications for jacketing of columns are:

(i) 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.

(ii) 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 135^0bends and 10ϕ leg lengths.

(iii) Minimum jacket thickness should be 100 mm.

(iv) Lateral support to all the longitudinal bars should be provided by ties with an included angle of not more than 135°.

(v) Minimum diameter of ties should be 8 mm and not less than 1/3 of the longitudinal bar diameter.

(vi) 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 should not exceed 100 mm. Preferably, the spacing of ties should not exceed the thickness of the jacket or 200 mm whichever is less.

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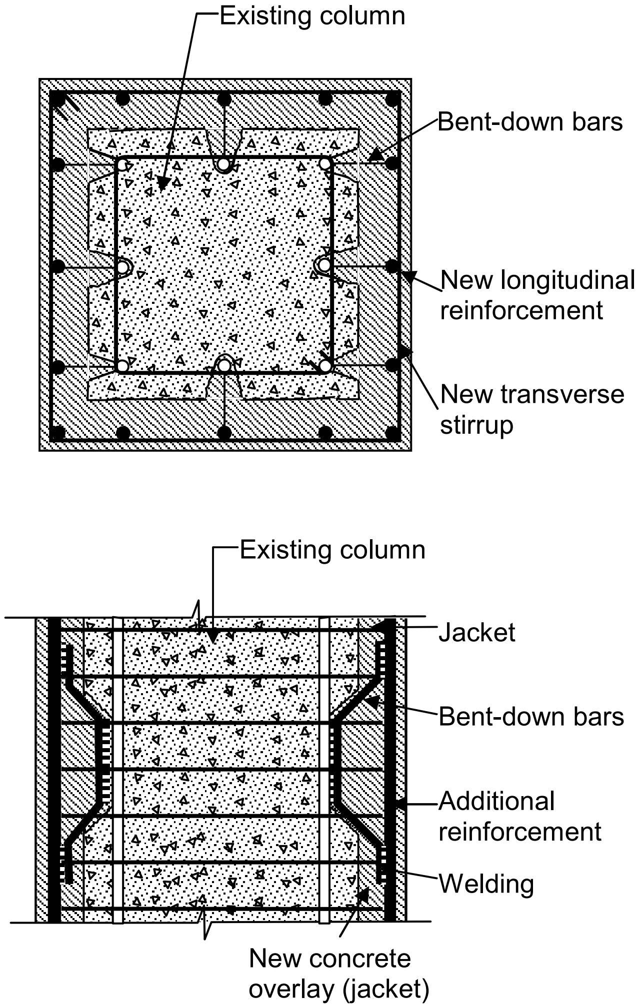

Figure 2: Reinforced Concrete Jacketing

8.4.1.2 –Addition of RC 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 as,

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

C8.4.1.2– Addition of RC Shear wall

Incorporation of new structural components in an existing building will change the dynamic behavior of the whole structure considerably during the earthquake. The choice of the type, number and size of the added elements depends on the particularities of the existing structure and the functional layout of the building.

Shear walls, because of their large stiffness and lateral strength, may provide the most significant part of the earthquake resistance of the building structure. Shear walls are used for strengthening RC frames, especially with open storeys. Shear walls can also be added with stiffness that is considerably higher than that of the old structure. This is feasible for buildings where the interior space is already partitioned and existing partitions

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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).

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.

η = 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.

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could be replaced by earthquake resisting shear walls, or new walls could be added with no significant disturbance of the functional layout. In such cases, the new shear walls should have sufficient strength and stiffness to provide the entire lateral force resistance. Shear walls considerably add to the mass of the structure. The existing structure must be compatible with the strengthening elements and able to deform without failure in future earthquakes.

Designers must check for the following:

(i) Are the existing foundations adequate for the enhanced vertical compression/tension loads?

(ii) Is existing column steel detailing adequate to resist shear wall bending moments?

(iii) Can floor slabs work as diaphragms in conjunction with new walls?

(iv) Is column tie detailing adequate?

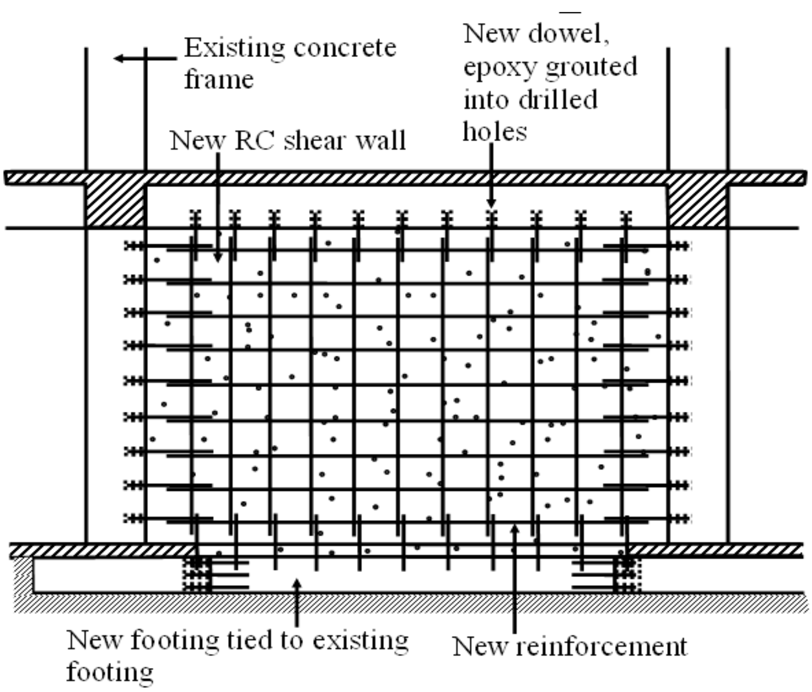

Figure 3 :Adding new shear walls

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8.4.1.3 –Addition of steel bracing

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 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.

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C8.4.1.3– Addition of steel bracing

When a large number of openings are required, diagonal bracing can be added as strengthening measures. Steel diagonal braces can be added within the concrete frame to form a vertical truss of existing beams and columns and new diagonals. One or two diagonal braces can be installed depending on the geometry and other considerations. The critical detail is providing proper attachment of the brace to the frame structure joints. The different configurations available are shown in Figure C38. Existing frame members, foundation and detailing must be capable of resisting all loads arising from the new braced frame action.

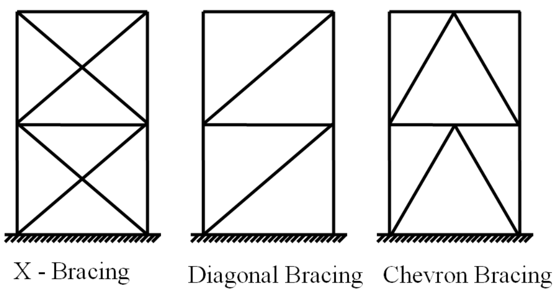

Figure C38: Steel bracing details

In braced frames, slender members and non-compact thin sections result in premature buckling instabilities at much lower level of seismic shaking, resulting in poor hysteretic response. Relatively stockier braces with sections of large width to thickness ratio have shown to possess adequate energy dissipation capacity even in post-buckling stage.

Chevron braces though very popular may subject the floor girder to carry additional bending moment when one of the braces buckles and its strength drops causing net unbalanced vertical force acting on the floor girder where the braces meet.

The gusset plate is the most critical component of thebrace connection. It should have enough strength when the brace buckles in the plane of the frame. It should provide for formation of hinge line, if brace buckles out-of-plane.

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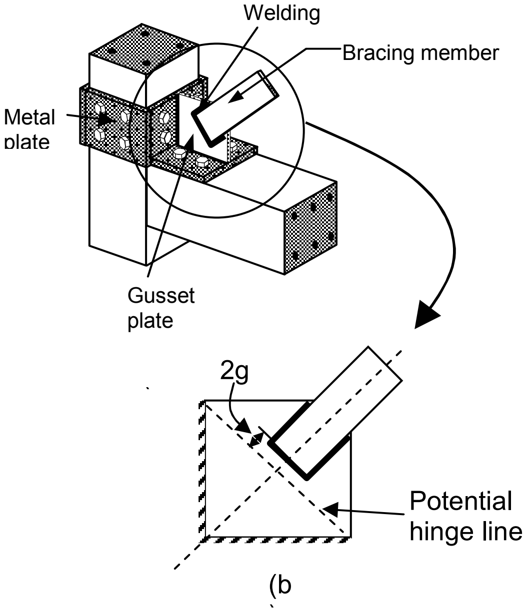

Figure 4: (a) Brace connection details (b) Gusset base connection detail

8.4.1.4 –

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 steelframe, which is then placed in frame bay and firmly connected.

C8.4.1.4 –

These kind of pre-fabricated bracing sub-assemblages consist of a heavy rectangular rim with bracing inside. These sub-assemblages are placed in the bays of the existing RC frame between adjacent beams and columns as shown in Figure 6. To accommodate variations in the dimensions of RC frame, a gap of sufficient tolerance is provided around the rim. Connection of the prefabricated sub-assemblages and the surrounding frame members is achieved through a system consisting of

a) closely spaced anchors in the form of headed bolts/sheets studs welded to the outer surface of the metal rim and projected into the gaps and

b) fasteners (normally epoxy-grouted and closely spaced) installed in the interior face of outer frame spiral reinforcement is often inserted along the mortar joint, between the fasteners and the headed studs).

c) cement mortar/grout filling the gap between the rim and the RC frame

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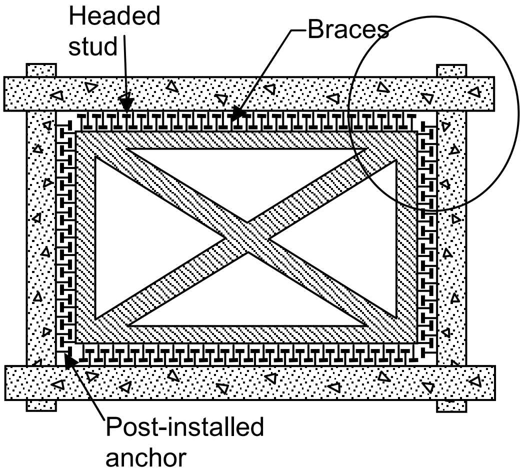

Figure 6(a): Prefabricated steel bracing

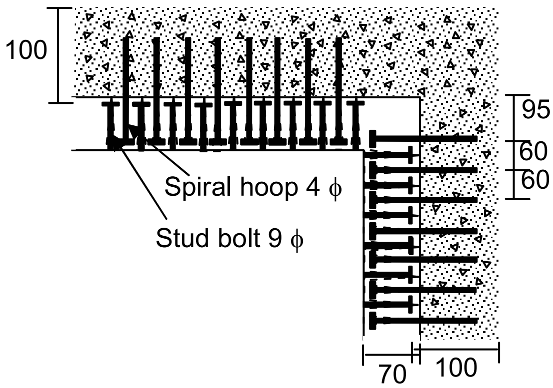

Figure 6(b): Detailing of corner view of figure 6(a)

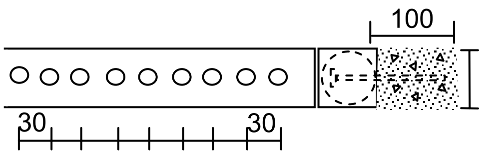

Figure 6(c): Detailing of top view of figure 6(a)

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Appendix A

A1 - Unreinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

The special procedure applies to the evaluation of unreinforced masonry bearing wall buildings with the following characteristics:

CA1 - Unreinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

The behavior mentioned in section C7.4.5 is considered in this section which results in a different vertical distribution of lateral forces than suggested by parabolic variation of IS:1893 for buildings and also different base shears. For the short spans of the flexible diaphragms with substantial strength, the base shear of the two procedures would be identical.

The provisions of special procedure for unreinforced masonry buildings with flexible diaphragms are based on extensive ABK research study which also forms basis for many US building codes, such as UCBC, NEHRP, ASCE, IBC, etc. The study showed that the static methods of lateral load analysis can not predict behaviour of flexible diaphragms because in the event of moderate-to-severe seismic shaking, the response of diaphragms is dominated by the non-linear hysteretic characteristics.

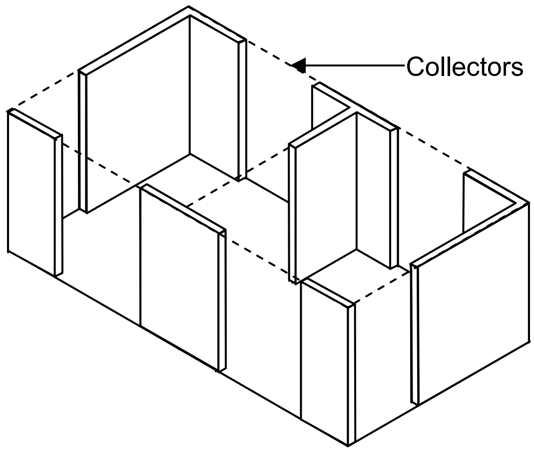

Figure AC1: Collector functions in a box system

The edge members called chords are used to take the tension and compressive force at the edges. The collection of forces into the diaphragm or the distribution of loads to vertical elements may induce a stress beyond the capacity of a deck alone. Figure AC1 shown a building in which a continuous roof diaphragm is connected to a series of shear wall. Load collection and force transfers require that some force be dragged along the dotted lines shown in figure. For outside walls the edge framing used for chords can do double service for

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the purpose.

1. Flexible diaphragms at all levels above the base of the structure. These are diaphragms of wood construction and similar types with inadequate connections among diaphragm elements for seismic loading.

The building must have flexible diaphragms at all storey levels to qualify for the special procedure and the seismic loading used in this section is exclusively for flexible diaphragms. The definition of rigid diaphragms and flexible diaphragms given in Sec. 3 are broad based. However, a more restrictive definition of flexible diaphragm is specified for the special procedure.

Existing unreinforced masonry bearing wall buildings use materials that do not fit into one of these narrow definitions. In such cases, a detailed analysis is warranted with adequate modeling of their stiffness characteristics including stiffness degradation, which will be specific for a unique structural system.

2. A minimum of two lines of walls in each principal direction, except for single-storey buildings with an open front on one side.

Two lines of vertical lateral-load-resisting elements of masonry along each axis of the building are required. If structural steel frames or reinforced concrete bracing is used for one or both of these vertical elements, a relative rigidity check of the bracing versus a masonry shear wall should be made to ensure that the building system will behave as a rigid wall-flexible diaphragm system in earthquakes.

Open front is meant to indicate those buildings which have a masonry shear wall, frame, braced frame at one side of the exterior wall line.

A1.1 – Cross Walls

(a) Cross walls shall not be spaced more than 12 m on center measured perpendicular to the direction under consideration and should be present in each storey of the building. Cross walls shall extend the full storey height between diaphragms.

CA1.1 – Cross Walls

Cross wall is a light and flexible frame, and/or interior partition wall parallel to direction of the loading which extends from floor diaphragm to floor diaphragm. They can effectively control excessivse deflections of flexible diaphragms and differential movements of adjacent floors. Further, they add to energy dissipation potential of the system by inelastic behaviour of their material. Though they participate in resisting lateral loads, they are not treated as masonry shear walls because their in-plane stiffness is not comparable with that of masonry walls. Steel frames with much lower lateral stiffness than in-plane shear stiffness of exterior masonry shear walls can qualify as cross walls.

The restriction on spacing of cross walls is to limit the higher mode response of the diaphragms. They also enhance the dynamic stability of out-of-plane walls and when used to increase the permissible height-to-thickness ratio, these cross walls must extend to full height of the building.

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(b) Exceptions:

1. Cross walls need not be present at all levels.

2. Cross walls that meet the following requirements need not be continuous:

I. Shear connections and anchorage at all edges of the diaphragm shall meet the code requirements;

II. Cross walls shall have a shear capacity of $0.6A_{hm}\Sigma W_d$ and shall interconnect the diaphragm to the foundation;

i. Diaphragms spanning between cross walls that are continuous shall comply with the following equation:

$$\frac{2.5A_{hm}W_d + V_{ca}}{2v_u D_d} \leq 2.5 \quad \text{...... (A.1)}$$

where,

$A_{hm} = A_h U$

A_h = Horizontal seismic coefficient as per IS 1893 (Part 1).

U = (reduced) Useable life factor

v_u = unit shear strength of the diaphragm

Note: The value of S_a/g shall be taken as 2.5 in the formula for A_h for unreinforced bearing wall buildings considering that their fundamental period is in short period range (or acceleration sensitive region) of the design response spectrum.

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Exceptions –

This relaxation is meant to permit the usage of cross walls which extend only between the roof and floor diaphragm. Such cross walls combine these diaphragms into a “Super Diaphragm” for the diaphragm analysis discussed in Section A1.2.

This condition is permitted to allow those buildings where many cross walls are not continuous to the ground, due to requirements of open spaces at the ground floor. The first floor diaphragm should have considerable strength and stiffness to limit the diaphragm deformation between the cross walls, which themselves should have stiffness comparable to shear walls. Further, the loading to the first floor diaphragm will consist of its tributary load and the total shear capacity of the cross walls above the first floor.

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Table 4: Allowable values for diaphragms and cross walls

Existing Materials	Allowable Values
Horizontal diaphragms	
a. Roofs with straight sheathing and roofing applied directly to the sheathing.	1.4 kN/m
b. Roofs with diagonal sheathing and roofing applied directly to the sheathing.	3.6 kN/m
c. Floors with straight tongue-and-groove sheathing.	1.4 kN/m
d. Floors with straight sheathing and finished wood flooring with board edges offset or perpendicular.	7.3 kN/m
e. Floors with diagonal sheathing and finished wood flooring.	8.7 kN/m
f. Plywood sheathing applied directly over existing straight sheathing with ends of plywood sheets bearing on joists or rafters and edges of plywood located on center of individual sheathing boards.	3.2 kN/m
Crosswalls	
a. Partition walls of plywood, timber, etc. not acting as shear walls.	4.3 kN/m

(c) Shear Capacity — Within any 12 m measured along the span of the diaphragm, the sum of the cross wall shear capacities shall be greater than or equal to 30% of the diaphragm shear capacity of the strongest diaphragm at or above the level under consideration.

(d) Existing Cross Walls — Existing cross

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Allowable values for diaphragms and cross walls –

The materials for diaphragms and cross walls and their allowable force values included in Table 4 are taken from the ABK research study and the UCBC. However, these diaphragm and cross wall systems have limited applications in Indian conditions, but serve as an indicator for designers to choose a suitable value for existing materials. There is urgent need to carry out experimental studies on diaphragm systems typically found in the country to establish appropriate allowable values.

Shear Capacity –
The stiffness of nailed or jointed systems such as flexible diaphragms and cross walls is generally linked to their peak strength. As a result, the minimum strength of a cross wall is related to the strength of the diaphragm to which it is connected and being deformed with. For cross walls to be effective in controlling diaphragm displacement, they should at least have 30 percent of the diaphragm capacity.

Existing Cross Walls

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walls when perforated with openings shall have a length-to-height ratio equal to or greater than 1.5 between openings. In cases where these cross walls are continuous across diaphragm level above, the adequacy of their connection with diaphragm need not be investigated.

(e) New Cross Walls — New cross walls shall be designed to resist the overturning moment equal to cross wall shear capacity times the storey height, which may not be accumulated for more than two stories. Its connections to diaphragm shall develop the shear capacity of the cross wall.

(f) Other Cross Wall Systems — Other cross wall system including moment resisting or braced frames can be designed as cross walls as long as their load-deformation behaviour is similar to the existing cross walls.

A1.2 – Flexible Diaphragms

(a) Acceptability Criteria – A diaphragm of given span is acceptable if the intersection of its span between walls *L*, and the demand-capacity ratio *DCR*, is located within Region 1, 2, or 3 as shown in Figure 2.

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Often existing cross walls have openings for doors and portions of the cross walls which have a height to length ratio less than 1.5 are not used in the cross walls strength calculations.

New Cross Walls –

Adequate connections between new cross walls and diaphragms are provided, which include the connection at the end of cross wall to the foundation or other cross wall, in order to protect the floor framing from overturning forces. The gravity loads on the floor can not be relied on resisting these overturning forces, which need not be summed up for more than two stories including the level of calculation.

Other Cross Wall Systems –

A moment resisting or braced frame can be designed to act as cross wall. Whereas a moment frame acts more like flexible 'yielding' system adding damping, a braced frame is rather stiffer system acting as a shear wall. The choice of cross wall system should be compatible with the lateral stiffness and strength of the existing structural system. Generally, the yielding drift of the frames used as cross walls should not exceed 25 mm, as it provides the required stiffness and a limit for non-linear behaviour. The design lateral load for such frames will be same as required of other cross walls. When cross wall is designed as shear wall, the design load will correspond to its tributary area of diaphragm and storey drift will not exceed 0.015 times the storey height.

CA1.2 – Flexible Diaphragms

Acceptability Criteria

Conventional diaphragm analysis is not to be carried out for flexible diaphragm under Special Procedure. A diaphragm is acceptable with respect to both strength and stiffness requirements, if its span length *L* and strength demand-capacity ratio (*DCR*) fall within Regions of 1, 2, or 3 of Figure 2.

Figure 2 has been taken from UCBC [1991] which is based on the methodology developed by ABK research study on existing masonry buildings. It is a plot of the estimated dynamic displacement of the center of the diaphragm of 127 mm measured relative to the shear walls that are shaking the diaphragm. The ground motions used for these calculations were scaled to the 5% damped response spectrum of ATC-3 having 0.40*g*

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effective peak ground acceleration and 0.30 m/s peak ground velocity. This spectrum is mostly similar to IS1893 (Part1) design spectrum in Zone V in terms of energy content. However, there is urgent need to develop Figure 2 corresponding to the design spectrum of IS 1893 (Part 1). The analysis of the DCR of diaphragms is only required in the seismic zones V and IV. The probable ground motions of seismic zone IV are accommodated by the substitution of A_{hm} in the formulae of DCR.

(b) Demand - Capacity Ratios – Demand-capacity ratios shall be calculated for a diaphragm at any level in accordance with the following equations:

Demand-Capacity Ratios –

The flexible diaphragms of unreinforced masonry buildings will be subjected to inelastic deformations for moderate to severe shaking and the special procedure analysis takes into account the behavior of yielding diaphragms. The required strength and stiffness is controlled by limiting the drift or the dynamic displacement of the diaphragm which is implemented by the calculation of demand-capacity ratios and the Figure 2.

The ratio of total seismic shear force that is acting on the diaphragm and total shear capacity of both ends of diaphragm is referred as the demand-capacity ratio (DCR). The total shear on diaphragm is taken as total load that is being transferred by the particular diaphragm multiplied by the spectral response factor.

The equations for DCR assume that the resistance offered by both ends of the diaphragm is nearly equal; however, in case of significant difference, the total capacity of the diaphragm can be suitably modified. A conservative approach is to use two times the capacity of the shortest end.

Furthermore, the DCR equations assume that the diaphragms are nearly rectangular in shape. These equations can be used for buildings with plan irregularities with respect to parallel walls, provided ties and collector elements are present to transfer the load to shear wall. In such cases, the diaphragm span is the distance between the shear walls. The following four conditions generally arise in calculations of DCR.

i. Diaphragms without cross walls at levels immediately above or below:

$$DCR = \frac{2.5 A_{hm} W_d}{\sum v_u D_d} \quad \text{...... (A.2)}$$

(i) For diaphragms with no qualifying cross walls above or below the diaphragm level, the maximum displacement at the center of the diaphragm (Point *a* in Figure AC2) in the direction of loading with respect to the end masonry shear wall (Point *b* in Figure AC3) is controlled by dynamic properties of the yielding diaphragm. The tributary weight of the diaphragm that can be associated to end masonry shear wall is limited by the yield capacity of the diaphragm. The DCR in equation represents the product of total load that is being transferred and

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base shear coefficient which is divided by total shear capacity of the diaphragm.

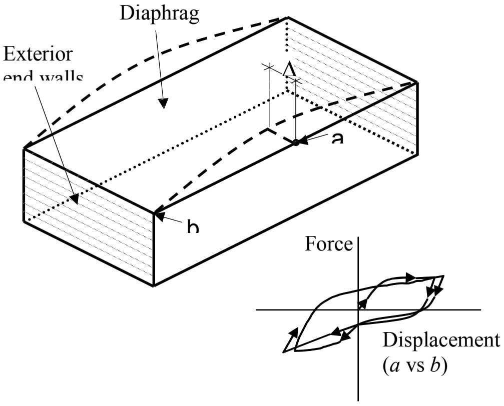

Figure AC2:Diaphragm Actions without cross wall

ii. Diaphragms in a one-storey building with cross walls:

iii. $DCR = \dfrac{2.5 A_{hm} W_d}{\sum v_u D_d + V_{cb}}$ ...... (A.3)

(ii) The cross walls in a single storey building provide an additional load path between the roof diaphragm and the ground. As shown in Figure AC3, the cross wall capacity can influence the diaphragm displacement (Point *a vs b,* and *a vs c*). Therefore, the capacity of the diaphragm is taken as summation of the capacity of both ends of the diaphragm and the cross walls below the diaphragm.

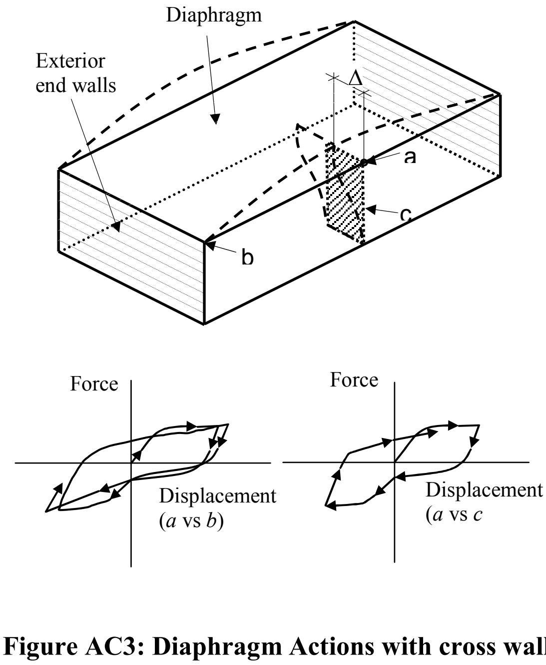

Figure AC3: Diaphragm Actions with cross wall

iv. Diaphragms in a multi-storey building with cross walls at all levels:

v. $DCR = \dfrac{2.5 A_{hm} \sum W_d}{\sum\left(\sum v_u D_d + V_{cb}\right)}$ ...... (A.4)

(iii). When in a multi-storey building qualifying cross walls are present in all storeys, then the demand-capacity ratio calculation begins from the roof level and continues to floors below. At each level the demand is the sum of the diaphragm loads above the floor level under consideration

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multiplied with base shear coefficient. The capacity will be sum of the diaphragm capacities of levels above and cross wall capacities below the level under consideration. Only qualifying cross walls shall be considered in the DCR calculation.

A cross wall capacity must be maintained vertically in the multi-storey building. In such cases, V_{cb} utilized for diaphragm analysis at any upper storey should be added to the W_d of the storey below of that storey. For the building shown in Figure AC4, level 3 and 4 must be added to W_d at level 3 for the diaphragm analysis at that level.

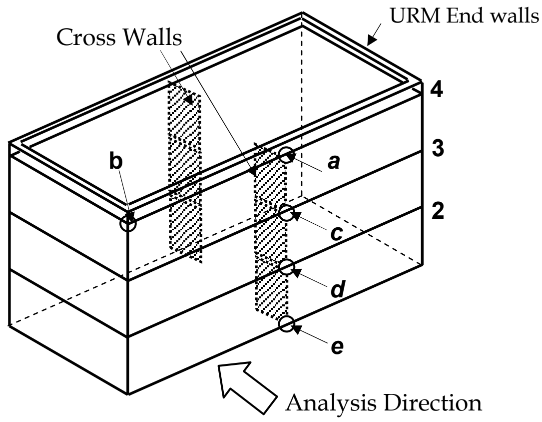

Figure AC4: Cross Walls

vi. Roof diaphragms and the diaphragms directly below if coupled by cross walls:

vii. $$DCR = \frac{2.5A_{hm}\sum W_d}{\sum\left(\sum v_u D_d\right)}$$ ...... (*A*.5)

viii. **Chords –** An analysis for diaphragm flexure need not be made and chords need not be provided.

(iv). In general roof diaphragms are quite flexible in comparison to floors below and when cross walls are present between the roof and floor below, these two diaphragms are considered coupled and their capacities can be added. It is not necessary that cross walls be present in the stories below.

Chords –

Conventional analysis of diaphragms assumes diaphragms are subjected to flexural deformations and significantly large forces develop in the edge members (chords), as shown in Figure AC6. Flexible diaphragms of Special Procedure do not undergo significant flexural deformations and as a result considerable chord forces do not develop. The primary response of diaphragm is through its yielding in the end zones adjacent to masonry shear walls. Having stiff edges around the diaphragm and openings do help to maintain internal continuity and edge restraint, but are not necessary.

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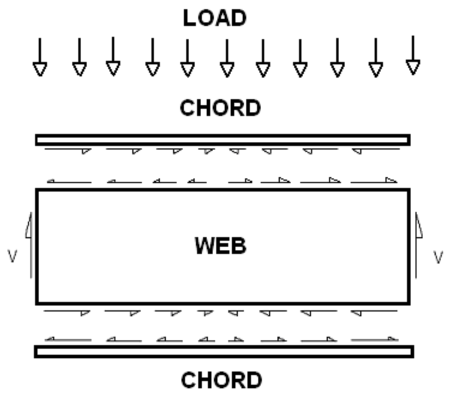

Figure AC5: Chords *(after FEMA 310)*

(d) Collectors — Please refer to general procedure for flexible diaphragm

Collectors –

(e) Diaphragm Openings – Diaphragm forces at corners of openings shall be investigated. The diaphragm shall have the tensile capacity to develop the strength of the diaphragm at opening corners. The demand-capacity ratio shall be calculated and evaluated in accordance with Section A1.2 (b) for the portion of the diaphragm adjacent to an opening using the opening dimension as the diaphragm span and when openings occur in the end quarter of the diaphragm span the diaphragm capacity shall be based on the net depth of the diaphragm.

Diaphragm Openings –

The equations of DCR for the analysis of diaphragms assume a rectangular shape and uniform distribution stress distribution is assumed. However, the presence of openings increases the flexibility of the diaphragms and can effect concentration of stresses in areas around the openings. It is necessary that the shear around the opening is transferred by collectors to vertical shear walls.

If openings are present close to the end shear walls, for example, opening *a* in the Figure AC5, diaphragm stiffness is significantly affected and a net depth D_1 shall be used in the capacity calculation rather than the full depth. However, when the opening is present elsewhere, such as opening b in Figure AC4, the full depth of diaphragm D_d can be used.

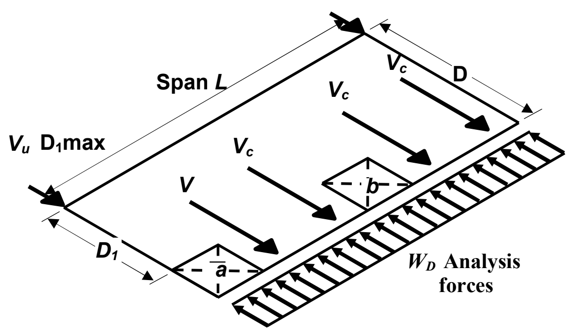

Figure AC6: Diaphragm Openings *(After ABK)*

(a) Diaphragm Shear Transfer— Diaphragms shall be connected to shear walls at each end and shall be able to develop the minimum of the forces as

Diaphragm Shear Transfer –

The diaphragm shear is to be transferred to vertical shear elements (walls) through positive connection. The capacity of these shear connectors are limited

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calculated below:

$$V_d = 1.5 A_{hm} C_p W_d \text{ ...... (A.6)}$$

$$V_d = v_u D_d \text{ ...... (A.7)}$$

Table 5: Horizontal force factor, C_p

Configuration of Materials	C_p
Roofs with straight or diagonal sheathing and roofing applied directly to the sheathing, or floors with straight tongue-and-groove sheathing	0.50
Diaphragm with double or multiple layers of boards with edges offset, and blocked structural panel systems	0.75

A1.3– Shear Walls (In-plane loading)

(a) Shear Wall Actions – The wall storey force distributed to a shear wall at any diaphragm level shall be determined in accordance with the following equations:

iii. For buildings without cross walls:

$$F_{wx} = A_{hm}\left(W_{WX} + 0.5W_d\right) \text{ ...... (A.8)}$$

but not exceed,

$$F_{wx} = A_{hm} W_{wx} + v_u D_d \text{ ...... (A.9)}$$

iv. For buildings with cross walls in all levels:

$$F_{wx} = 0.75 A_{hm}\left(W_{wx} + 0.5W_d\right) \text{..(A.10)}$$

$$F_{wx} = 0.75 A_{hm}\left(W_{wx} + \sum W_d \left(\frac{v_u D_d}{\sum\left(\sum v_u D_d\right)}\right)\right) \text{....(A.11)}$$

and need not exceed,

v. $F_{wx} = 0.75 A_{hm} W_{wx} + v_u D_d$ ....(A.12)

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to about one-half of total load of the diaphragm and normal walls, multiplied by base shear coefficient and a factor C_p, which represents the amplification of ground motions by diaphragms. This limit on the required capacity of connectors is result of expected non-linear (yielding) behaviour of diaphragm. The factor C_p ranges between 0.5 and 0.75 depending on the materials used for diaphragms; higher value is used for the stiffer diaphragm because greater amplification of motions is expected. Table 5 needs to be expanded to include other materials which are more often used in the country.

The second limit on the shear connection capacity equal to the shear capacity of the diaphragm ensures that the provided connection strength is adequate to causes shear yielding in the diaphragm.

CA1.3 - Shear Walls (In-plane loading)

Shear Wall Actions –

The vertical distribution of storey shears in shear walls for buildings of flexible diaphragms of special procedure differs from that of the buildings with rigid diaphragm. In Special Procedure, this distribution depends on shear capacity of the diaphragms rather than assumed linear or parabolic distribution of base shear.

(i) Where no cross walls are present, the storey force at a diaphragm level is base shear coefficient multiplied by the tributary weight of the wall at the level and half the dead weight of the diaphragm. However, the shear force transmitted to the shear wall will be limited by a shear force which is equal to base shear coefficient times the tributary weight of the shear wall plus the shear capacity of the diaphragm at the level of consideration.

(ii) In the presence of qualifying cross walls, the shear force transferred to end shear wall has been reduced to account for shear which can be transmitted by the cross walls.

However, the distribution of diaphragm loading to shear wall is in proportion to ratio of the strength of the diaphragm at the level of consideration to the total strength of all diaphragms meeting at the shear wall. In other words, the diaphragm couples total diaphragm load to a shear wall in proportion to their relative rigidity, expressed in terms of their shear capacity, in Equation 1.11. Equation 1.12 limits the shear load transfer based on the diaphragm shear yielding capacity.

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vi. The wall storey shear shall be calculated as below:

vii. $V_{wx} = \sum F_{wx}$ ….(A.13)

A1.4 – Wall Anchorage

(c) Anchors shall be capable of developing the maximum of:

iii. 2.5A_{hm} times the weight of the wall, or

iv. 3 kN per linear meter, acting normal to the wall at the level of the floor or roof.

Walls shall be anchored at the roof and all floor levels at a spacing of equal to or less than 1.8 m on center. At the roof and all floor levels, anchors shall be provided within 0.6 m horizontally from the inside corners of the wall. The connection between the walls and the diaphragm shall not induce cross-grain bending or tension in the wood ledgers.

(d) Buildings with Open Fronts

i. Single-storey buildings with an open front on one side shall have cross walls parallel to the open front. The effective diaphragm span, L_i, shall be calculated as follows:

ii. $L_i = 2L\left(\frac{W_w}{W_d}+1\right)$ ….(A.14)

iii. The diaphragm demand-capacity ratio shall be calculated as follows:

$$DCR = \frac{2.5A_{hm}\left(W_d + W_w\right)}{\left(v_u D_d + V_c\right)} \quad ….(A.15)$$

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At any level the design shear load for a given shear wall will be the sum of the storey shear forces calculated by Equations (1.8) through (1.12) for stories above that level. The least value of calculated storey shear is to be used for analysis of the existing shear wall.

CA1.4 –

Buildings with open fronts -

In an "open-front" building (Figure AC7) the horizontal displacement can be controlled by using "solid" shear walls at both ends, with the effective (equivalent) diaphragm span L_i.

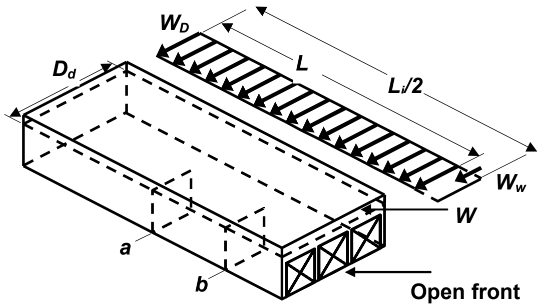


Figure AC7: Open front buildings

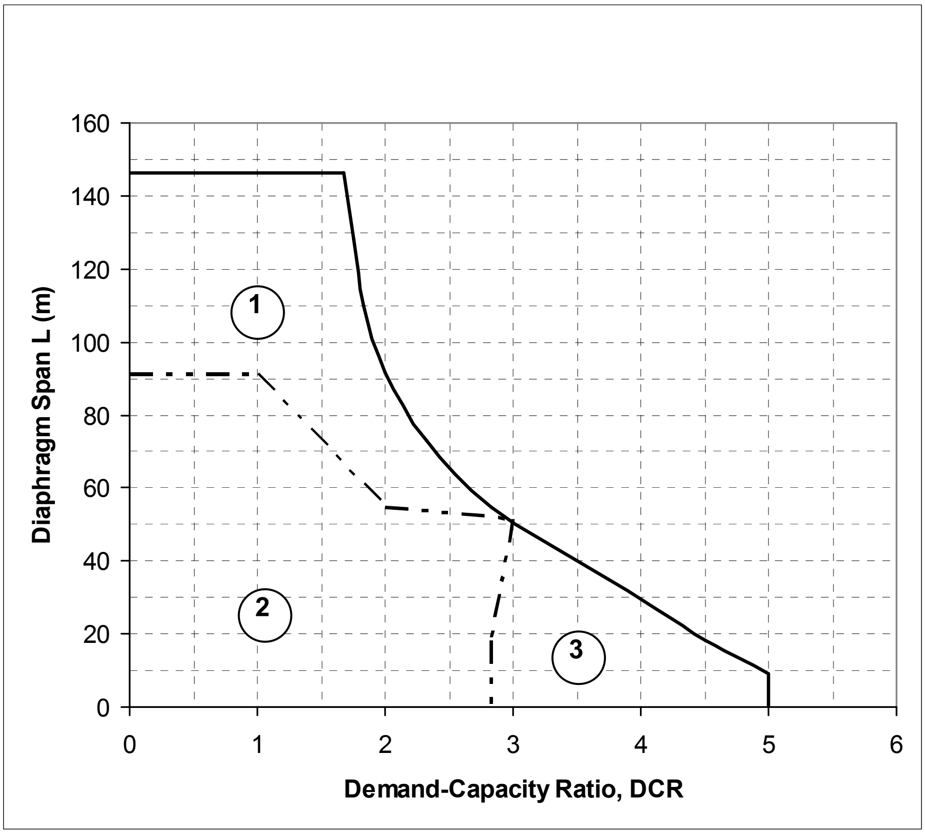


Figure 5: Acceptability criterion for flexible diaphragm

Appendix C1

Techniques for In situ Testing for masonary

Masonry Tests:

1. **Shove test (in-plane shear test) -** Its a partially destructive testing designed to measure the sliding shear strength a masonry joint. It involves displacing a single masonry unit horizontally using hydraulic jack. This necessitates removal of a single masonry unit to provide access for the hydraulic jack. A head joint opposite to the test brick is excavated carefully to isolate it from remaining masonry. This test gives a measure of shear resistance of a masonry wall. This test is time consuming but relatively easy.
2. **Flatjack test –** To measure *insitu* compressive stress in masonry. Pair of Demec studs are placed above and below a bed joint. A special flatjack is inserted into a horizontal slot cut into this bed joint. Pressure is applied on through the flatjack until the distance between the Demec studs return to their original values due to relief in stress caused by cutting of slot. Before carrying out the test each flatjack has to be calibrated. Flatjacks can also be used to evaluate the deformation properties of the masonry. Two flatjacks are put one over the other with a number of courses between them. Flatjacks are pressurized equally and deformation in the masonry may be related to the imposed loads through flatjack.
3. **Split-cylinder test –** Compression failure of the masonry is caused due to lateral tensile stress generated due to mortar going through greater transverse deformation than masonry units. This behavior is utilized in split-cylinder test. Compressive strength of the units is determined by direst test on vertical cores. Cylinder splitting strength of the units is determined by testing cores taken horizontally form the units. Reduction in the transverse tensile strength due to mortar is found by performing cylinder-splitting tests on horizontal cores incorporating a bed joint in the horizontal diametric plane. *Insitu* compressive strength is then calculated by multiplying the ratio of splitting strength of jointed cylinder and splitting strength of solid cylinder with compressive strength of units.

IITK-GSDMA GUIDELINES
for SEISMIC EVALUATION
and STRENGTHENING
of EXISTING BUILDINGS

PART 2: EXPLANATORY EXAMPLES

EXAMPLE 1: SEISMIC EVALUATION OF RC MOMENT RESISTING FRAME BUILDING

Problem Statement

An existing five-storey reinforced concrete moment frame building is located in the seismic zone V and on firm soil. The seismic frames of the building do not have the details required for ductile behavior in zones of high seismicity. The building measures approximately 68 m each way in plan at all floor levels. Storey heights are about 4 m at all elevations. The roof and floors are reinforced concrete waffle slabs consisting of a 75 mm slab over reinforced concrete two-way joists of size 0.15 m x 0.35 m. The floor slabs are essentially rigid in plane and distribute the lateral forces so that most of the lateral forces will be resisted by the exterior frames since the perimeter frames are much stiffer laterally than the interior frames. The beams in the building are of 0.8 m x 1.5 m and columns in the building are 0.8 m square. The material properties are f_{ck} = 20 MPa and f_y = 415 MPa. Evaluate the building for seismic resistance and provide strengthening options for the deficiencies identified.

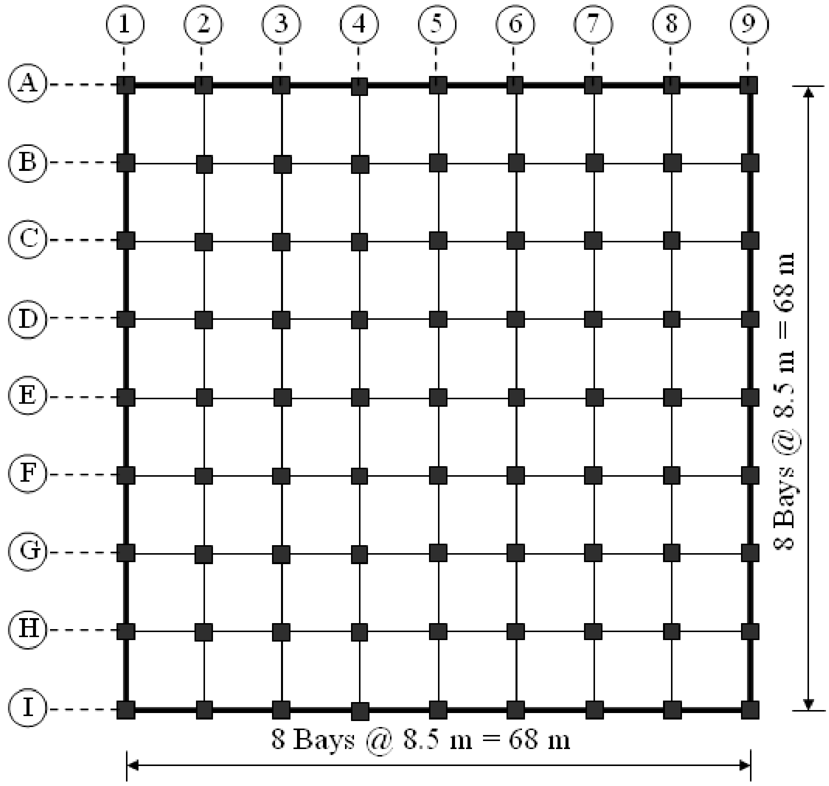


Figure 1.1: Plan of building

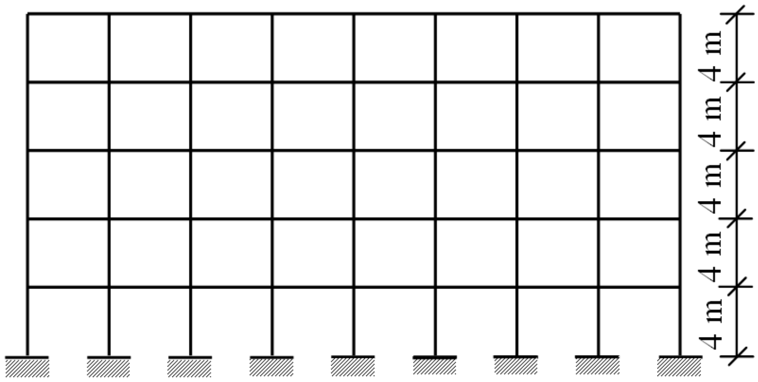


Figure 1.2: Elevation of building

Solution:

1.1. Preliminary Evaluation

1.1.1. Configuration-Related Checks

These checks for the study building are summarized in Table 1.1 as below:

Table 1.1: Configuration related checks

S. No.	Check	Remark
1.	*Load Path*	One complete load path exists which transfers the inertial forces from the mass to the foundation.
2.	*Geometry*	Horizontal dimension is equal at all the stories.
3.	*Weak Storey*	There are no abrupt changes in the column sizes from one storey to another and no significant geometrical irregularities. Thus, weak or soft storey does not exist.
4.	*Soft Storey*	
5.	*Vertical Discontinuities*	Vertical elements in the lateral force resisting system are continuous to the foundation.
6.	*Mass*	Effective mass at all the floors is equal except the roof. The effective mass at the roof varies by 20% (<100%).
7.	*Torsion*	The building being symmetrical, centre of mass and centre of rigidity coincide.
8.	*Adjacent Buildings*	Not applicable.
9.	*Short Columns*	Short columns do not exist.

1.1.2. Strength-Related Checks

1.1.2.1. Design Seismic Base Shear

As per the draft code on seismic evaluation and strengthening, the design seismic base shear is modified considering the existing conditions of the building. Hence, the modified base shear is given as,

$V_{bm} = A_{hm}\, W = (U A_h)\, W$

where,
U = factor for reduced useable life = 0.67

A_h = design horizontal seismic coefficient is given as,

$$A_h = \frac{ZIS_a}{2Rg}$$ (Clause.6.4.2 of IS 1893: 2002)

Time period in both directions,

$T = 0.075h^{0.75}$ (Clause.7.6.1 of IS 1893: 2002)

$\therefore T = 0.075 \times 20^{0.75} = 0.709\,\text{sec}$.

Corresponding to *T=0.709 sec.*, stiff soil and 5% damping,

$\frac{S_a}{g} = 1.41$ (Figure 2 of IS 1893: 2002)

The building is situated in an area of high seismicity.

$\therefore Z = 0.36$ (Table 2 of IS 1893: 2002)

Importance factor,

$I = 1.0$ (Table 6 of IS 1893: 2002)

Response reduction factor,

$R = 3.0$ (Table 7 of IS 1893: 2002)

Thus,

$A_h = 0.0846$

and

$A_{hm} = 0.67\, A_h = 0.057$

Calculation of seismic weight:

Average roof dead load = 7.0 kN/m^2

Average floor dead load = 8.0 kN/m^2

Live load intensity = 3.0 kN/m^2

As per clause 7.3.1 of IS 1893: 2002, imposed load to be considered in seismic weight,

25% of imposed load = 0.75 kN/m^2

Total live load on each floor except roof

$$= 0.75 \times 68^2 = 3468 \text{ kN}$$

Dead load on roof $= 7 \times 68^2 = 32.4 \times 10^3$ kN

Dead load on other floors

$$= 8 \times 68^2 = 36.9 \times 10^3 \text{ kN}$$

Seismic weight on roof $= 32.4 \times 10^3$ kN

Seismic weight on other floors

$$= 36.9 \times 10^3 + 3468 = 40.4 \times 10^3 \text{ kN}$$

Total seismic weight of the building

$$= \left(32.4 + \left(4 \times 40.4\right)\right) \times 10^3$$

$$= 194.0 \times 10^3 \text{ kN}$$

Hence, modified seismic base shear

$$= A_{hm} W$$

$$= 0.057 \times 194.0 \times 10^3 \text{ kN}$$

$$= 11.2 \times 10^3 \text{ kN}$$

1.1.2.2. Shear Stress In RC Frame Columns

As per clause 6.5.1 of the draft code, an estimation of the average shearing stress in columns is given as,

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

For ground storey columns,

n_c = total no. of columns resisting lateral forces in the direction of loading

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

A_c = Summation of the cross-section area of all columns in the storey under consideration

V_j = maximum storey shear at storey level *'j'*

Table 1.2: Shear stress in columns

Storey	n_c	n_f	A_c, m^2	V_jx1.5, kN	$\tau_{col,}$ MPa	DCR
5	18	2	11.52	6700	0.66	1.64
4	18	2	11.52	12060	1.18	2.94
3	18	2	11.52	15080	1.48	3.68
2	18	2	11.52	16420	1.60	4.00
1	18	2	11.52	16760	1.64	4.10

$$\left(\tau_{col}\right)_{all.} < \min.of\ 0.4\,\text{MPa}\ and\ 0.1\sqrt{f_{ck}}$$

$$< 0.4\,\text{MPa}$$

But, $\tau_{col} > \left(\tau_{col}\right)_{all.}$ NG.

The check is not satisfied. Hence, a more detailed evaluation of the structure should be performed.

1.1.2.3. Axial Stress in Moment Frames

Axial force in columns of moment frames at base due to overturning forces,

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

V_B = Base shear x Load factor

$$= 11.2 \times 10^3 \times 1.5$$

$$= 16.8 \times 10^3 \text{ kN}$$

n_f = No. of frames in the direction of loading = 2

H = total height = 20 m

L = length of the building = 68 m

$$F_O = \frac{2}{3}\left(\frac{16.8 \times 10^3}{2}\right)\left(\frac{20}{68}\right)$$

1647 kN

Axial stress,

$$\sigma = \frac{1647 \times 10^3}{0.8^2 \times 10^6} = 2.57$$

$$\sigma_{all.} = 0.25 f_{ck} = 0.25 \times 20$$

$$= 5\,\text{MPa}$$

$\therefore \sigma < \sigma_{all.}$ OK.

DCR = 0.52 Hence, the check is satisfied.

1.2. Detailed Evaluation

The use of a dynamic analysis procedure is required for tall buildings, buildings with vertical irregularities caused by significant mass or geometric irregularities, or other cases where the distribution of the lateral forces departs from that assumed in the equivalent lateral force procedure.

This example is a five-storey building without any significant geometrical irregularities. There are no significant mass irregularities and all frames are continuous to the foundation. Therefore, this example may be evaluated using the equivalent lateral force procedure.

1.2.1. *Vertical Distribution of Lateral Forces by Static Method*

Table 1.3: Lateral Force Distribution

(as per Clause. 7.7.1 of IS 1893: 2002)

Storey	W_i x 10^3, kN	h_i, m	$W_ih_i^2$x 10^6	$\frac{W_ih_i^2}{\sum W_ih_i^2}$	Lateral force for EQ load in X & Y direction x 10^3, kN
5	32.4	20	12.95	0.4	4.5
4	40.4	16	10.36	0.32	3.6
3	40.4	12	5.83	0.18	2.0
2	40.4	8	2.59	0.08	0.9
1	40.4	4	0.65	0.02	0.2
∑	194.0		32.4		11.2

1.2.2. *Eccentricity Calculation for Additional Torsional Moment*

As the building is symmetrical in geometry, centre of mass and centre of stiffness of the building will coincide. Hence, there will be no static eccentricity.

Accidental eccentricity

As per clause 7.9.2 of IS 1893: 2002,

For loading in x & y direction

$e_x = e_y = 0.05 \times 68 = 3.4\,\text{m}$

Table 1.4:Additional torsional moment

Storey	Torsional moment in X & Y direction, kNm
Roof	15.2 x 10^3
4th	12.1 x 10^3
3rd	6.8 x 10^3
2nd	3.0 x 10^3
1st	0.8 x 10^3

1.2.3. *Shear Distribution to Frames*

Shear distribution to frames on ground floor.

Shear in ground floor,

V =11.2 x 10^3 kN

Accidental eccentricity,

e = 0.05 x 68 = 3.4 m

Accidental torsional moment,

M_T = 11.2 x 10^3 x 3.4
= 38.0 x 10^3 kNm

Table 1.5: Distribution of translational and torsional shears at ground floor

Frame	Rigidity	d_x, m	Rd_x	Rd_x^2	$F_v = \frac{R}{\sum R} \times V$	$F_m = \frac{Rd}{\sum Rd^2} \times M_T$	F=F_v+F_m
1	9	34	306	10404	5.6 x 10^3	279	5879
9	9	34	306	10404	5.6 x 10^3	279	5879
∑	18						
		d_y	Rd_y	Rd_y^2			
A	9	34	306	10404	0	279	279
J	9	34	306	10404	0	279	279
∑				41616			

Shear to frames 1 & 9 on ground floor = 5879 kN.

Increase in shear due to eccentricity =

$$\frac{5879}{5585} = 1.05$$

Hence, the vertical distribution of lateral forces to the frame on column line 1 is as below,

$$F_5 = \left(\frac{4468}{2}\right) \times 1.05 = 2345.7\,\text{kN}$$

$$F_4 = \left(\frac{3574}{2}\right) \times 1.05 = 1876.4\,\text{kN}$$

$$F_3 = \left(\frac{2010}{2}\right) \times 1.05 = 1055.3\,\text{kN}$$

$$F_2 = \left(\frac{894}{2}\right) \times 1.05 = 469.4\,\text{kN}$$

$$F_1 = 5864 - (2345.7 + 1876.4 + 1055.3 + 469.4)$$
$$= 117.2\,\text{kN}$$

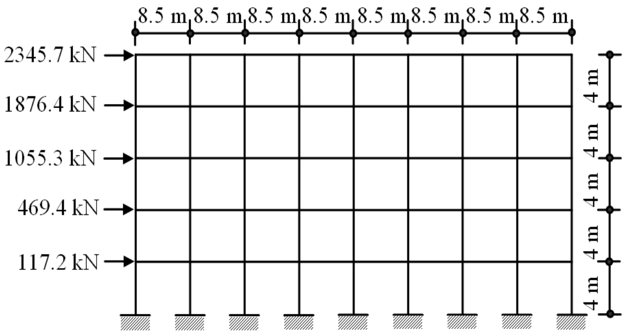


Figure 1.3: Vertical distribution of lateral forces to the frame

1.2.4. Beam Forces

Load coming on each external beam depends on the tributary area given as,

$$A = \frac{1}{2} \times 8.5 \times 4.25$$
$$= 18.06\,\text{m}^2$$

Uniformly distributed load on beams due to dead load at roof level

$$= \frac{7 \times 18.06}{8.5} = 14.9\,\text{kN/m}$$

Uniformly distributed load on beams due to dead load on typical floor level

$$= \frac{8 \times 18.06}{8.5} = 17\,\text{kN/m}$$

Uniformly distributed load on beams due to live load on typical floor level

$$= \frac{3 \times 18.06}{8.5} = 6.4\,\text{kN/m}$$

The linear static analysis of the frame shown in figure 1.3 is performed using SAP software. The various load combinations used for the above mentioned loads are:

Comb. 1 – 1.5 (DL+LL)

Comb. 2 – 1.2 (DL+LL+EL)

Comb. 3 – 1.2 (DL+LL-EL)

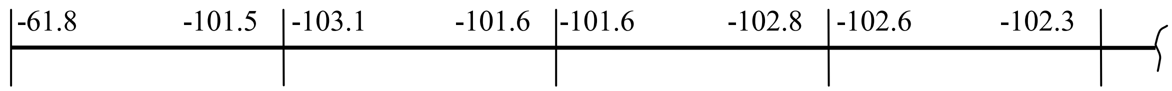
Comb. 4 – 1.5 (DL+EL)

Comb. 5 – 1.5 (DL-EL)

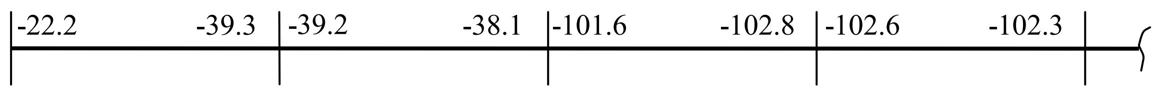
Comb. 6 – 0.9DL+1.5EL

Comb. 7 – 0.9DL-1.5EL

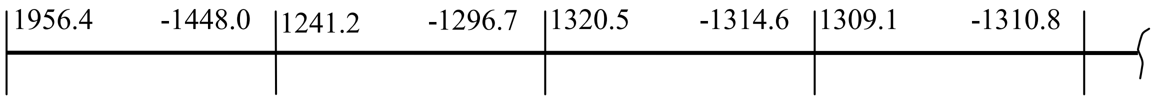
The analysis results gave the following moment values for the beam:



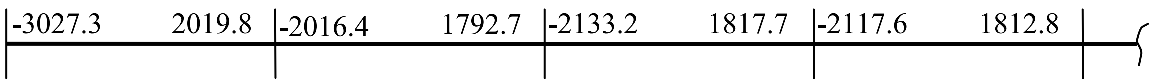
(a) Beam End Moments due to Dead Loads



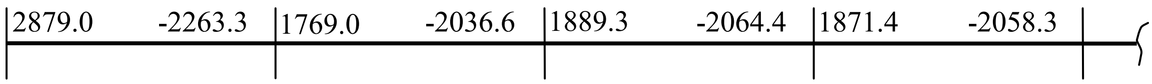
(b) Beam End Moments due to Live Loads



(c) Beam End Moments due to Earthquake Loads



(d) Beam End Moments under Load Combination 5



(e) Beam End Moments under Load Combination 6

Figure 1.4: Beam moments due to various load combinations

1.2.5. Column Forces

The column forces in the internal column on grid line 1 at ground floor level are given as,

For dead load case, $P = -700.4\,\text{kN}$

For live load case, $P = -216.3\,\text{kN}$

For earthquake load, $P = -213.2\,\text{kN}$

Moment for earthquake load, $M = 1433.3\,\text{kNm}$

Load combination 7 gives the critical forces in column,

Axial Load $P = -310.5\text{kN}$
Moment $M = -2149.9\,\text{kN}$
Shear Force $V = -1023.6\,\text{kN}$

1.3. Component Level Analysis

The moment frame at grid line 1 will be used as an example to illustrate the determination of the structural deficiencies.

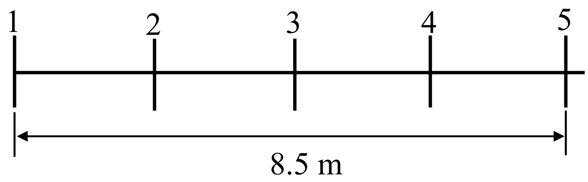
Analysis results of beam member on the first floor are taken for further calculations. The beam is divided in 5 sections of 1.7 m. each.

Moments at different sections due to load combination 5,

At section 1 = -3027.3 kNm
At section 2 = -1592.8 kNm
At section 3 = -273.4 kNm
At section 4 = 930.8 kNm
At section 5 = 2019.8 kNm

Moments at different sections due to load combination 6,

At section 1 = 2879.0 kNm
At section 2 = 1697.1 kNm
At section 3 = 446.0 kNm
At section 4 = -874.1 kNm
At section 5 = -2263.3 kNm



(a) Beam section details

b	=	800 mm
D	=	1500 mm
Clear cover	=	20 mm
Diameter of main r/f.	=	25 mm
Diameter of stirrups	=	12 mm
Effective depth d	=	1455.5 mm
d' (effective cover)	=	44.5 mm
Length of beam (L)	=	8500 mm
Size of adjoining col.	=	800 mm
Clear span (L_c)	=	7700 mm
Area of sec. (b x d)	=	1164400 mm^2

(b) Material Properties:

f_{ck}	=	20 MPa
f_y	=	415 MPa
σ_{cc}	=	8.92 MPa
τ_{bd} (deformed bars)	=	1.92 MPa
K	=	1

(c) Capacity at Section-1:

Using 25 mm dia. bars for main reinforcement

Tension Steel:

No. of bars provided = 6

$A_{st\,prov.}$ = 2945 mm^2 = 0.25%

Compression Steel:

No. of bars provided = 5

$A_{sc\,prov.}$ = *2454* mm^2 = *0.21%*

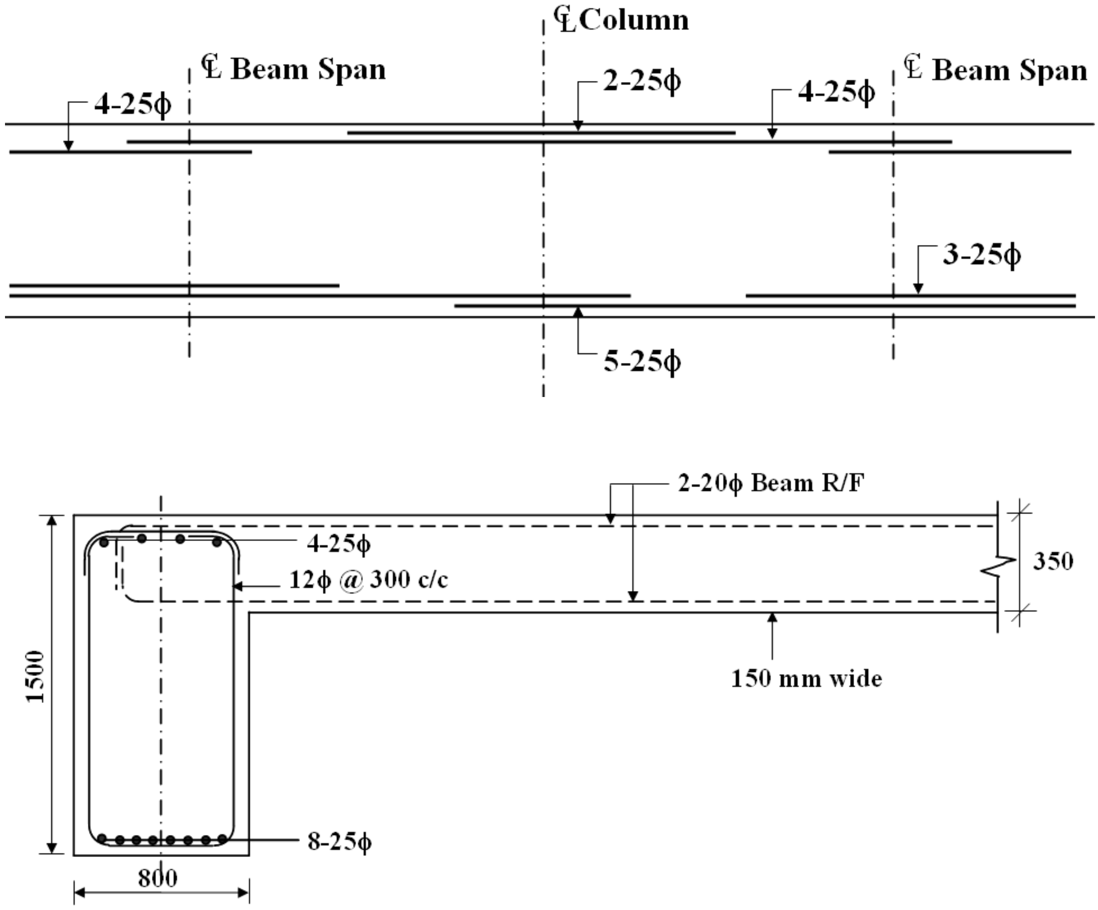


Figure 1.5: Flexural reinforcement in beam span

1.3.1. Calculation of Moment of Resistance in Hogging

Force in Compression,

$C = 0.36 f_{ck} bx + (\sigma_{sc} - \sigma_{cc}) A_{sc}$

Force in Tension,

$T = 0.87 f_y A_{st} = 1063\,\text{kN}$

Assuming $x = x_{\text{lim.}} = 0.48d = 698.64\,\text{mm}$

Assuming $\sigma_{sc} = 0.87 f_y = 361.1\,\text{MPa}$

By equating $C = T$, $x = 35\,\text{mm}$

which is less than $x_{lim.}$

Now, $\varepsilon_{sc} = 0.0035(1 - d'/x) = -0.0010$

The assumed value of σ_{sc} gives tensile strain in concrete. Hence revising the value of σ_{sc} and applying trial and error method,

Assuming $\sigma_{sc} = 271.56\,\text{MPa}$

By equating $C = T$, $x = 72.7\,\text{mm}$

$\varepsilon_{sc} = 0.0035(1 - d'/x) = 0.00136$

From Fig. 23A, IS 456: 2000

$\sigma_{sc} = 271.56\,\text{MPa}$

Hence, $\sigma_{sc} = 271.56\,\text{MPa}$

$x = 72.7\,\text{mm}$

Moment of resistance of the section

$= [0.36 f_{ck} bx(d - 0.42x) + (\sigma_{sc} - \sigma_{cc}) A_{sc} (d - d')K]$

=1506.2 kNm<3027.2 kNm

DCR = 2, NG.

1.3.2. Calculation of Moment of Resistance in Sagging

$A_{st\,prov.} = 2454\,\text{mm}^2$

$A_{sc\,prov.} = 2945\,\text{mm}^2$

Force in Compression,

$C = 0.36 f_{ck} bx + (\sigma_{sc} - \sigma_{cc}) A_{sc}$

Force in Tension,

$T = 0.87 f_y A_{st} = 886\,\text{kN}$

Assuming, $x = x_{\lim.} = 0.48d = 698.64\,\text{mm}$

Assuming,

$\sigma_{sc} = 0.87 f_y = 361.1\,\text{MPa}$

By equating $C = T$, $x = -26\,\text{mm}$

which is less than $x_{\lim.}$

Now, $\varepsilon_{sc} = 0.0035(1 - d'/x) = -0.0024$

The assumed value of σ_{sc} gives tensile strain in concrete. Hence revising the value of σ_{sc} and applying trial and error method,

Assuming $\sigma_{sc} = 190.26\,\text{MPa}$

By equating $C = T$, $x = 61.1\,\text{mm}$

$\varepsilon_{sc} = 0.0035(1 - d'/x) = 0.0010$

From Fig. 23A, IS 456: 2000

$\sigma_{sc} = 190.25\,\text{MPa}$

Hence, $\sigma_{sc} = 190.25\,\text{MPa}$

$x = 61.1\,\text{mm}$

Moment of resistance of the section

$= [0.36 f_{ck} bx(d - 0.42x) + (\sigma_{sc} - \sigma_{cc}) A_{sc}(d - d')K]$

=1256.8 kNm<2879.0 kNm

DCR = 2.3

NG.

1.3.3. Check of Shear Capacity of beam

The shear reinforcement provided in the existing beam at support section is 2 legged 10 ϕ @ 300 c/c.

$A_s = 2945\,\text{mm}^2$

$$\frac{100A_s}{bd} = \frac{100 \times 2945}{800 \times (1500 - 32.5)} = 0.25$$

From table 19 of IS 456: 2000, for M20 grade of concrete and $\frac{100A_s}{bd} = 0.25$,

$\tau_c = 0.36$ MPa

Stirrups are 2 legged 10ϕ @ 300 c/c

From cl. 40.4 of IS 456: 2000

$$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$$

$$V_u = V_{us} + \tau_c bd$$

$$= \frac{0.87 \times 415 \times 78 \times 2 \times 1467.5}{300} + 0.36 \times 800 \times 1467.5$$

= 699 kN

$\therefore V_u \times K = 699\,\text{kN}$

<u>Shear demand in beam</u>

V as per analysis = 716.2 kN

Moment capacity of beam

$M_R^{\,H} = 1506.2$ kNm

$M_R^{\,S} = 1256.8$ kNm

L_c=7.7 m

$V_a^{D+L} = V_b^{D+L} = 119.2$ kN

V_u from capacity design (IS:13920)

$$V_u = 119.2 + 1.4\frac{M_R^{\,H} + M_R^{\,S}}{L_c}$$

= (119.2+502) kN

= 621.2 kN

So, final Shear demand 716.2 kN

699.0 < 716.2

DCR = 1.02

NG.

Thus, the shear capacity is less than the shear demand on the beam, indicating the deficiency of beam in shear under seismic loads. This deficiency will further increase if the flexure capacity of the beam is increased.

1.3.4. Column Flexural Capacity

Calculating the column bending capacity for ground storey column:

The column demand given by load combination 7 is,

$P_u = 310.5\,\text{kN}$
$M_u = 2149.9\,\text{kNm}$

Properties of the column are,

$f_{ck} = 20\,\text{MPa}$

$f_y = 415\,\text{MPa}$

$K = 1$

Clear cover = 40 mm

$d' = 40 + 12 + 15$
$\quad = 67\,\text{mm}$

$\frac{d'}{D} = \frac{67}{800} = 0.083 \approx 0.1$

$A_s = 16 \times \frac{\pi}{4} \times 30^2 = 11310\,\text{mm}^2$

Percentage of reinforcement,

$p = \frac{11310}{800 \times 800} \times 100 = 1.77$

$\frac{p}{f_{ck}} = \frac{1.77}{20} = 0.089$

$\frac{P_u}{f_{ck} bD} = 0.024$

Referring to Chart 44 of SP: 16,

$\frac{M_u'}{f_{ck} bD^2} = 0.13$

$\therefore M_u' = 1331\,\text{kNm}$

$M_u' \times K = 1331\,\text{kNm} < 2149.9\ \text{kNm}$

DCR = 1.6

NG.

Since the bending moment demand is larger than the capacity, the columns were found to be deficient in bending under seismic loads.

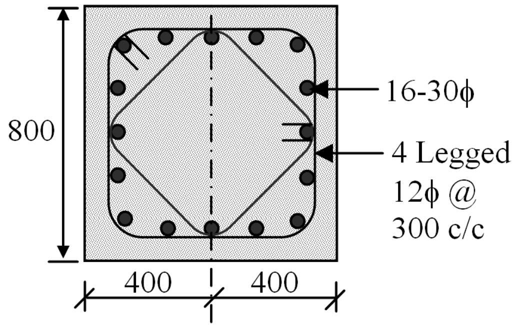


Figure 1.6: Column details

1.3.5. Column Shear Capacity

Considering that the steel in one face will be in tension,

$A_s = 5 \times \frac{\pi}{4} \times 30^2$
$\quad = 3534\,\text{mm}^2$

$\therefore \frac{100 A_s}{bd} = \frac{100 \times 3534}{800(800 - 67)} = 0.6$

From table 19 of IS 456: 2000, for M20 grade of concrete and $\frac{100 A_s}{bd} = 0.6$,

$\tau_c = 0.51\,\text{MPa}$

Stirrups are 4 legged 12ϕ @ 300 c/c

From cl. 40.4 of IS 456: 2000

$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$

$V_u = V_{us} + \tau_c bd$
$\quad = \frac{0.87 \times 415 \times 452 \times 733}{300} + 0.51 \times 800 \times 733$

$V_u = 699\,\text{kN}$

$\therefore V_u \times K = 699\,\text{kN}$

Shear demand in column

V as per analysis = 1023.6 kN

Moment capacity of beam

$M_{u,\lim}^{bR} = 1506.2\ \text{kNm}$

$M_{u,\lim}^{bL} = 1256.8\ \text{kNm}$

$h_{st} = 4\,\text{m}$

V from capacity design (IS 13920)

$= V_u = 1.4\left(M_u^{bL} + M_u^{bR}\right)/h_{st}$

Hence, $V_u = 967.05$ kN

So, final Shear demand 1023.6 kN

699.0 < 1023.6

DCR = 1.46

NG.

Thus, the shear capacity is less than the shear demand on the column, indicating the deficiency of column in shear under seismic loads.

1.3.6. Strong Column / Weak Beam Considerations

The flexural strengths of the columns shall satisfy the following condition:

$$\sum M_C \geq 1.1 \sum M_B$$

1st storey interior column capacity = 1331 kNm.

$$\therefore \sum M_C = 1331 + 1331 = 2662 \text{ kNm}$$

1st storey beam,

-ve moment capacity = 1506.2 kNm

+ve moment capacity = 1256.8 kNm

$$\therefore \sum M_B = 1506.2 + 1256.8 = 2763.0 \text{ kNm}$$

$$1.1 \sum M_B = 3039.3 \text{ kNm}$$

$$\therefore \sum M_C < 1.1 \sum M_B$$

NG.

Hence, the strong column/weak beam requirement is not satisfied.

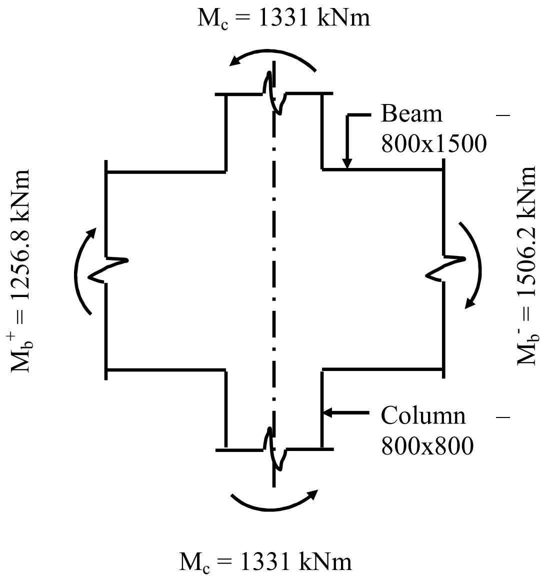


Figure 1.7: Design Flexural Strength at the Column/Beam Joint

1.3.7. Storey Drift of the frame

The deflections and storey drifts of the frames due to seismic forces are summarised in table 1.6.

Table 1.6: Storey Drift

Storey	Displacement, m	Storey drift, m	Allowable storey drift m
1st	0.00544	0.00544	0.016
2nd	0.01164	0.00620	0.016
3rd	0.01738	0.00574	0.016
4th	0.02198	0.00460	0.016
5th	0.02460	0.00262	0.016

The design storey drift is calculated as the differences of deflections at the top and bottom of the storey under consideration.

The allowable storey drift shall be limited to 0.004 h_x (where h_x is the storey height of storey x). So, for the example building as all the stories are of same height the allowable storey drift is 0.004 x 4 m = 0.016 m.

Hence, it is found that all the stories of the frame are satisfying the storey drift limitation requirement.

1.4. Evaluation Summary

i. The results of the preliminary evaluation (strength-related checks) indicate deficiency in the shear stress carrying capacity of the columns. The Demand-Capacity ratio (DCR) lies in the range of 2. Hence, indicating the need of a detailed analysis.

ii. Detailed evaluation of the beam and column elements of one of the perimeter frame can be summarized as below:

Table 1.7 Summary of Detailed Evaluation

S. No.	Check	DCR	Remarks
1.	Moment of Resistance of beam in hogging	2	Check not satisfied.
2.	Moment of Resistance of beam in sagging	2.3	Check not satisfied
3.	Column Flexural Capacity	1.6	Check not satisfied.
4.	Column Shear Capacity	1.46	Check not satisfied
5.	Strong Column/Weak Beam	--	Check not satisfied
6.	Storey Drift	--	Check satisfied

- Thus, the above evaluation suggests that the frame needs to be strengthened and retrofitted.

1.5. Strengthening Options

Feasible strengthening schemes for the deficiencies identified in the moment frames include:

1) Increasing the ductility and capacity of the moment frames by encasing the existing beams and columns in a reinforced concrete jacket.
2) Reducing the seismic stresses in the existing frames by providing additional lateral load resisting elements within the building. This could include new shear walls infilled in the existing interior or perimeter concrete frames.
3) Reducing the seismic stresses in the existing frames by providing one or more structural additions that are designed to provide supplemental lateral load resistance to the existing building.
4) Reducing the seismic stresses in the existing frames by constructing parallel concrete moment frames in the exterior of the building, connected for seismic shear transfer at each floor level.
5) Removing all the existing perimeter ordinary moment frames and replacing them with new reinforced concrete special moment frames that are designed and detailed for ductile behavior.
6) Modification and/or limited replacement of the existing perimeter concrete to improve their strength and ductility

EXAMPLE 2: **SEISMIC RETROFIT WITH LIMITED REPLACEMENT/MODIFICATION OF FRAME MEMBERS**

Problem Statement:

Seismic evaluation of the RC moment resisting frame building of Example 1 indicated that laterally more stiff ordinary moment resisting frames at perimeter do not have the strength and details required for ductile behavior. In this example a detailed procedure is given for the retrofitting of the building, by modification and/or limited replacement of the perimeter frames for the required strength and ductility.

2.1. Seismic Strengthening of the Building

Out of different feasible schemes of strengthening of the building frame, modification and/or limited replacement of existing perimeter ordinary moment resisting frames technique is investigated in this example. The modified structure has to meet the load demand and ductility demand as per appropriate provisions of the codes.

The reason behind this scheme instead of new SMRFs is that same amount of ductility and strength can be given without doing much salvage of existing frame. Thus the scheme is advantageous over other in the sense of its less necessity of salvage, shoring and less construction cost. But feasibility of the scheme is highly dependant on the design and detailing of the existing frame.

2.2. Retrofit of the existing frame

The exterior beam in the first floor on grid line 1 is chosen for proposed retrofit.

For the retrofitting here 2.5 m portion of the existing beams on each side of the existing columns are removed and replaced with new reinforced concrete of the same dimensions (800 mm x 1500 mm).

Thus, $\frac{b}{D} = \frac{800}{1500} = 0.533 > 0.3$ Hence, ok.

Estimated new column size

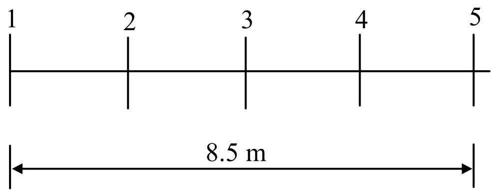
= 1100 mm x 1100 mm

Width of beam = 800 mm > 200 mm.

Hence, ok.

Force values are used from the analysis shown in Example 1.

2.3. Retrofit of the beam at support



The beam is divided in 5 sections of 1.7 m. each.

Moments at different section for load combination 5,

At section 1 = 3027.3 kNm
At section 2 = -1592.8 kNm
At section 3 = -273.4 kNm
At section 4 = 930.8 kNm
At section 5 = 2019.8 kNm

Moments at different sections for load combination 6,

At section 1 = 2879.0 kNm
At section 2 = 1697.1 kNm
At section 3 = 446.0 kNm
At section 4 = -874.1 kNm
At section 5 = -2263.3 kNm

(a) Beam section details

b = 800 mm
D = 1500 mm
Clear cover = 20 mm
Dia. of main reinf. = 25 mm
Dia. of stirrups = 12 mm
Effective depth d = 1455.5 mm
d' (effective cover) = 44.5 mm
Length of beam (L) = 8500 mm
Size of adjoining col. = 1100 mm
Clear span (L_c) = 7700 mm
Area of section (b x d)= 1164400 mm^2

(b) Material Properties:

f_{ck} = 20 MPa
f_y = 415 MPa
σ_{cc} = 8.92 MPa
τ_{bd} for deformed bars = 1.92 MPa

(c) Capacity check at section – 1

Capacity of the existing section with 6 nos. 25 mm dia bar at top and 5 nos. 25 mm dia at bottom:

Maximum negative moment capacity = 1506.2 kNm < Maximum negative moment demand

Maximum positive moment capacity = 1256.8 kNm < Maximum positive moment demand

Therefore, use 25 mm dia bars for main reinforcement.

For the retrofitting, 2.5 m portion of the existing beams on each side of the existing columns are removed and replaced with new reinforced concrete of the same dimension. In the modified sections, 7 nos. 25 mm dia. bars are added at both top and bottom and the section is checked for the required capacity.

Tension steel
No. of bars provided = 13
$A_{st\,prov.}$ = 6381 mm^2 = 0.548%
Compression steel
No. of bars provided = 12
$A_{sc\,prov.}$ = 5890 mm^2 = 0.506%
Max. tension reinf., which can be provided in a beam section

$= 0.04bD = 48000\,\text{mm}^2$

Hence, OK
Min. tension reinf. as per IS 13920

$= 0.24\left(f_{ck}\right)^{0.5}/f_y = 0.259\%$

Max. tension reinf as per IS13920 = 2.5%
Hence, ok.

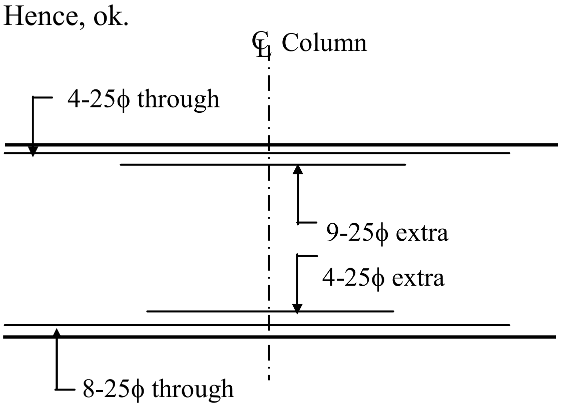


Figure 2.1: Reinforcement at beam support

2.3.1. Calculation of Moment of Resistance in Hogging

Force in compression,

$C = 0.36 f_{ck} bx + \left(\sigma_{sc} - \sigma_{cc}\right) A_{sc}$

Force in tension,

$T = 0.87 f_y A_{st} = 2304\,\text{kN}$

Assuming $x = x_{\text{lim.}} = 0.48d = 698.64\,\text{mm}$

Assuming $\sigma_{sc} = 0.87 f_y = 361.1\,\text{MPa}$

By equating $C = T,\quad x = 40\,\text{mm}$

which is less than $x_{lim.}$

Now, $\varepsilon_{sc} = 0.0035\left(1 - d'/x\right) = -0.0004$

The assumed value of σ_{sc} gives tensile strain in concrete. Hence revising the value of σ_{sc} and applying trial and error method,

Assuming $\sigma_{sc} = 313.55\,\text{MPa}$

By equating $C = T,\quad x = 88.5\,\text{mm}$

$\varepsilon_{sc} = 0.0035\left(1 - d'/x\right) = 0.00174$

From Fig. 23A, IS 456: 2000

$\sigma_{sc} = 313.54\,\text{MPa}$

Hence, $\sigma_{sc} = 313.54\,\text{MPa}$

$x = 88.5\,\text{mm}$

Moment of resistance of the section

$= \left[0.36 f_{ck} bx\left(d - 0.42x\right) + \left(\sigma_{sc} - \sigma_{cc}\right) A_{sc}\left(d - d'\right)\right]$

$= 3254.4\,\text{kNm} > 3027.3\,\text{kNm}$

DCR = 0.93

Hence ok.

2.3.2. Calculation of Moment of Resistance in Sagging

$A_{st\,prov.} = 5890\,\text{mm}^2$

$A_{sc\,prov.} = 6381\,\text{mm}^2$

Force in compression,

$C = 0.36 f_{ck} bx + \left(\sigma_{sc} - \sigma_{cc}\right) A_{sc}$

Force in tension,

$T = 0.87 f_y A_{st} = 2127\,\text{kN}$

Assuming $x = x_{\text{lim.}} = 0.48d = 698.64\,\text{mm}$

Assuming $\sigma_{sc} = 0.87 f_y = 361.1\,\text{MPa}$

By equating $C = T, \quad x = -21\,\text{mm}$

which is less than $x_{lim.}$

Now, $\varepsilon_{sc} = 0.0035(1 - d'/x) = -0.0040$

The assumed value of σ_{sc} gives tensile strain in concrete. Hence revising the value of σ_{sc} and applying trial and error method,

Assuming $\sigma_{sc} = 275.89\,\text{MPa}$

By equating $C = T, \quad x = 73.4\,\text{mm}$

$\varepsilon_{sc} = 0.0035(1 - d'/x) = 0.0014$

From Fig. 23A, IS 456: 2000

$\sigma_{sc} = 275.88\,\text{MPa}$

Hence, $\sigma_{sc} = 275.88\,\text{MPa}$

$x = 73.4\,\text{mm}$

Moment of resistance of the section

$= \left[0.36 f_{ck} bx(d - 0.42x) + (\sigma_{sc} - \sigma_{cc}) A_{sc} (d - d')\right]$

$= 3006.3\,\text{kNm} > 2879.0\,\text{kNm}$

DCR = 0.96

Hence ok.

2.3.3. Design of Shear Reinforcement

$\text{Factored shear force} = 716.2\,\text{kN}$

$V_a^{D+L} = V_b^{D+L} = 119.2\,\text{kN}..................(i)$

$1.4\left(M_R^{\,S} + M_R^{\,H}\right)/L_c = 1184.5\,\text{kN}....(ii)$

$V_u = (i) + (ii) = 1303.7\,\text{kN}$

Final $V_u = 1303.7\,\text{kN}$

Nominal shear stress

$= V_u/bd = 1.12\,MPa$

% of tensile steel, $p = 100 A_{st}/bd = 0.55$

From Table 19, IS 456: 2000,

$\tau_c = 0.48\,\text{MPa}$

Permissible shear strength,

$\tau_c bd = 553.5\,\text{kN}$

Hence, shear reinforcement is required.

Unbalanced shear force, $V_s = 750.2\,\text{kN}$

Area of shear reinforcement with 2-legged 12ϕ bars, $A_{sv} = 226.19\,\text{mm}^2$

Spacing of stirrups (as per IS 456)

$S_v = 158.45\,\text{mm}$

Provided spacing of stirrups as per IS: 13920 $= 150\,\text{mm}$

2.4. Retrofit of the Beam at Mid Span

Moments at different sections for load combination 5,

At section 1	=	-3027.3 kNm
At section 2	=	-1592.8 kNm
At section 3	=	-273.4 kNm
At section 4	=	930.8 kNm
At section 5	=	2019.8 kNm

Moments at different sections for load combination 4,

At section 1	=	2841.9 kNm
At section 2	=	1723.1 kNm
At section 3	=	489.2 kNm
At section 4	=	-859.9 kNm
At section 5	=	-2324.2 kNm

(a) Beam Details

b	=	800 mm
D	=	1500 mm
Clear cover	=	20 mm
Dia. of main reinf.	=	25 mm
Dia. of stirrups	=	12 mm
Effective depth d	=	1455.5 mm
d' (effective cover)	=	44.5 mm
Length of beam (L)	=	8500 mm
Size of adjoining col.	=	800 mm
Clear span (L_c)	=	7700 mm
Area of sec. (b x d)	=	1164400 mm^2

(b) Section Properties

f_{ck}	=	20.0 MPa
f_y	=	415 MPa
σ_{cc}	=	8.92 MPa
τ_{bd} for deformed bars	=	1.92 MPa

(c) Capacity check at section – 3

Firstly, the capacity of the existing section is checked.

Tension steel

No. of bars provided = 8

$A_{st\,prov.} = 3926\ \text{mm}^2 = 0.34\%$

Compression steel

No. of bars provided = 4

$A_{sc\,prov.} = 1963\ \text{mm}^2 = 0.17\%$

Maximum tensile reinforcement, which can be provided in a beam section

$= 0.04bD = 48000\,\text{mm}^2$. Hence, ok.

Minimum tensile reinforcement as per IS 13920 $= 0.24\left(f_{ck}\right)^{0.5}/f_y = 0.259\%$

Maximum tensile reinforcement as per IS 13920 = 2.5%. Hence, ok.

2.4.1. Calculation of Moment of Resistance in Hogging

Force in compression,

$C = 0.36 f_{ck} bx + \left(\sigma_{sc} - \sigma_{cc}\right) A_{sc}$

Force in tension,

$T = 0.87 f_y A_{st} = 1417\,\text{kN}$

Assuming $x = x_{\text{lim.}} = 0.48d = 698.64\,\text{mm}$

Assuming $\sigma_{sc} = 0.87 f_y = 361.1\,\text{MPa}$

By equating $C = T, \quad x = 126\,\text{mm}$

which is less than $x_{lim.}$

Now, $\varepsilon_{sc} = 0.0035\left(1 - d'/x\right) = 0.0023$

From Fig. 23A, IS 456: 2000 $\sigma_{sc} = 337.5\,\text{MPa}$

But, this value does not match with the assumed value of σ_{sc}.

Assuming $\sigma_{sc} = 339.92\,\text{MPa}$

By equating $C = T, \quad x = 133.3\,\text{mm}$

$\varepsilon_{sc} = 0.0035\left(1 - d'/x\right) = 0.0023$

From Fig. 23A, IS 456: 2000

$\sigma_{sc} = 339.92\,\text{MPa}$

Hence, $\sigma_{sc} = 339.92\,\text{MPa}$

$x = 133.3\,mm$

Moment of resistance of the section

$= \left[0.36 f_{ck} bx\left(d - 0.42x\right) + \left(\sigma_{sc} - \sigma_{cc}\right) A_{sc}\left(d - d'\right)\right]$

$= 1991.3\,kNm > 489.2\,kNm$

DCR = 0.25

Hence, ok.

2.4.2. Calculation of Moment of Resistance In Sagging

$A_{st\,prov.} = 1963\,\text{mm}^2$

$A_{sc\,prov.} = 3926\,\text{mm}^2$

Force in compression,

$C = 0.36 f_{ck} bx + \left(\sigma_{sc} - \sigma_{cc}\right) A_{sc}$

Force in tension,

$T = 0.87 f_y A_{st} = 709\,\text{kN}$

Assuming $x = x_{\text{lim.}} = 0.48d = 698.64\,\text{mm}$

Assuming $\sigma_{sc} = 0.87 f_y = 361.1\,\text{MPa}$

By equating $C = T, \quad x = -117\,\text{mm}$

which is less than $x_{lim.}$

Now, $\varepsilon_{sc} = 0.0035\left(1 - d'/x\right) = 0.0022$

From Fig. 23A, IS 456: 2000 $\sigma_{sc} = 333.9\,\text{MPa}$

But, this value does not match with the assumed value of σ_{sc}.

Assuming $\sigma_{sc} = 111.76\,\text{MPa}$

By equating $C = T, \quad x = 53.0\,\text{mm}$

$\varepsilon_{sc} = 0.0035\left(1 - d'/x\right) = 0.0006$

From Fig. 23A, IS 456: 2000

$\sigma_{sc} = 111.75\,\text{MPa}$

Hence, $\sigma_{sc} = 111.75\,\text{MPa}$

$x = 53.0\,\text{mm}$

Moment of resistance of the section

$= \left[0.36 f_{ck} bx\left(d - 0.42x\right) + \left(\sigma_{sc} - \sigma_{cc}\right) A_{sc}\left(d - d'\right)\right]$

$= 1006.8\,\text{kNm} > 489.2\,\text{kNm}$

DCR = 0.49

Here the existing section is sufficient to take the maximum moments at mid span. Therefore, the same section can be kept.

2.4.3. *Design of Shear Reinforcement*

Factored shear force = 607.8 kN

$V_a^{D+L} = V_b^{D+L} = 119.2\,\text{kN}.................(i)$

$1.4\left(M_R^{\,S} + M_R^{\,H}\right)/L_c = 567.2\,\text{kN}....(ii)$

$V_u = (i) + (ii) = 686.4\,\text{kN}$

Final $V_u = 686.4\,\text{kN}$

Nominal shear stress

$= V_u/bd = 0.59\,\text{MPa}$

% of tensile steel, $p = 100A_{st}/bd = 0.34$

From Table 19, IS 456: 2000,

$\tau_c = 0.39\,\text{MPa}$

Permissible shear strength,

$\tau_c bd = 4522$ kN

Hence, shear reinforcement is required.

Unbalanced shear force, $V_s = 234.2\,\text{kN}$

Area of shear reinforcement with 2-legged 12ϕ bars, $A_{sv} = 226.19\,\text{mm}^2$

Spacing of stirrups (as per IS 456)

$S_v = 507.48\,\text{mm}$

Provided spacing of stirrups as per IS: 13920 $= 300\,\text{mm}$

Calculation of development length (L_d)

$$L_d = \frac{\phi f_y}{4\tau_{bd}} = 1351\,\text{mm}$$

2.4.4. *Requirement of Side Face Reinforcement*

As per cl. 26.5.1.3. of IS 456: 2000, as the depth of web is more than 750 mm, total area of side face reinforcement required

$= 0.001 \times 1500 \times 800 = 1200\,\text{mm}^2$

So, area required per side $= \dfrac{1200}{2} = 600\,\text{mm}^2$

Providing 6 nos. 12ϕ at each side with equal spacing.

2.5. Design of New Columns

2.5.1. *Design of Ground Floor Internal Column on Grid Line 1 for Load Combination 7*

Design Forces:

$P_u = 310.5\,\text{kN}$
$M_{uxa} = 0.0\,\text{kNm}$
$M_{uxb} = 0.0\,\text{kNm}$
$M_{uya} = 1944.3\,\text{kNm}$
$M_{uyb} = -2149.9\,\text{kNm}$

(M_{ua} – Moment at top, M_{ub} – Moment at bottom)

Column Section Details

b	= 1100 mm
D	= 1100 mm
Clear cover	= 40 mm
Dia. of main reinforcement	= 30 mm
Dia. of stirrups	= 12 mm
d' (effective cover)	= 67 mm
Length of column *(L)*	= 4000mm
Effective Length (l_e)	= 8000mm
d'/D	= 0.1
Area of section *(b.D)*	=1210000 mm2
l_e/D	=7.27 < 12
l_e/b	=7.27 < 12

Hence, it is a short column.

For the design of new columns, moment due to eccentricity is also considered.

Calculation of initial moment

$M_{ux} = 0\,\text{kNm}$
$M_{uy} = 2149.9\,\text{kNm}$

Moments due to minimum eccentricity

$e_x = \dfrac{L}{500} + \dfrac{D}{30} = 44.67\,\text{mm}$ or 20 mm whichever is maximum.

$e_y = \dfrac{L}{500} + \dfrac{b}{30} = 44.67\,\text{mm}$ or 20 mm whichever is maximum.

Hence,

$e_x = 44.67\,\text{mm}$
$e_y = 44.67\,\text{mm}$
$M_{ux} = 13.9\,\text{kNm}$
$M_{uy} = 13.9\,\text{kNm}$

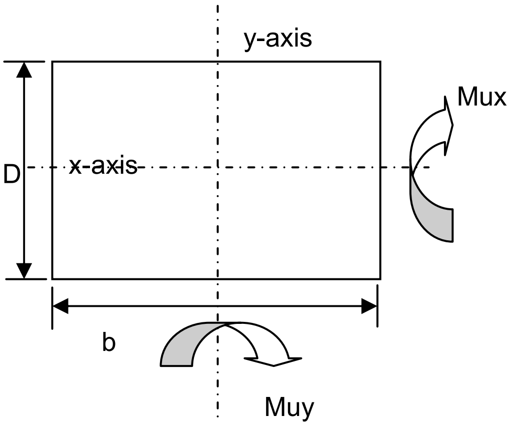
Thus, initial $M_{ux} = 13.9\,\text{kNm}$
$M_{uy} = 2149.9\,\text{kNm}$

Additional moment:

There will be no additional moment due to slenderness as the column is a short column.

Final moments

$M_{ux} = 13.9\,\text{kNm}$
$M_{uy} = 2149.9\,\text{kNm}$



Assumption: Reinforcement is distributed equally on four sides.

$$\frac{P_u}{f_{ck}bD} = 0.010$$
$$\frac{M_{ux}}{f_{ck}bD^2} = 0.000$$
$$\frac{M_{uy}}{f_{ck}bD^2} = 0.065$$

From chart no. 44 of SP:16

For $\dfrac{P_u}{f_{ck}bD}$ and maximum of $\dfrac{M_{ux}}{f_{ck}bD^2}$ & $\dfrac{M_{uy}}{f_{ck}bD^2}$,

$$\frac{p}{f_{ck}} = 0.06$$

Providing $\dfrac{p}{f_{ck}} = 0.07$

Amount of steel required = 21175 mm^2

Using 30 mm dia. bars for main reinforcement.

No. of bars required = 29.96
No. of bars provided = 32
So, no. of bars in each face = 9
Steel provided (A_{sc}) = 22619 mm^2

$\therefore A_{sc\,prov.} > A_{sc\,reqd.}$

Hence, ok.

Check for gap between bars

Gap b/w bars = 90.75 mm > 25 mm.

Hence, ok.

Check for slenderness limit

Unsupported length / 60
= 66.67 < 1100 mm. Hence, ok.

Check for column subjected to combined axial load and biaxial bending

$$P_{uz} = 0.45 f_{ck}\left(bD - A_{sc}\right) + 0.75 f_y A_{sc}$$
$$= 20398\,\text{kN} > P_u$$

Hence, ok.

$$\frac{P_u}{P_{uz}} = 0.02$$
$$\alpha_n = 1.00$$
$$p_{prov.} = 1.87$$
$$p_{prov.}/f_{ck} = 0.075$$

From Chart no. 43 to 46 of SP: 16,

for $\dfrac{P_u}{f_{ck}bD}$ *and* $\dfrac{p_{prov.}}{f_{ck}}$,

$$\frac{M_u}{f_{ck}bD^2} = 0.11$$

$M_{ux1} = 3660.3\,\text{kNm} > M_{ux}$ Hence, ok.
$M_{uy1} = 3660.3\,\text{kNm} > M_{uy}$ Hence, ok.

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = 0.59 \le 1.0 \quad \text{Hence, ok.}$$

Shear check

V as per analysis = 1023.6 kN

Moment capacity of beam

$$M_{u,\lim}{}^{bR} = 3254.4\,\text{kNm}$$
$$M_{u,\lim}{}^{bL} = 3006.3\,\text{kNm}$$
$$h_{st} = 4\,\text{m}$$

V from capacity design

$$= V_u = 1.4\left(M_u{}^{bL} + M_u{}^{bR}\right)/h_{st}$$

Hence, $V_u = 2191.23\,\text{kN}$

Area of cross section (as per clause 7.4.8, proposed IS 13920)

$$A_{sh} = 0.18Sh\frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right]....(i) \text{ or}$$

$$A_{sh} = 0.05Sh\frac{f_{ck}}{f_y}....(ii)$$

S	=	100 mm
No. of cross tie	=	3
h	=	255 mm
f_{ck}	=	25 MPa
f_y	=	415 MPa
A_g	=	1210000 mm^2
A_k	=	1040400 mm^2
A_{sh} [as per (i)]	=	45 mm^2
A_{sh} [as per (ii)]	=	77 mm^2

Provide 12 mm dia stirrups having area=113 mm^2

Check for Shear strength

(Clause 40.4, IS 456: 2000)

Assuming reinforcement of one face is in tension

$$A_s = 6362\,\text{mm}^2$$
$$\frac{100A_s}{bd} = 0.56$$
$$d = 1033\,\text{mm}$$

For M25, $\tau_c = 0.5\,\text{MPa}$

$$V_{us} = V_u - \tau_c bd = 1623.1\,\text{kN}$$
$$A_{sv} = 5 \times 113 = 565\,\text{mm}^2$$
$$V_{us\,prov.} = \frac{0.87 f_y A_{sv} d}{S_v} = 2107.3\,\text{kN}$$

Hence, ok.

2.5.2. Design of Ground Floor Internal Column on Grid Line 1 for Load Combination 6

Design Forces:

$$P_u = 1589.7\,\text{kN}$$
$$M_{uxa} = 0.0\,\text{kNm}$$
$$M_{uxb} = 0.0\,\text{kNm}$$
$$M_{uya} = 1944.3\,\text{kNm}$$
$$M_{uyb} = 2149.9\,\text{kNm}$$

Column Section Details

b	=	1100 mm
D	=	1100 mm
Clear cover	=	40 mm
Dia. of main reinforcement	=	30 mm
Dia. of stirrups	=	12 mm
d' (effective cover)	=	67 mm
Length of column (L)	=	4000mm
Effective Length (l_e)	=	8000mm
d'/D	=	0.1
Area of section ($b.D$)	=	1210000 mm^2
l_e/D	=	7.27 < 12
l_e/b	=	7.27 < 12

Hence it is a short column.

For the design of new columns, moment due to eccentricity is also considered.

Calculation of initial moment

$$M_{ux} = 0\,\text{kNm}$$
$$M_{uy} = 2149.9\,\text{kNm}$$

Moments due to minimum eccentricity

$e_x = \frac{L}{500} + \frac{D}{30} = 44.67\,\text{mm}$ or 20 mm whichever is maximum.

$e_y = \frac{L}{500} + \frac{b}{30} = 44.67\,\text{mm}$ or 20 mm whichever is maximum.

Hence,

$$e_x = 44.67\,\text{mm}$$
$$e_y = 44.67\,\text{mm}$$
$$M_{ux} = 71.0\,\text{kNm}$$
$$M_{uy} = 71.0\,\text{kNm}$$

Thus, initial $M_{ux} = 71.0\,\text{kNm}$

$$M_{uy} = 2149.9\,\text{kNm}$$

Additional moment:

There will be no additional moment due to slenderness as the column is a short column.

Final moments

$$M_{ux} = 71.0\,\text{kNm}$$
$$M_{uy} = 2149.9\,\text{kNm}$$

Assumption: Reinforcement distributed equally on four sides.

$$\frac{P_u}{f_{ck}bD} = 0.053$$
$$\frac{M_{ux}}{f_{ck}bD^2} = 0.002$$
$$\frac{M_{uy}}{f_{ck}bD^2} = 0.065$$

From chart no. 44 of SP:16

For $\dfrac{P_u}{f_{ck}bD}$ and maximum of $\dfrac{M_{ux}}{f_{ck}bD^2}$ & $\dfrac{M_{uy}}{f_{ck}bD^2}$,

$$\frac{p}{f_{ck}} = 0.06$$

Providing $\dfrac{p}{f_{ck}} = 0.07$

Amount of steel required = 21175 mm^2

Using 30 mm dia. bars for main reinforcement.

No. of bars required = 29.96
No. of bars provided = 32
So, no. of bars in each face = 9
Steel provided (A_{sc}) = 22619 mm^2

$\therefore A_{sc\ prov.} > A_{sc\ reqd.}$

Hence, ok.

Check for gap between bars

Gap b/w bars = 90.75 mm > 25 mm.
Hence, ok.

Check for slenderness limit

Unsupported length / 60
= 66.67 < 1100 mm. Hence, ok.

Check for column subjected to combined axial load and biaxial bending

$$P_{uz} = 0.45 f_{ck}\left(bD - A_{sc}\right) + 0.75 f_y A_{sc}$$
$$= 20398\,\text{kN} > P_u$$

Hence, ok.

$$\frac{P_u}{P_{uz}} = 0.08$$
$$\alpha_n = 1.00$$
$$p_{prov.} = 1.87$$
$$p_{prov.}/f_{ck} = 0.075$$

From Chart no. 43 to 46 of SP: 16,

for $\dfrac{P_u}{f_{ck}bD}$ and $\dfrac{p_{prov.}}{f_{ck}}$,

$$\frac{M_u}{f_{ck}bD^2} = 0.116$$

$M_{ux1} = 3859.9\,\text{kNm} > M_{ux}$ Hence, ok.

$M_{uy1} = 3859.9\,\text{kNm} > M_{uy}$ Hence, ok.

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = 0.58 \le 1.0 \quad \text{Hence, ok.}$$

Shear check

V as per analysis = 1023.6 kN

Moment capacity of beam

$$M_{u,\lim}{}^{bR} = 3254.4\,\text{kNm}$$
$$M_{u,\lim}{}^{bL} = 3006.3\,\text{kNm}$$
$$h_{st} = 4\,\text{m}$$

V from capacity design (IS 13920)

$$= V_u = 1.4\left(M_u{}^{bL} + M_u{}^{bR}\right)/h_{st}$$

Hence, $V_u = 2191.2\,\text{kN}$

Area of cross section (as per clause 7.4.8, proposed IS 13920)

$$A_{sh} = 0.18 Sh \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right] ....(i) \text{ or}$$

$$A_{sh} = 0.05 Sh \frac{f_{ck}}{f_y} ....(ii)$$

S = 100 mm
No. of cross tie = 3
h = 255 mm
f_{ck} = 25 MPa
f_y = 415 MPa
A_g = 1210000 mm^2
A_k = 1040400 mm^2
A_{sh} [as per (i)] = 45 mm^2
A_{sh} [as per (ii)] = 77 mm^2

Provide 12 mm dia stirrups having area=113 mm^2

Check for Shear strength

(as per cl. 40.4, IS 456: 2000)

Assuming reinf. of one face is in tension A_s.

$$A_s = 6362\,\text{mm}^2$$
$$\frac{100 A_s}{bd} = 0.56$$

$d = 1033\,\text{mm}$

For M25, $\tau_c = 0.5\,\text{MPa}$

$V_{us} = V_u - \tau_c bd = 1623.1\,\text{kN}$

$A_{sv} = 5 \times 113 = 565\,\text{mm}^2$

$$V_{us(provided)} = \frac{0.87 f_y A_{sv} d}{S_v} = 2107.3\,\text{kN}$$

Hence, ok.

Figure 2.2 shows the details of new replacement columns.

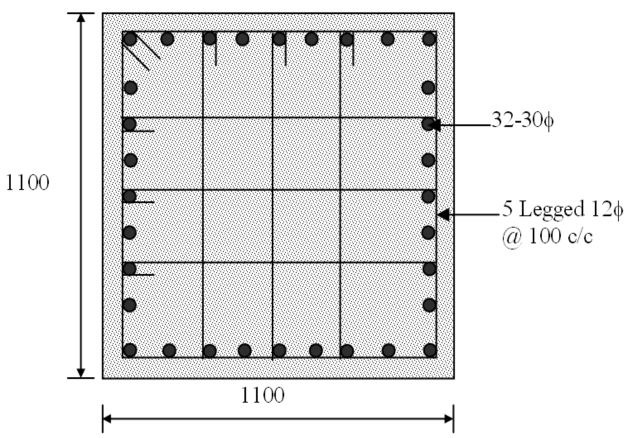


Figure 2.2: Column Details

2.5.3. Check for Strong Column /Weak Beam Action

The flexural strengths of the columns shall satisfy the following condition:

$$\sum M_C \geq 1.1 \sum M_B$$

Moment forces acting on the beam-column joint in consideration are shown in Figure 2.3.

1st storey interior column capacity = 3660.3 kNm.

$$\therefore \sum M_C = 3660.3 + 3660.3 = 7320.6\,\text{kNm}$$

1st storey beam,

-ve moment capacity = 3254.4 kNm

+ve moment capacity = 3006.3 kNm

$$\therefore \sum M_B = 3254.4 + 3006.3 = 6260.7\,\text{kNm}$$

$$1.1 \sum M_B = 6886.8\,\text{kNm}$$

$$\therefore \sum M_C > 1.1 \sum M_B$$

Hence, the strong column/weak beam requirement is satisfied.

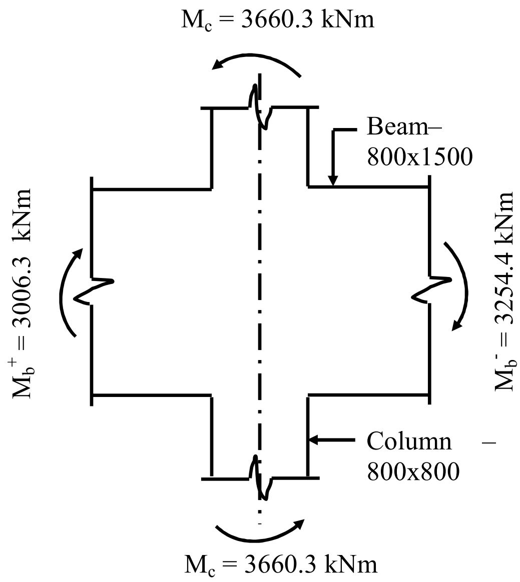


Figure 2.3: Design flexural strength at the column/beam joint

EXAMPLE 3: SEISMIC RETROFIT WITH ADDITION OF NEW SHEAR WALL TO EXISTING FRAME

Problem Statement:

For the building in Example 1, which was found to seismically deficient, can be retrofitted by the addition of new shear wall on the perimeter as shown below. This examples illustrates the design of new shear walls and connections with existing frames as per IS 13920 and the draft code on seismic evaluation and strengthening.

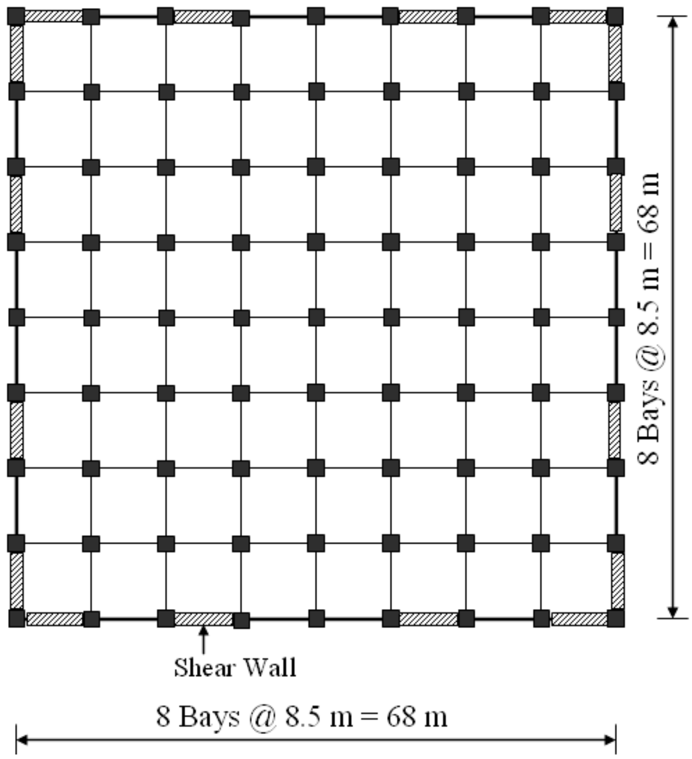


Figure 3.1: Location of new shear walls in plan

Solution:

3.1. Retrofitting With Shear Walls

New shear wall is constructed in four bays of each of the existing frames at the perimeter. Shear walls are constructed in between the panels and proper connection is made in the horizontal shear plane to carry full moment and shear and separate foundations is to be constructed for the each of new shear walls.

As per the provision of the draft code of IS 13920, new construction in seismic zone IV and V should be carried out with minimum concrete grade of M25, which is used in this example.

3.2. Load data

3.2.1. Design Seismic Base Shear

As per the draft code on seismic evaluation and strengthening, the design seismic base shear is modified considering the existing conditions of the building. Hence, the modified base shear is given as,

$V_{bm} = A_{hm}\, W = (U A_h)\, W$

where,
U = factor for reduced useable life = 0.67

A_h = design horizontal seismic coefficient is given as,

$$A_h = \frac{ZIS_a}{2Rg} \qquad \text{(as per cl.6.4.2 of IS 1893: 2002)}$$

Time period in both directions,

$T = \dfrac{0.09h}{\sqrt{d}}$ (as per cl.7.6.2 of IS 1893: 2002)

d = 68 m as the total panel length infilled by shear walls is used.

$\therefore T = \dfrac{0.09 \times 20}{\sqrt{68}} = 0.218\,\text{sec}$

Corresponding to T=0.218 sec., firm soil and 5% damping,

$\dfrac{S_a}{g} = 2.5$ (Figure 2 of IS 1893: 2002)

The building is situated in an area of high seismicity.

$\therefore Z = 0.36$ (Table 2 of IS 1893: 2002)

Importance factor,

$I = 1.0$ (Table 6 of IS 1893: 2002)

Response reduction factor,

$R = 4.0$ (Table 7 of IS 1893: 2002)

Thus,

$A_h = 0.1125$

and

A_{hm} = (0.67) A_h = 0.07538

Calculation of Seismic Weights:

Average roof dead load = 7.0 kN/m^2

Average floor dead load = 8.0 kN/m^2

Live load intensity = 3 kN/m^2

As per cl. 7.3.1 of IS 1893: 2002, imposed load to be considered in seismic weight,

25% of imposed load = 0.75 kN/m^2

Total live load on each floor except roof

$$= 0.75 \times 68^2 = 3468\,\text{kN}$$

Dead load on roof $= 7 \times 68^2 = 32.4 \times 10^3$ kN

Dead load on other floors

$$= 8 \times 68^2 = 36.9 \times 10^3\,\text{kN}$$

Seismic weight on roof $= 32.4 \times 10^3$ kN

Seismic weight on other floors

$$= 36.9 \times 10^3 + 3468 = 40.4 \times 10^3\,\text{kN}$$

Total seismic weight of the building

$$= \left(32.4 + \left(4 \times 40.4\right)\right) \times 10^3$$

$$= 194.0 \times 10^3\,\text{kN}$$

Hence,

Modified seismic base shear

= A_{hm} W = 0.67 A_h W

$= 0.67 \times 0.1125 \times 194.0 \times 10^3$ kN

$= 14.62 \times 10^3$ kN

3.2.2. Lateral Force Distribution

The calculation of lateral forces for the building is performed as shown in Example 1 and summarized in Table 3.1.

Table 3.1: Lateral force distribution

Storey	$\dfrac{W_i h_i^2}{\sum W_i h_i^2}$	Lateral force x 10^3 kN	Due to torsional moment x 10^3 kN
5	0.4	5.85	6.14
4	0.32	4.68	4.91
3	0.18	2.63	2.76
2	0.08	1.17	1.23
1	0.02	0.29	0.31
Σ		14.62	15.35

Figure 3.2 shows the lateral forces on each of the two perimeter frames resisting lateral loads in either of two principal directions.

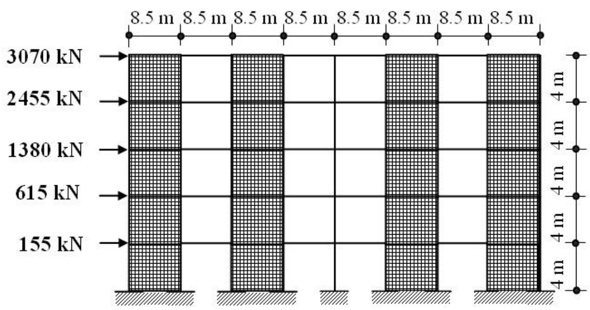
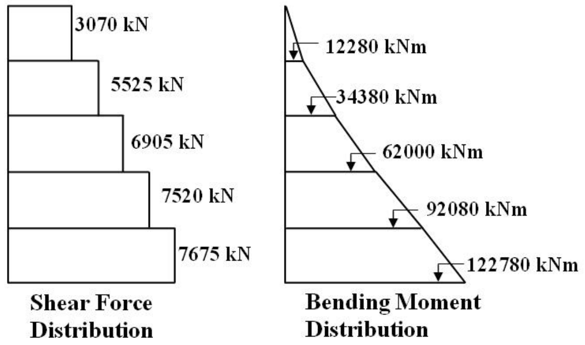
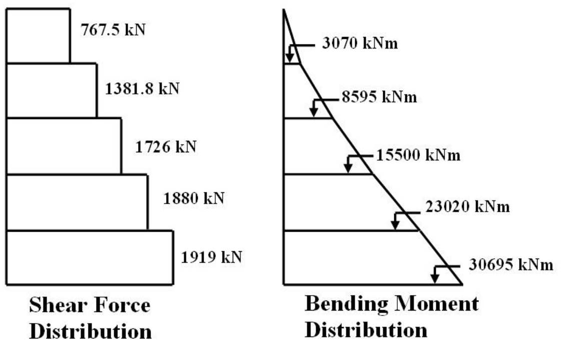


Figure 3.2 : Vertical distribution of lateral forces to the frame

It is assumed that the existing frame members offer no lateral resistance and all the shear force and bending moment is to be resisted by four identical shear walls as shown in Figure 3.3. The lateral loads will be equally shared between these shear walls.



(a) Total shear force and bending moment for four shear walls



(b) Shear force and bending moment for each of four shear walls

Figure 3.3: Design loads on new shear walls on the perimeter

Considering that axial stress in shear walls will be proportional to their distance from centre-line of frame.

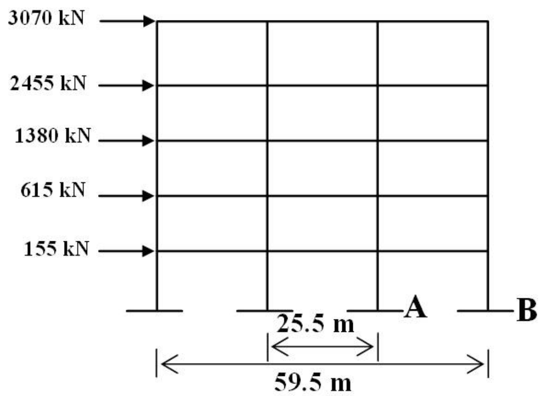


Figure 3.4 Centre line location of shear walls

$$\frac{f_A}{f_B} = \frac{25.5/2}{59.5/2}$$

$$\therefore f_B = 2.33 f_A$$

$$P_A \times 25.5 + P_B \times 59.5 = 122780$$

$$P_A (25.5 + 2.33 \times 59.5) = 122780$$

$$P_A = 748\,\text{kN}$$

DL intensity at roof level beam = 14.9 kN/m

DL intensity at floor beam = 17 kN/m

Axial load due to DL at ground floor level

$$= (14.9 + 17 \times 4) \times 7.7$$
$$= 638.2\,\text{kN}$$

Moment and shear forces from DL will be insignificant with respect to the moment and shear force due to EL, and therefore they will be not pursued further.

Axial compression from EL = 748.0 kN

Moment from EL = 30695.0 kNm

Shear from EL = 1919.0 kN

It is found that the load combination 0.8DL+1.5EL is most critical and therefore used for design. The load combination 1.5DL+1.5EL results in larger axial load, which helps in enhancing the moment resistance of the cross-section. However, moment and shear demands due to both load combinations are not different.

Axial compression,

$$P_u = (0.8 \times 638.2 + 1.5 \times 748.0) = 1632.5 \text{ kN}$$

Factored design moment,

$$M_u = 1.5 \times 30695 = 46042.5 \text{ kN-m}$$

Factored design shear,

$$V_u = 1.5 \times 1919 = 2878.5 \text{ kN}$$

Material data

Grade of concrete, $f_{ck} = 25\,\text{MPa}$

Grade of steel, $f_y = 415\,\text{MPa}$

Elastic modulus of steel, $E_s = 2 \times 10^5$ MPa

3.3. Design data

Thickness of shear wall, $t_w = 300$ mm

Thickness is more than 150 mm. Hence, ok.

Length of shear wall, $l_w = 7700\,\text{mm}$

Height of shear wall, $h_w = 2500\,\text{mm}$

Length of boundary element = 600 mm

Width of boundary element = 800 mm

Effective depth, $d_w = 6160\,\text{mm}$

Gross c/s area of the shear wall $= 2910000$ mm^2

Gross M.I. of the shear wall $=1.899\times10^{13}$ mm^2

Horizontal spacing of vertical reinforcement.,

$S_h = 100$ mm

Maximum allowable spacing is smaller of $l_w/5$, $3t_w$ and 450 mm = 450 mm

Vertical spacing of horizontal reinforcement.,

$S_v = 350$ mm

Diameter of vertical bar = 12 mm (maximum allowable dia. = 1/10th of thickness = 25 mm)

Diameter of horizontal bar = 12 mm (maximum allowable dia. = 1/10th of thickness = 25 mm)

Factored shear stress in wall

$= 1.246MPa > 0.25(f_{ck})^{0.5} = 1.25MPa$

Nominal shear stress in wall, $\tau_v = 1.558MPa$

Reinforcement shall be provided in 2 curtains as thickness is more than 200 mm.

Area of vertical reinforcement,

$A_{st,v} = 17402$ mm^2

(minimum value is 0.0025 of the gross cross sectional area = 5775 mm^2)

Area of horizontal reinforcement,

$A_{st,h} = 1614$ mm^2

(minimum value is 0.0025 of the gross cross sectional area = 1875 mm^2)

$A_{st,v} > A_{st,h}$. Hence, ok

% of vertical reinforcement = 0.753

Corresponding $\tau_c = 0.57$ MPa

Corresponding $\tau_{c,\max} = 3.1$MPa for M25

Nominal shear stress is less than maximum allowable. Hence, O.K.

Horizontal shear reinforcement is to be provided as per cl. 9.2.5 of IS 13920.

3.4. Provision of Shear Check

Shear to be resisted by horizontal reinforcement,

$V_{us} = V_u - \tau_c t_w d_w = 1825.1$ kN

Shear resisted by horizontal reinforcement

$$V_{us,provided} = \frac{0.87 f_y A_h d_w}{S_v} = 10257.9\,\text{kN}$$

Hence, O.K.

Moment of resistance calculation of rectangular web portion (as per cl. 9.3.1 of IS: 13920)

Assuming axial load to be distributed uniformly,

Ratio of axial compression carried by web portion = 0.67

Axial load on web =0.67×1632.54=1094.0 kN

Vertical reinforcement ratio, $\rho = \frac{A_{st}}{t_w l_w} = 0.008$

$$\beta = \frac{0.87 f_y}{0.0035 E_s} = 0.516$$

$$\phi = \frac{0.87 f_y \rho}{f_{ck}} = 0.109$$

$$\lambda = \frac{P_u}{f_{ck} t_w l_w} = 0.019$$

$$\frac{x_u}{l_w} = \frac{(\phi + \lambda)}{(2\phi + 0.36)} = 0.221$$

$$\frac{x_u^*}{l_w} = \frac{0.0035}{\left(0.0035 + \frac{0.87 f_y}{E_s}\right)} = 0.660$$

$$\frac{x_u}{l_w} < \frac{x_u^*}{l_w}$$

Moment of resistance calculation from formula (a) of Annex A of IS 13920

$$\frac{M_{uv}}{f_{ck} t_w l_w^2} = \phi\left[\left(1+\frac{\lambda}{\phi}\right)\left(\frac{1}{2}-0.416\frac{x_u}{l_w}\right)-\left(\frac{x_u}{l_w}\right)^2\left(0.168+\frac{\beta^2}{3}\right)\right]$$

$$\frac{M_{uv}}{f_{ck} t_w l_w^2} = 0.05075$$

$$M_{uv} = 22568.0kNm < 46042.5kNm$$

Balance amount of moment will be carried by the boundary element.

3.5. Design of Boundary Element

Axial compression in extreme fiber

due to bending = 9.33MPa

due to axial load = 0.56 MPa

Total compression = 9.89 MPa > $0.2f_{ck}$ (= 5MPa)

Provision of boundary elements along the wall edges is mandatory.

Distance between boundary elements on centers = 7100 mm

Axial force due to earthquake moments on the boundary elements as per cl. 9.4.2 of IS 13920

$$\frac{M_u - M_{uv}}{C_w} = 3306.27$$

Ratio of axial compression carried by single boundary element = 0.165

Maximum compression

$$= [3306.3 + 0.165(0.9 \times 638.2 + 1.5 \times 748)kN$$

$$= 3586.1kN$$

Maximum tension

$$= [-3306.3 + 0.165(0.8 \times 638.2 - 1.5 \times 748)kN$$

$$= -3407.1kN$$

As DL is adding to the strength of the column, load factor of 0.8 is used.

Area of steel required for tension

$$= \frac{P_{ut}}{0.87 f_y} = 9436.72 \text{ mm}^2$$

Using 28 mm dia. bars for main reinforcement

No. of bars required = 15.33

No. of bars provided = 16

Steel provided (A_{sc}) = 9852 mm^2

% Steel provided = 2.05

Assuming short column action axial load carrying capacity of the boundary element

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} = 7440.8kN$$

Hence, O.K.

Reinforcement details of new shear wall are shown in Figure 3.5.

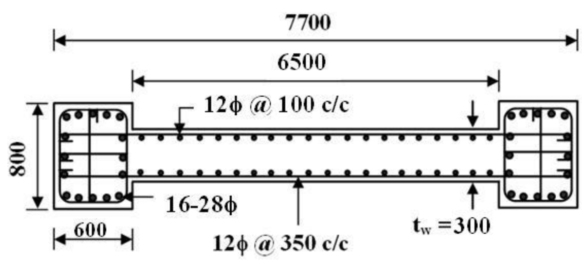


Figure 3.5: Reinforcement details for shear wall

3.6. Provision of Special Confining Reinforcement

Amount of special confining steel required as per second equation of cl. 7.4.8, of the draft IS 13920.

$$A_{sh} = 0.05Sh\frac{f_{ck}}{f_y}$$

S	= 100 mm
Clear cover	= 40 mm
No. of cross tie	= 1
h	= 360 mm
f_{ck}	= 25 MPa
f_y	= 415 MPa
A_g	= 400000 mm^2
A_k	= 302400 mm^2
A_{sh}	= 108.43 mm^2

Provide 12 mm dia stirrups.

(Area provided = 113 mm^2 > 108.43 mm^2) OK

3.7. Connection Between Shear Wall and Existing Beam of The Frame

Horizontal Shear = 2878.5 kN

The shear transfer reinforcement (dowel bars), perpendicular to the shear plane, is given as per cl. 8.5.2.1 of IS 13920 draft code.

Maximum allowable horizontal shear force

= minimum of $0.2 f_{ck} A_c$ *and* $5.5 A_c$

= minimum of 14550 kN and 16005 kN

= 14550 kN

Dowel provided at horizontal face

Area required,

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

η = Efficiency factor = 5

μ = Co-efficient of friction = 1

$$A_{vf} = 34680.7 \text{ mm}^2$$

Using 25 mm dia. bar as dowel

$$A_{vf}' = 490.87\,\text{mm}^2$$

Hence, no. of bars required $\frac{A_{vf}}{A_{vf}'} = 71$, which is to be provided over a span of 7700 mm.

Dowels are provided in 2 layers.

Spacing of dowels = 217 mm.

Providing 200 mm spacing of dowels.

As the shear walls are designed for the full value of moments and shear and separate foundation is constructed for the shear walls only connection in the horizontal face between beam and shear wall is required. The connection between columns and shear wall is not structurally required.

EXAMPLE 4: **SEISMIC EVALUATION AND STRENGTHENING OF UNREINFORCED MASONRY BUILDING**

Problem Statement:

An existing four-storey unreinforced masonry building is located in the highest seismic zone V and is founded on medium soil. The building is 13.25 m in height, 24 m in length and 10.5 m in width. The lateral load resistance to the building is provided by perimeter load bearing unreiforced masonry walls which decrease in thickness from 0.46 m at the ground floor to 0.23 m at the top floor with 0.35 m thick walls for the second and third floors. There are no interior cross walls present in the building. The short walls are perforated with openings for door and windows as shown in the Figure 4.1. The building is provided with wooden joist flooring system which results in flexible diaphragms. This example illustrates detailed evaluation of the building in accordance with the provisions of draft code on seismic evaluation and seismic strengthening with provisions of new steel braced frames as interior cross wall and shear frame at the ends.

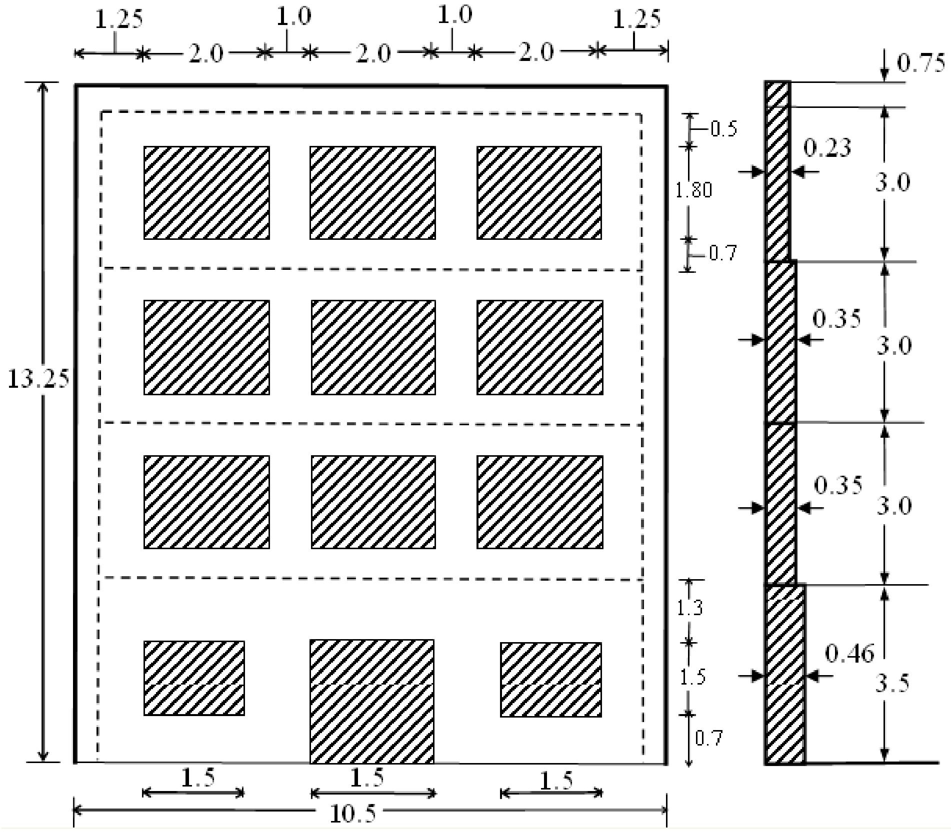


Figure 4.1: Elevation of the building

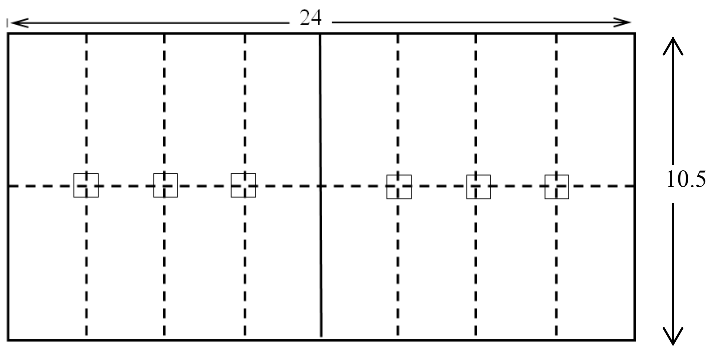


Figure 4.2: Plan of the building

PART A:

Seismic Evaluation

The URM building has flexible diaphragms at all levels and masonry walls on short ends are perforated with large openings which render them weak for in-plane shear forces. In this example, a detailed evaluation will be carried out in accordance with the Special Procedure of the draft code for the URM building with flexible diaphragms.

4.1. Detailed Evaluation

4.2. Floor and Roof Dead Loads

Floor: 1.50 kN/m^2
Roofs: 1.20 kN/m^2

4.3. Unit Weight of Walls

Weight of the wall per meter run:
For 0.23 m thick wall: 4.6 kN/m^2
For 0.35 m thick wall: 7.0 kN/m^2
For 0.46 m thick wall: 9.2 kN/m^2

4.4. Seismic Weights

4.4.1. Roof

Roof dead load = $1.2\times(10.5\text{-}0.23)\times(24\text{-}0.23)$ = 293 kN

Side walls = $4.6\times(0.75+1.5)\times24\times2$
= $4.6\times2.25\times24\times2$
= 496.8 kN

End walls = $4.6\times(0.7+0.5)\times10.5\times2$
= $4.6\times1.2\times10.5\times2$
= 120.8 kN

Walls weights between openings
= $4.6\times(10.5\text{-}6)\times1.0\times2$
= $4.6\times4.5\times1.0\times2$
= 41.4 kN

Total seismic weight on roof = 952 kN

4.4.2. Fourth Storey

Floor dead load = $1.5\times10.15\times 23.65$ = 360.1 kN

Side walls = $4.6\times1.5\times24.0\times2$ = 331.2 kN
= $7.0\times1.5\times24.0\times2$ = 504 kN

End walls = $4.6\times0.8\times4.5\times2$ = 33.2 kN
= $4.6\times 0.7\times10.5\times2$ = 67.7 kN
= $7.0\times0.5\times10.5\times2$ = 73.5 kN
= $7.0\times1.0\times4.5\times2$ = 63 kN

Total seismic weight of fourth Storey = 1434 kN

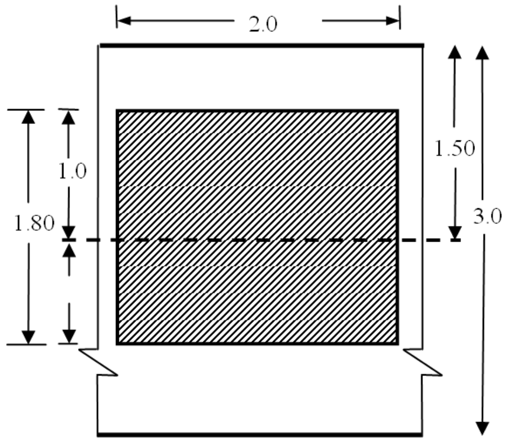


Figure 4.3: Details of opening to calculate weight

4.4.3. Third Storey

Floor dead load = $1.5\times10.15\times 23.65$ = 360.1 kN

Side walls = $7.0\times1.5\times24.0\times2$ = 504 kN
= $7.0\times1.5\times24.0\times2$ = 504 kN

End walls = $7.0\times0.8\times4.5\times2$ = 50.4 kN
= $7.0\times0.7\times10.5\times2$ = 103 kN
= $7.0\times0.5\times10.5\times2$ = 73.5 kN
= $7.0\times1.0\times4.5\times2$ = 63 kN

Total seismic weight of third Storey =1658 kN

4.4.4. Second Storey

Floor dead load = $1.5\times10.04\times 23.54$ = 354.6 kN

Side walls = $7.0\times1.5\times24.0\times2$ = 504 kN
= $9.2\times1.5\times24.0\times2$ = 662.4 kN

End walls = $7.0\times0.8\times4.5\times2$ = 50.4 kN
= $7.0\times0.7\times10.5\times2$ = 102.9 kN
= $9.2\times1.3\times10.5\times2$ = 251.2 kN
= $9.2\times0.45\times5.5\times2$ = 45.6 kN

Total seismic weight of second storey = 1971 kN

Total seismic weight of the building = 6015 kN

4.5. Demand / Capacity Ratios of Diaphragms

The special procedure is considered on the assumption that the walls and flexible diaphragms of the URM building are not coupled dynamically and that the building responds to ground motion essentially as a rigid block. As such primary concern is for material capacities and system stability both based on established displacement limits. In case of the diaphragms, a check is made for capacity based on 'elastic' deflection limit expressed through the demand-capacity ratio and diaphragm span as shown in the Figure 2 of the draft code.

Since the building has no cross walls present, the *DCR* equation to be used is:

$$DCR = \frac{2.5A_{hm}W_d}{K\sum v_u D_d} \qquad \text{(Draft Code: Eq. A1.2)}$$

Where,

DCR =Demand-capacity ratio of the diaphragm

W_d =Total dead load tributary to the diaphragm

$\sum v_u D_d$ =Sum of diaphragm shear capacities for both ends of the diaphragm.

A_{hm} = Modified base shear coefficient

K = Knowledge Factor (assumed 0.8 for minor deterioration of materials)

4.5.1. Calculation of A_{hm}

$A_{hm} = A_h U$

where,

A_h = Horizontal Seismic Coefficient (IS 1893:2002: Sec 6.4.2)

U = Factor for reduced usable life (Draft Code: Sec 5.3)

4.5.2. Calculation of A_h

$$A_h = \left(\frac{Z}{2}\right)\left(\frac{I}{R}\right)\left(\frac{S_a}{g}\right)$$

where,

Z = Zone factor

I = Importance factor

R = Response reduction factor

$\frac{S_a}{g}$ = Average response acceleration coefficient

As per the Zone IV of IS (Part 1)1893 the Horizontal Seismic coefficients are as follows:

Z = 0.24

I = 1.0

$\frac{S_a}{g}$ = 2.50

R = 1.50

$$A_h = \frac{0.24 \times 1.0 \times 2.50}{2 \times 1.5} = 0.20$$

Now, $A_{hm} = A_h U$

U = 0.67 (Draft Code: Sec 5.3)

$$A_{hm} = 0.20 \times 1.0 \times 0.67 = 0.134$$

4.5.3. Calculation of DCR Values

4.5.3.1. Roof Diaphragm

W_d = Roof load + Load on side walls
= 293 + 496.8 = 790 kN

v_u = 3.6 kN/m (Draft Code: Table A1)

In-plane width dimension of masonry = 10.27 m

$\sum v_u D_d$ = 2 x 10.27 x 3.6 = 74 kN

K= 0.8 (Draft code : Table 1)

$$DCR = \frac{2.5 \times 0.134 \times 790}{0.8 \times 74} = 4.47$$

Since this (*L, DCR* = 24, 4.47) does not fall in any of three regions of Figure 2 of the draft code, therefore, the roof diaphragm must be provided with a cross wall.

4.5.3.2. Fourth-Storey Diaphragm

W_d = 1195.3 kN

v_u = 7.3 kN/m (Draft Code: Table A1)

In-plane width dimension of masonry = 10.15 m

$\sum v_u D_d$ = 2 x 10.15 x 7.3 = 148.2 kN

K= 0.8 (Draft Code: Table 1)

$$DCR = \frac{2.5 \times 0.134 \times 1195.3}{0.8 \times 148.2} = 3.37$$

Point (24, 3.37) falls in region 2 of the Figure 2, therefore fourth storey diaphragm does not need cross walls.

4.5.3.3. Third Storey Diaphragm

W_d = 1368.1 kN

v_u = 7.3 kN/m (Draft Code: Table A1)

In-plane width dimension of masonry = 10.15 m

$\sum v_u D_d$ = 2 x 10.15 x 7.3 = 148.2 kN

K= 0.8 (Draft Code: Table 1)

$$DCR = \frac{2.5 \times 0.134 \times 1368.1}{0.8 \times 148.2} = 3.85$$

Point (24, 3.85) falls in region 2 of the Figure 2, therefore fourth storey diaphragm does not need cross walls.

4.5.3.4. Second-Storey Diaphragm

W_d = 1521 kN

v_u = 7.3 kN/m (Draft Code: Table A1)

In-plane width dimension of masonry = 10.04 m

$\sum v_u D_d$ = 2 x 10.04 x 7.3 = 146.6 kN

K= 0.8 (Draft Code: Table 1)

$$DCR = \frac{2.5 \times 0.134 \times 1521}{0.8 \times 146.6} = 4.34$$

Point (24, 4.34) does not fall in any of the Region of Figure 2. Hence cross walls need not to be designed.

The above calculations are summarized in Table 4.1. Points (*L, DCR*) of 4th, 3rd, and 2nd storey building diaphragm fall in the Region 2 of the Figure 2, therefore, they are acceptable for special procedure. But the points (*L, DCR*) of the roof diaphragm doesn't fall in any of the acceptable regions of the Figure 2, therefore the diaphragm span of 24 m is not acceptable for the more flexible roof diaphragm. Introduction of an interior cross wall at the mid-span will help stiffen the diaphragm and control its in-plane deformation.

4.6. Strength Check of Diaphragms

The ability of floor and roof diaphragms to transfer lateral forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm needs to be checked.

Floor and roof diaphragms should be able to resist diaphragm forces F_{px} as given below:

$$F_{px} = \frac{\sum_{i=x}^{n} Q_i}{\sum_{i=x}^{n} w_i} w_{px}$$

where, w_{px} is weight of roof or floor diaphragm at the level x and Q_i and w_i are lateral loads and seismic weights at the *i*-th floor.

The force F_{px} determined from the above equation shall not be more than 0.75 ZIw_{px} and less than 0.35 ZIw_{px}.

The calculations of these diaphragm forces are shown as follows:

4.6.1. Design Seismic Base Shear

$V_B = A_{hm} W$

$= 0.134 \times 6015$

= 806 kN

4.6.2. Distribution of Base Shear to Floor Levels

The distribution of seismic forces should be done according to IS 1893(Part 1) and the following formula should be used for the purpose.

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Where,

Q_i = Design lateral force at floor i,
W_i = Seismic Weight of the floor
h_i = Height of the floor I measured from base
n = number of storey in the building. Number of levels at which the masses are located.

Clearly if there are no vertical resisting elements other than the end walls, then the roof diaphragm is likely to be the most stressed. The Diaphragm's shear transfer force per unit length is

$$\frac{301.5}{2 \times 10.15} = 14.9 \text{ kN/m}$$

which is larger than the largest capacity of 12.0 kN/m allowed. Another line of vertical resisting element in the middle of the building is required to reduce the span to one half. Since the floors are flexible, the lateral loads are distributed by tributary area, i.e, each end vertical elements carries one fourth of the seismic shear, while the center element carries the remaining half. Now the maximum design diaphragm shear force decreases to

$$\frac{301.5}{4 \times 10.15} = 7.43 \text{ kN/m}$$

Which is less then allowable values for floors with straight or diagonal sheathing and finished wood flooring.

Table 4.1: Checking Acceptability of Diaphragm Span (Special Procedure)

Level	D, m	W_d, kN	$\sum v_u D_d$, kN	$DCR = \dfrac{2.5A_{hm}W_d}{K\sum v_u D_u}$	Region from Figure 2 of Draft Code for Coordinates (L, DCR)
Roof	10.27	790.0	74.0	4.47	None(24, 4.47)
4	10.15	1195.3	148.2	3.37	Region 2 (24, 3.37)
3	10.15	1368.1	148.2	3.85	Region 2 (24, 3.85)
2	10.05	1521.0	146.6	4.28	Region 2 (24, 4.28)

Table 4.2 : Checking Strength of the Diaphragm

Level	$Q_i = V_B \dfrac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$	$\sum_{i=x}^{n} Q_i$, kN	w_i, kN	$\sum_{i=x}^{n} w_i$	w_{px} kN	$0.35\ ZIw_{px}$ kN	$0.75\ ZIw_{px}$ kN	$F_{px} = \dfrac{\sum_{i=x}^{n} Q_i}{\sum_{i=x}^{n} w_i} w_{px}$ kN	$\dfrac{F_{px}}{w_{px}}$
Roof	322	322	952	952	790.0	66.4	142.2	**265.5**	0.34
4	280	602	1434	2386	1195.1	100.4	215.2	**301.5**	0.25
3	152	754	1658	4044	1368.1	114.9	246.3	**255.0**	0.18
2	52	806	1971	6015	1521.0	127.8	273.8	**203.8**	0.134

4.7. *h/t* Ratio for Walls

According to special procedure, the walls are judged for out-of-plane stability based on their *h/t* (height/thickness) ratios.

From clause 7.4.3 (a), the allowable *h/t* ratio:

Top Storey of Multi-storey Building: 9
First Storey of Multi Storey Building: 15
All other conditions: 13

4.7.1. Actual h/t Ratios

Top Storey: $\dfrac{3.0}{0.23}=13.0>9.0$ Do not satisfy.

(Does not govern)

First Storey: $\dfrac{3.3}{0.46}=7.17<15$ O.K.

Other Storey: $\dfrac{3.0}{0.35}=8.60<13$ O.K.

∴ Top storey needs bracing.

4.8. Diaphragm Shear Transfer

As per the special procedure, the design connection between the diaphragm and the masonry walls is to be the lesser of:

$V_d = 1.5A_{hm}C_pW_d$ or (Draft Code: Eq A1.6)

$V_d = v_u D_d$ (Draft Code: Eq A1.7)

where, C_p is the coefficient as given in Table 3 of draft code. Shear connectors should have capacity to transfer these forces from diaphragm to end walls which are computed as below for the study building:

4.8.1. Roof Level

C_p = 0.50
A_{hm} = 0.134
W_d = 790 kN
v_u = 3.6 kN/m

$V_d = 1.5 \times .134 \times 0.50 \times 790 = 79.40$ kN or

$V_d = 3.6 \times 10.27 = 37$ kN

4.8.2. Fourth-Storey

C_p = 0.50
A_{hm} = 0.134
W_d = 1195.3 kN
v_u = 7.3 kN/m

$V_d = 1.5 \times 0.134 \times 0.50 \times 1195.3 = 120.1$ kN or

$V_d = 7.3 \times 10.15 = 74$ kN

4.8.3. Third-Storey

C_p = 0.50
A_{hm} = 0.134
W_d =1369 kN
v_u =7.3 kN/m

$V_d = 1.5 \times 0.134 \times 0.50 \times 1369 = 137.6$ kN or

$V_d = 7.3 \times 10.15 = 74$ kN

4.8.4. Second-Storey

C_p = 0.50
A_{hm} = 0.134
W_d = 1521 kN
v_u =7.3kN/m

$V_d = 1.5 \times 0.134 \times 0.50 \times 1521 = 152.9$ kN or

$V_d = 7.3 \times 10.05 = 73.4$ kN

4.9. In-Plane Shears for Masonry Walls

4.9.1. Design In-Plane Shears

$$F_{wx} = A_{hm}(W_{wx} + 0.5W_d)$$

But should not to exceed,

$$F_{wx} = A_{hm} \times W_{wx} + v_u D_d$$

where,
F_{wx} = Force applied to a wall at level x
W_{wx} = Dead load of an unreinforced masonry wall assigned to level x halfway above and below the level under consideration

4.9.1.1. Roof Level

Side (Long) Walls

$$W_{wx} = \frac{4.6 \times 2.25 \times 24 \times 2}{2} = 248.4 \text{ kN}$$

W_d = 790 kN

$F_{wx} = 0.134(248.4 + 0.5 \times 790) = 86.2$ kN

but should not exceed,

$F_{wx} = 0.134 \times 248.4 + 3.6 \times 23.77 = 118.9$ kN

$\therefore$ use, F_{wx} = 86.2 kN

End (Short) Walls

$$W_{wx} = \frac{120.8+41.4}{2} = 81.1 \text{ kN}$$

W_d = 790 kN

$F_{wx} = 0.134(81.1 + 0.5 \times 790) = 63.8$ kN

but should not exceed,

$F_{wx} = 0.134 \times 81.1 + 3.6 \times 10.27 = 47.9$ kN

47.9 < 63.8, $\therefore$ use, F_{wx} = 47.9 kN

4.9.1.2. Fourth Storey

Side (Long) Walls

W_{wx} = 417.6 kN
W_d = 1195.1 kN
$F_{wx} = 0.134(417.6 + 0.5 \times 1195.1) = 136.1$ kN
but should not exceed,

$F_{wx} = 0.134 \times 417.6 + 7.3 \times 23.65 = 229.5$ kN

$\therefore$ use, F_{wx} = 136.1 kN

End (Short) Walls

$W_{wx} = 118.7$ kN

W_d = 1195.1 kN
$F_{wx} = 0.134(118.7 + 0.5 \times 1195.1) = 96$ kN

but should not exceed,

$F_{wx} = 0.134 \times 118.7 + 7.3 \times 10.15 = 90$ kN

90 < 96, $\therefore$ use, F_{wx} = 90kN

4.9.1.3. Third Storey

Side (Long) Walls

W_{wx} = 504 kN
W_d = 1368.1 kN
$F_{wx} = 0.134(504 + 0.5 \times 1368.1) = 159.2$ kN

but should not exceed,

$F_{wx} = 0.134 \times 504 + 7.3 \times 23.65 = 241.1$ kN

$\therefore$ use, F_{wx} = 159.2 kN

End (Short) Walls

$W_{wx} = 145$ kN

W_d = 1368.1 kN

$F_{wx} = 0.134(145 + 0.5 \times 1368.1) = 111$ kN

but should not exceed,

$F_{wx} = 0.134 \times 145 + 7.3 \times 10.15 = 93.5$ kN

93.5 <111, $\therefore$ use, F_{wx} = 93.5 kN

4.9.1.4. Second Storey

Side (Long) Walls

W_{wx} = 356.2 kN

W_d = 1521 kN

$F_{wx} = 0.134(356.2 + 0.5 \times 1521) = 149.7$ kN

but should not exceed,

$F_{wx} = 0.134 \times 356.2 + 7.3 \times 23.54 = 219.6$ kN

$\therefore$ use, F_{wx} = 149.7 kN

End (Short) Walls

$W_{wx} = 225$ kN

W_d = 1521 kN

$F_{wx} = 0.134(225 + 0.5 \times 1521) = 132$ kN

but should not exceed,

$F_{wx} = 0.134 \times 225 + 7.3 \times 10 = 103.2$ kN

103.2 <132, $\therefore$ use, F_{wx} =103.2 kN

Table 4.3: Summary of Storey Shear Forces

Wall	Level	Storey Force F_{wx} kN	Wall Storey Shear Force $\sum F_{wx}$ kN
Side (Long) Walls	Roof	86.2	86.2
	4th	136.1	222.3
	3rd	159.2	381.5
	2nd	149.7	531.2
End (Short) Walls	Roof	47.9	47.9
	4th	90.0	137.9
	3rd	93.5	231.4
	2nd	103.2	334.6

4.10. Check for In plane Shears Strength of End (Short) Masonry Walls

The end (short) masonry walls are weakened by openings in in-plane shear strength. These walls need to be checked for the shear forces computed above in Section 4.9 (Figure 4.4 and 4.5). The wall piers of end walls are analysed in accordance with Sec 7.4.3(b) of the draft code. These calculations are summarized in Table 4.4.

The value of mortar shear strength v_t *is* assumed to be 0.4 MPa, a typical value for old masonry buildings. The analysis showed that the top three stories are rocking-critical, whereas the bottommost story is shear-critical. The following formulae are used to calculate the shear and rocking strength of shear walls.

Shear wall strength,

$$V_a = v_a D t \qquad \text{(Draft Code: Eq 7.1)}$$

$$v_a = 0.1 v_{te} + 0.15 \frac{P_{CE}}{A_n} \qquad \text{(Draft Code: Eq 7.2)}$$

Rocking Shear Strength

$$V_r = 0.5 P_D \frac{D}{H} \qquad \text{(Draft Code: Eq 7.4)}$$

If $V_a > V_r$ (Rocking controlled mode), the rocking shear capacity of pier will be less than pier shear capacity.

If $V_a < V_r$ (Shear controlled mode), the shear capacity of pier will be less than pier rocking shear capacity.

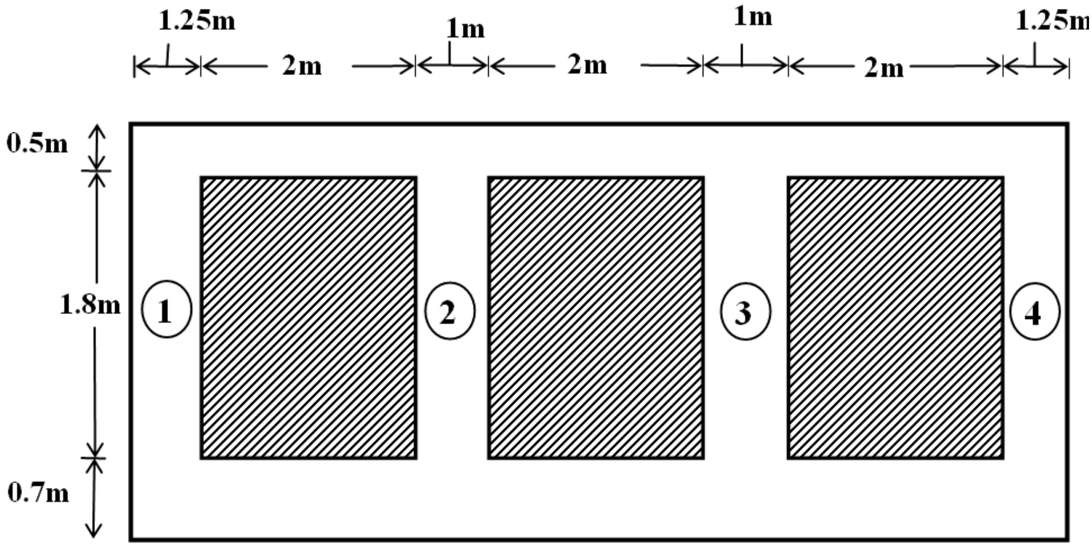


Figure 4.4: Section of end wall at 2nd, 3rd and 4th storey

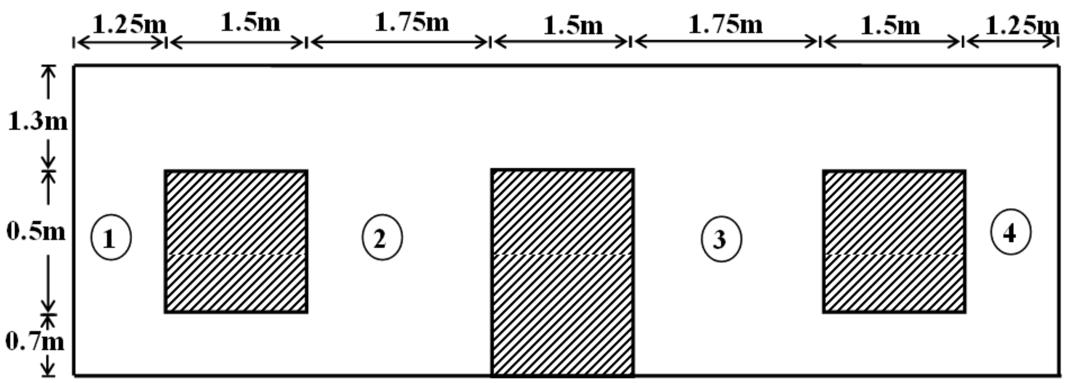


Figure 4.5: Section of end wall at ground floor

Table 4.4: Summary of Pier Analysis

Story	Pier	P_d, kN	D, m	H, m	t, m	V_a, kN	V_r, kN	Mode
4	1 & 4	20.1	1.25	1.8	0.23	14.5	7.0	Rocking
	2 & 3	40.5	1.00	1.8	0.23	15.3	11.3	Rocking
3	1 & 4	36.8	1.25	1.8	0.35	23.0	12.9	Rocking
	2 & 3	99.5	1.00	1.8	0.35	28.9	27.9	Rocking
2	1 & 4	53.5	1.25	1.8	0.35	25.5	18.7	Rocking
	2 & 3	158.5	1.00	1.8	0.35	37.8	44.4	Rocking
	1 & 4	85.7	1.25	1.5	0.46	35.7	35.7	Shear
	2 & 3	210	1.75	1.5	0.46	71.5	122.5	Shear

PART B:

Strengthening of Building – New Cross Wall and Out-of-plane Wall Bracing

4.11. Design of Cross wall Frames

Cross walls are required where diaphragm span exceeds the limits as indicated in 4.6 of Part A. A brace frame will be provided at midway at all levels to reduce the diaphragm span. A chevron braced frame will be designed as a cross wall. The braced frame member forces are based on capacity of diaphragm and shown in Figure 4.6.

As per draft code sec A1.1(c), the minimum capacity of cross walls is 30% of diaphragm capacity. The diaphragm capacities are calculated as follows:

$$C_{diaphragm} = 30\% \times v_u \times D_d$$

$$C_{roof} = 0.30 \times 3.6 \times 10.5 = 11.40\text{kN}$$

$$C_{floor} = 0.30 \times 7.3 \times 10.5 = 23\text{kN}$$

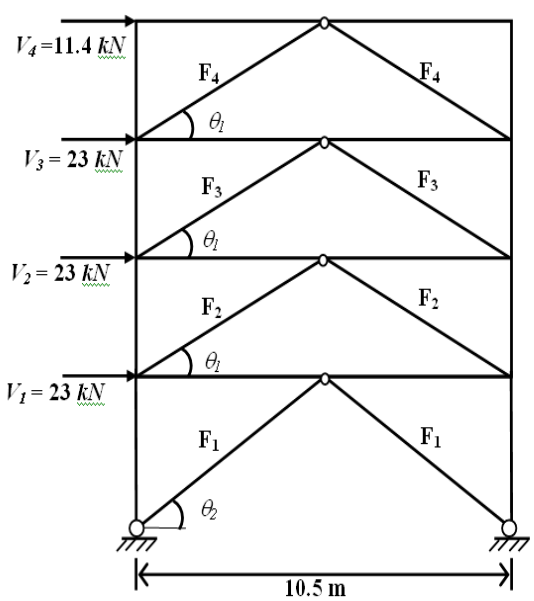


Figure 4.6: Steel braced frame as cross wall frame

4.11.1. Design of Frame Members

4.11.1.1. Design of Inclined Members

Angle of inclinations as shown in figure:

$$\theta_1 = \tan^{-1}\left(\frac{3.0}{5.25}\right) = 29.75^\circ$$

$$\theta_2 = \tan^{-1}\left(\frac{3.5}{5.25}\right) = 33.69^\circ$$

Forces in each brace:

$$F_4 = \frac{V_4}{2\cos\theta_1} = 6.55\,\text{kN}$$

$$F_4 = \frac{V_4 + V_3}{2\cos\theta_1} = 19.8\,\text{kN}$$

$$F_4 = \frac{V_4 + V_3 + V2}{2\cos\theta_1} = 33\,\text{kN}$$

$$F_1 = \frac{V_4 + V_3 + V_1}{2\cos\theta_2} = 48.3\,\text{kN}$$

The inclined members will be designed as compression members in accordance with IS 800 and draft code provisions. Assume f_y=250 N/mm^2 and effective length factor *k*=1

Table 4.5: Summary of Design of Crosswalls

Story	Braces Members		
	Design Force, kN	Length, m	Member Sections
4	6.55	6.0	ISJB 150 $A_{req=}3.0$ cm^2 $A_{prov}= 9.01$cm^2 kl/r =100.34 $P_{allowable}$ = 72.1 kN ∴ ok.
3	19.8	6.0	
2	33.0	6.0	
1	48.3	6.3	ISJB 150 $A_{req=}4.39$cm^2 $A_{prov}= 9.01$cm^2 kl/r = 105.35 $P_{allowable}$ = 68.76 kN ∴ ok.

4.11.1.2. Design of Column

Design force in Columns:

$$F_5 = \frac{23(3.5+6.5+9.5)+11.40\times 12.5}{10.5} = 57\text{kN}$$

k=2

Table 4.6: Summary of Design of Columns

Story	Design Force, kN	Column Members	
		Length, m	Member Sections
4	57	3.0	ISJB 150 $A_{req=}5.3$cm^2 $A_{prov}= 9.01$cm^2 kl/r =50.16 $P_{allowable}$ = 72kN $\therefore$ ok.
3			
2			
1	57	3.5	ISJB 150 $A_{req}= 5.3$cm^2 $A_{prov}= 9.01$cm^2 kl/r = 117 $P_{allowable}=60.2$kN $\therefore$ ok.

4.11.1.3. Design of Chord Member

Chord members should be designed as tension members and their design is summarised in table 4.7. The maximum permissible tensile stress as per IS 800: 1984 = $0.60\times f_y$=150N/mm^2.

Table 4.7: Summary of Design of Chords

Strorey	Design Force, kN	A_{req} cm^2	Section Choosed	$A_{provided}$ cm^2
4	11.4	0.76	ISJB 150	9.01
3	23.0	1.53		
2	23.0	1.53		
1	23.0	1.53		

4.11.2. Design of Column Base

Providing M15 Concrete pedestal.

Safe bearing pressure in concrete = 4 kN/m

Axial Load = 57kN

For ISJB150

D = 150mm

B =50mm

Area of slab base = $\dfrac{57}{4000} = 0.01425\text{m}^2$

Providing a square base plate

Width = $\sqrt{0.01425} = 0.119\text{m}^2$

The dimensions are not sufficient to accommodate column section and anchor bolts therefore providing a base plate of dimensions 300×200mm.

w = pressure on underside of the base

$$= \frac{57\times 10^3}{300\times 200} = 0.95N/\text{mm}^2$$

The thickness of base plate, is given by

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

$$a = \frac{200-50}{2} = 75 \qquad b = \frac{300-150}{2} = 75$$

$$t = \sqrt{\frac{3\times 0.95}{185}\left(75^2 - \frac{75^2}{4}\right)} = 6.06\text{mm}$$

Provide 8 mm thick plate.

4.11.3. Connections

4.11.3.1. Beam/Column Connection

The Beam-Column connection will be designed as welded connection for collector force at that level.

Providing 6mm thick fillet weld (IS 800: 1984)
Force in chord member = 23 kN
Effective throat thickness of weld $= 0.7\times 6$
$= 4.2$mm

Weld Length $= \dfrac{23\times 10^3}{4.2\times 108} = 50.7\text{mm} \approx 55\text{mm}$

The weld length should be provided on both flanges of 28mm at all levels in the frame section. The details are show in Figure 4.7.

4.11.3.2. Gusset Plate to Beam and Columns

Assuming a 12 mm gusset plate to connect Brace members to Beam-Column junction. In order to find out design forces for connection the force coming from brace members are resolved into its components.

$$F_x = 48.3\cos\theta_2 = 40.2\text{kN}$$
$$F_y = 48.3\sin\theta_2 = 26.8\text{kN}$$

Weld length between plate and column

$$= \frac{40.2 \times 10^3}{4.2 \times 108} = 88.62\text{mm} \approx 90\text{mm}$$

Weld length between plate and chord

$$= \frac{26.8 \times 10^3}{4.2 \times 108} = 59\text{mm} \approx 60 \text{ mm}$$

The weld length should be provided on both sides of plate with flanges of I sections. The details are show in figure 4.8.

4.11.3.3. Braces Members and Gusset Plate

The weld connection between brace member and gusset plate is provided.

$$\text{Weld Length} \frac{48 \times 10^3}{4.2 \times 108} = 105.8\text{mm} \approx 110\text{mm}$$

The weld length should be provided on both flanges of races members with the plate. The details are show in Figure 4.8.

4.11.3.4. Weld Joint Between Column and Base Plate

The load on column = 58.3kN

A fillet weld between the plate and the column will be provided, around the ISJB150.

Gross length available for welding along the periphery of the section

$$= 2[2 \times 50\text{-}3\text{+}150\text{-}4.6] = 484.4\text{mm}$$

less the end returns of the weld at the rate of 2 sizes at each end is

$$b = 484.4 - 2(4\text{-}2)2a = 484.4\text{-}2a$$

Since the size of the weld is not yet known, assume a size and check the design.

Let size of weld = a = 6 mm, then length of weld

$$= b = 484.4\text{-}24 \times 6 = 340.4$$

Allowable stress in compression= 150 N/mm^2

Weld capacity = F_w = 0.707 × 6 × 150 = 630N/mm

Weld length required =

$$\frac{P}{F_w} = \frac{58.3 \times 10^3}{630} = 92.53\text{mm} < 340.4\text{mm} \therefore \text{ OK}$$

4.11.3.5. Anchor Bolts

Since the force acting is very low so a nominal size anchors bolts should be designed for it.

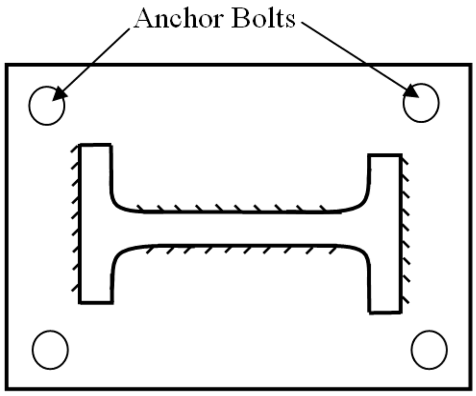


Figure 4.7: Base plate

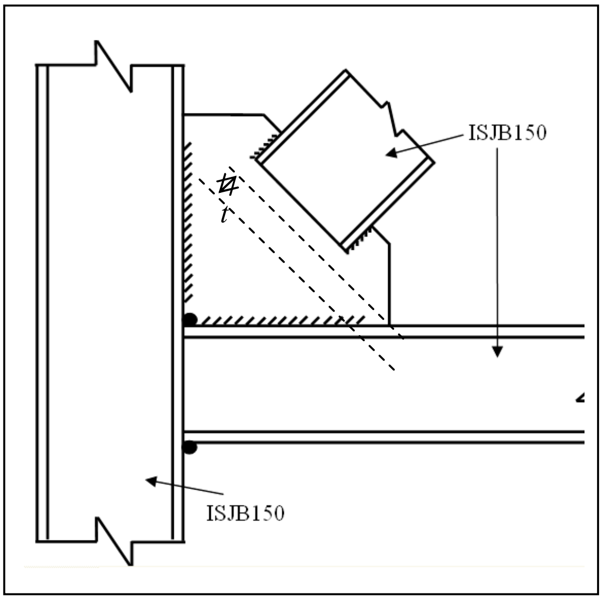


Figure 4.8: Beam column junction details

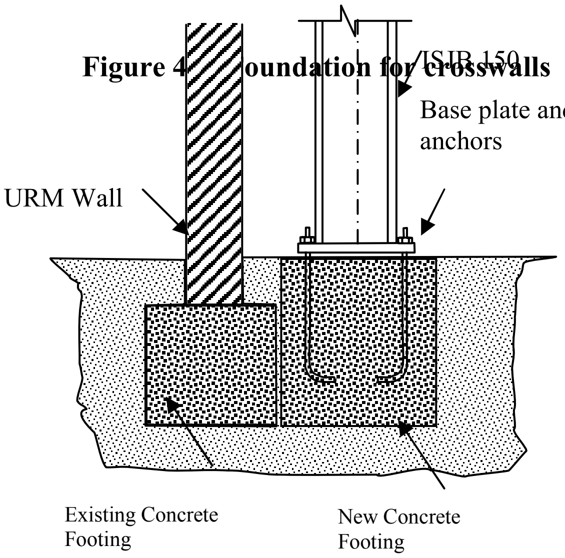


Figure 4.9: Foundation for crosswalls

4.12. Design of Wall Bracings

The analysis in Section 4.7 showed that the need of wall bracing at the top storey.

The spacing of bracings should not be more than $h/2$ = 1.5 m

$\therefore$ spacing between the bracings = 1.5m c/c

Uniform out-of-plane load on brace(w) =

$$w = 0.5Z\left(1+\frac{z}{H}\right)I_p W_e \qquad \text{(IS 1893:2002)}$$

$W_e = 1.5 \times 4.6 = 6.9\,kN/m$

$$w = 0.5\times 0.24\left(1+\frac{12.5}{12.5}\right)\times 1.0\times 6.9 = 1.656\text{kN/m}$$

$$M = \frac{1.656\times 3^2}{8} = 1.86\text{kNm}$$

$$\sigma_{bc} = 0.66 f_y = 0.66\times 250 = 165\text{N/mm}^2$$

$$Z_{req} = \frac{1.86\times 10^3}{165} = 11.2\text{cm}^3$$

Try channel section ISJC125

$$\Delta_{actual} = \frac{5\times 1.65\times\left(3\times 10^3\right)^4}{384\times 2\times 10^5\times 37.9\times 10^4} = 22.95\text{mm}$$

$$\Delta_{allowable} = \frac{\text{wall thickness}}{10} = \frac{230}{10} = 23\text{mm}$$

Since $\Delta_{actual} < \Delta_{allowable}$
$\therefore ok$

The channel should be bolted through the wall at $1/4^{th}$ points of wall height as shown in Figure 4.10.

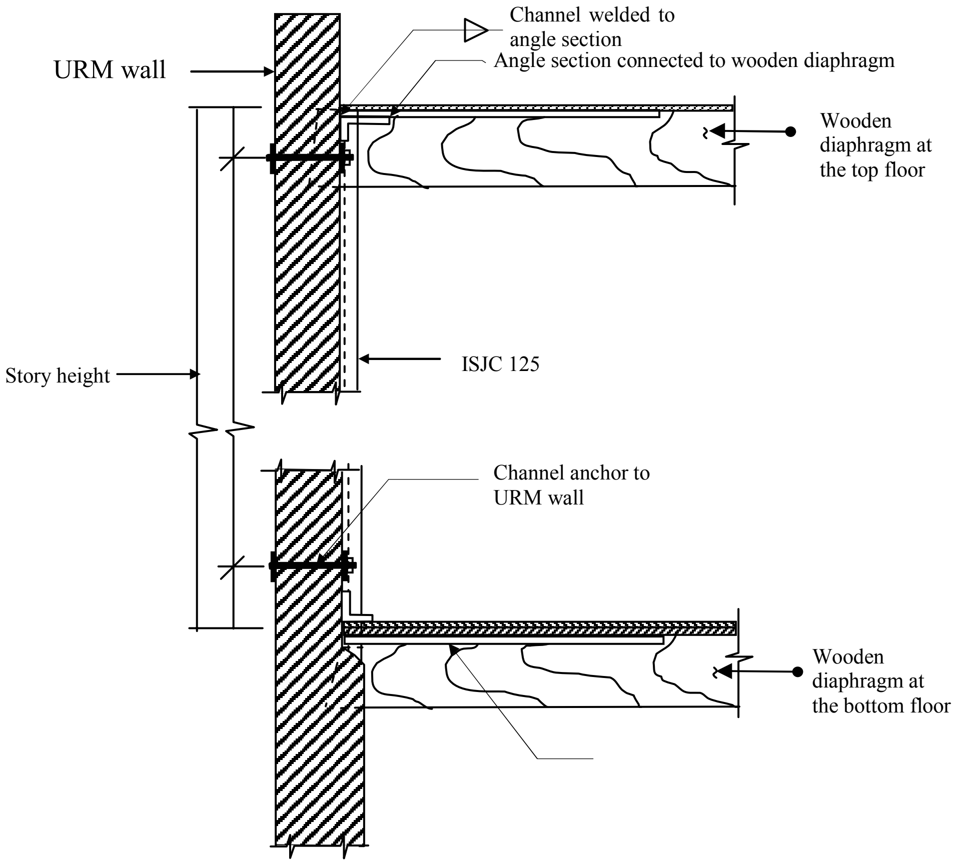


Figure 4.10: Bracing details at top storey

PART C:

Strengthening of Building – Steel Braced Frame for In-Plane Shear Resistance of End Walls

4.13. Design of Steel Frame to Replace End Wall

The existing end masonry walls are essentially open at the first level and the effective wall area is very small because of large openings. End walls can be either replaced with structural steel braced frames or installed next to the wall. In either case, it is assumed that full lateral resistance will be provided by the new braced frames. The steel frame will be designed as shear wall in accordance with the Draft Code.

For this example, it is assumed that the steel braced frame will be provided inside the building next to the end masonry wall. If a total redesign of first floor and entrance were possible, replacement of end walls with RCC or masonry frame might be considered. Such a scheme would allow for the provision of brick veneer to match existing masonry.

Using previously derived story forces and assumed member sizes as in Section 4.9 the following member forces are derived.

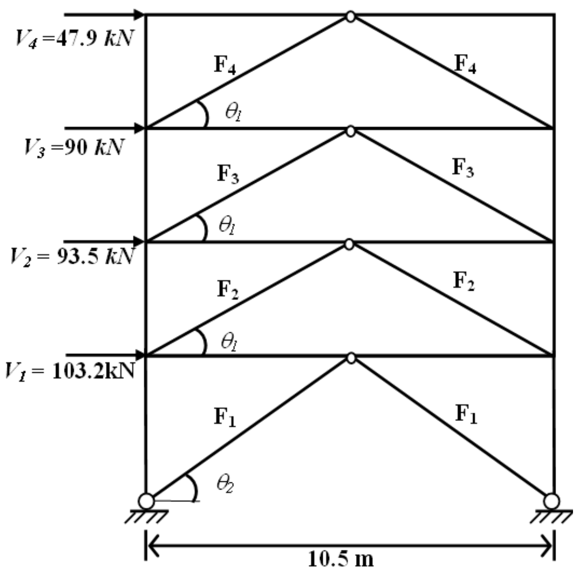


Figure 4.11: Steel Braced Frame as Shear Frame for In-Plane Resistance

4.13.1. Design of Frame Members

4.13.1.1. Design of Frame Members

Forces in members:

$$F_4 = \frac{V_4}{2cos\theta_1} = 27.6\,\text{kN}$$

$$F_3 = \frac{V_4 + V_3}{2cos\theta_1} = 79.3\ \text{kN}$$

$$F_2 = \frac{V_4 + V_3 + V_2}{2cos\theta_1} = 133\ \text{kN}$$

$$F_1 = \frac{V_4 + V_3 + V_2 + V_1}{2cos\theta_2} = 200.8\,\text{kN}$$

The braced members will be designed as compression members as per IS 800:1980 and the Draft Code. Assume f_y= 250 N/mm^2 and effective length factor k=1

Table 4.8 : Summary of Design of Steel Frame

Story	Frame Members		
	Design Force, kN	Length, m	Member Sections
4	27.6	6.0	ISJB 150 A_{req}= *2.5*cm^2 A_{prov}= *9.01*cm^2 kl/r =100.34 $P_{allowable}$ = 72.1kN $\therefore$ ok.
3	79.3	6.0	ISJB 175 A_{req} = *7.18*cm^2 A_{prov} = *10.28*cm^2 kl/r = 87.84 $P_{allowable}$ = 94.9 kN $\therefore$ ok.
2	133	6.0	ISLB 150 A_{req}= *12.09cm^2* A_{prov}= *18.08*cm^2 kl/r = 97.2 $P_{allowable}$ = 149.6 kN $\therefore$ ok.
1	200.8	6.3	ISLB 175 A_{req}= *18.25*cm^2 A_{prov}= *21.3*cm^2 kl/r = 87.9 $P_{allowable}$ =197 kN $\therefore$ ok.

4.13.1.2. Design of Column

Design force for Columns

$$F_5 = \frac{47.9\times12.5+90\times9.5+93.5\times6.5+103.2\times3.5}{10.5}$$

$= 230.8$kN

Length = 3500mm k=2

Table 4.9 : Summary of Design of Columns

Story	Design Force, kN	Column Members	
		Length, m	Member Sections
4	230.8	3.0	ISLB 200 A_{req}=*20.98*cm^2 A_{prov}= *25.27*cm^2 kl/r = 73.26 $P_{allowable}$ = 274kN ∴ ok.
3			
2			
1	230.8	3.5	ISLB 200 A_{req}= *20.98*cm^2 A_{prov}= *25.27*cm^2 kl/r = 85.4 $P_{allowable}$=*240*kN ∴ ok.

4.13.1.3. Design of Chord Member

Design of chord members should be designed as tension members and their design is summarised in table 4.10. The maximum permissible tensile stress as per IS 800: 1984 = $0.60\times f_y$=150N/mm^2.

Table 4.10 : Summary of Design of Chords

Strorey	Design Force, *kN*	A_{req} cm^2	Section Choosed	$A_{provided}$ cm^2
4	47.9	3.19	ISJB 150	9.01
3	90.0	6.0		
2	93.5	6.23		
1	103.2	6.88		

4.13.2. Design of Column Base

Providing M15 Concrete pedestal.

Safe bearing pressure in concrete = 4kN/m

Axial Load = 230.8kN

For ISLB200

D = 200mm
B =100mm

Area of slab base = $\frac{230.8}{4000} = 0.0577\text{m}^2$

Provide a square base plate $\sqrt{0.0577} = 0.24\text{m}$

∴ provide a square plate of dimensions 300×300mm

w = pressure on underside of the base

$$= \frac{230.8\times10^3}{300\times300} = 2.56\text{N/mm}^2$$

The thickness of base plate,

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

$$a = \frac{300-100}{2} = 100 \qquad b = \frac{300-200}{2} = 50$$

$$t = \sqrt{\frac{3\times2.56}{185}\left(100^2 - \frac{50^2}{4}\right)} = 19.72\text{mm} \approx 20\text{mm}$$

Provide the plate of thickness 20mm.

4.13.3. Connections

4.13.3.1. Beam/Column Connection

The Beam-Column connection will be designed as welded connection for collector force at that level.

Providing 6 mm thick fillet weld (IS 800: 1984)
Force in chord member = 23*kN*
Effective throat thickness of weld = 0.7×6
= 4.2 mm

Weld Length = $\frac{103.2\times10^3}{4.2\times108}$ =227.5mm ≈230 mm

The weld length should be provided on both flanges of 28mm at all levels in the frame section. The details are show in Figure 4.12

4.13.3.2. Gusset Plate to Beam and Columns

Assuming a 12 mm gusset plate to connect Brace members to Beam-Column junction. In order to find out design forces for connection the force coming from brace members are resolved into its components.

$F_x = 200.8 \times \cos\theta_2 = 167\text{kN}$

$F_y = 200.8 \sin\theta_2 = 111.4\text{kN}$

Weld length between plate and column

$$= \frac{167 \times 10^3}{4.2 \times 108} = 369\text{mm}$$

Weld length between plate and chord

$$= \frac{111.4 \times 10^3}{4.2 \times 108} = 245.6\text{mm}$$

The weld length should be provided on both sides of plate with flanges of I sections. The details are show in Figure 4.12.

4.13.3.3. Braces Members and Gusset Plate

The weld connection between brace member and gusset plate is provided.

Weld Length $\frac{200.8 \times 10^3}{4.2 \times 108} = 442.7\text{mm} \approx 445\text{mm}$

The weld length should be provided on both flanges of races members with the plate. The details are show in Figure 4.13.

4.13.3.4. Weld Joint Between Column and base Plate

The load on column = 200.8kN

A fillet weld between the plate and the column will be provided, around the ISLB200.

Gross length available for welding along the periphery of the section

$$= 2[2 \times 100\text{-}5.4\text{+}200\text{-}7.3] = 774.6\text{mm}$$

less the end returns of the weld at the rate of 2 sizes at each end is

$$b = 774.6 - 2(4\text{-}2)2a = 484.4\text{-}2a$$

Since the size of the weld is not yet known, assume a size and check the design.

Let size of weld = a = 6 mm, then length of weld

$$= b = 774.6\text{-}24 \times 6 = 630.6$$

Allowable stress in compression= 150 N/mm^2

Weld capacity = F_w = $0.707 \times 6 \times 150$ = 630N/mm

Weld length required =

$$\frac{P}{F_w} = \frac{200.8 \times 10^3}{630} = 319\text{mm} < 630.6\text{mm} \therefore \text{ ok.}$$

4.13.3.5. Anchor Bolts

Intial pretension in the bolts is assumed to be approximately equal to compression. Selecting four bolts, the required pretension in each bolt

$$P_i = \frac{P}{4} = \frac{200.8}{4} = 50.2\text{kN}$$

Approximate gross area of the bolt

$$A_g = \frac{P_i}{0.63 f_y} = \frac{50.2 \times 10^3}{0.63 \times 250} = 318.7\text{mm}^2$$

Required diameter of bolt = 4.5 mm

Since the size of bolt is very less, therefore provide bolt of 12 mm diameter.

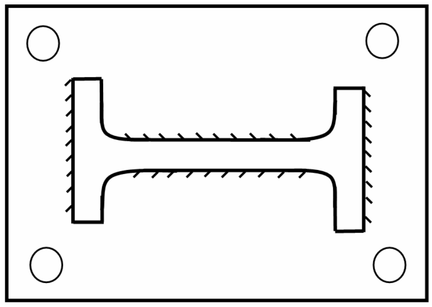


Figure 4.12: Base plate joints

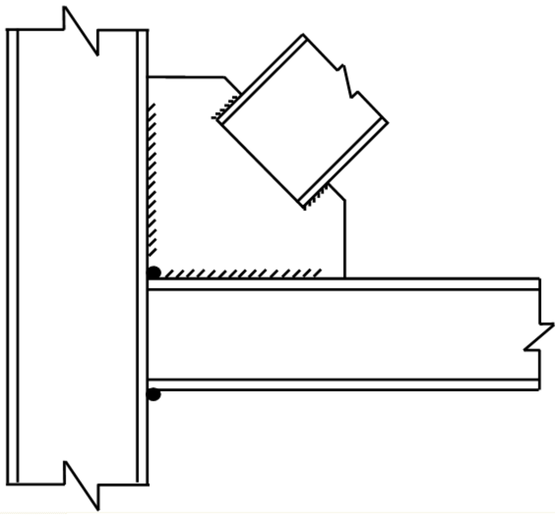


Figure 4.13: Beam column junction details