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SEISMIC RESPONSE OF MULTISTORIED RCC BUILDINGS

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Abstract

A significant number of the structures in this setting have layouts that are erratic from both the elevation and the plan perspectives.

It is possible that terrible earthquakes may occur here in the future. It is essential to determine the ability of both new and existing buildings to endure the effects of a catastrophe by analyzing the performance of the structures. These days, openings in the floors are widespread for various reasons such as stair cases, lighting architecture, and so on. These openings in diaphragms produce strains at discontinue junctions with building parts. Openings in floors are prevalent for numerous reasons.

When discontinuous diaphragms are developed, stress calculations are not performed, and the diaphragms are judged to be appropriate while gap effects are disregarded. In this thesis, an effort is made to differentiate between a building with a diaphragm discontinuity and a building that does not have a diaphragm discontinuity. This is an attempt to know the difference between the two types of buildings.

***Index Terms* - Diaphragm Discontinuity, Structural Detailing, Seismic Analysis**

1. INTRODUCTION

During earthquakes, structures are exposed to severe ground seismic excitation, and it is reasonable to anticipate that they may deform within a range that is inelastic, therefore squandering a significant amount of seismic energy. It is normal practice to equip buildings with shear walls so that they can resist the lateral strains induced by earthquakes. It is necessary to evaluate the real seismic performance of structures with different kinds of shear walls in order to estimate the distribution of stresses and deformations during an earthquake that may occur at the site. This investigation must take place during the earthquake.

Reinforced concrete shear walls, also known as RCSW, have been used in the construction of buildings for a considerable amount of time. The goal is to ensure that the structure can withstand the normal lateral loads imposed on it by external forces such strong winds and earthquakes. The most significant drawback associated with such walls is the fact that they make it possible for fractures to spread in the tension zones and crush in the restricted compression portions, particularly when subjected to increased cyclic loadings. This might result in a loss of strength and stiffness, as well as splitting and ultimately the collapse of the wall. Steel plate shear walls have also been used as lateral load withstanding systems in mid-rise and high-rise structures. It is possible that supplying a steel shear wall may result in substantial drawbacks, such as lower stiffness, shear strength, and energy dissipation capacities. These disadvantages may be caused by buckling in the compression zone.

In order to get around this issue, steel profiles are used to take the place of a significant percentage of the reinforcement in steel-concrete composite shear walls (SCCMSW). Steel profiles that have been encased in concrete have a better strength and deformation capacity, and they prevent steel components from buckling as a result of concrete that has been constricted as a result of lateral ties. An experimental investigation found that composite shear walls collapsed due to bending with compressed concrete crushing followed by local buckling of steel sections. This was the sequence of events that contributed to the failure. The SCCMSW has a broad variety of possible applications; this is mostly due to the fact that it has a high bearing capacity in multistory structures.

Damage caused by earthquakes often begins at points of structural weakness that are present in the lateral load resisting frames of a multi-story framed structure. This behavior of multi-story framed structures during large earthquake movements is dependent on the distribution of mass, stiffness, and strength over both the horizontal and vertical planes of the buildings. Discontinuities in stiffness, strength, or mass along the diaphragm may sometimes be the cause of these deficiencies. However, this is only the case in a small number of instances.

When there are discontinuities like this between diaphragms, the geometry of the frame over the length of the structure typically undergoes radical transformations. Engineering professionals have come to put a lot of faith in structural designers who create

buildings with consistent mass, stiffness, and strength distributions. Less confidence is placed in the design of non-symmetrical or otherwise unconventional structures (diaphragm discontinuities).
Diaphragm discontinuity and its effect on the seismic response of a specific multi-story building has been studied in this thesis.
As part of this research, we are analyzing a multi-story structure in two different configurations: model-1, which has a diaphragm discontinuity; and model-2, which does not have a diaphragm discontinuity. STAAD-Pro was used to model and design the structure of the building; from this model, reinforcing features were developed. In addition, the structure is modelled in SAP2000 using the aforementioned reinforcing details, and then pushover analysis and time history analysis are carried out on the model.

1.1 Diaphragm Discontinuity

As defined by IS-1893:2002, diaphragms with sharp breaks or gradations in stiffness are unacceptable. The effective diaphragm stiffness varies by more than fifty percent from one story to the next, or the diaphragm has cut-out or open portions that are more than fifty percent of the total enclosed diaphragm area.
Diaphragms play an important role in structural engineering and are one kind of structural system. Mainly, it works to transfer in-plane shear stress from lateral loads to shear walls or frames. The most frequent lateral stresses come from wind and seismic pressure. Both stiff and flexible diaphragms are considered to be fundamental kinds. Regardless of the flexibility of the components to which they are transmitting force, flexible diaphragms are able to withstand lateral forces in varying degrees depending on the region they cover. In a construction, rigid diaphragms may transmit load to shear walls or frames, depending on the degree to which they are flexible and where they are located. The pliability of a diaphragm has an effect on the way that lateral forces are distributed to the vertical components of the parts in a structure that are responsible for lateral force resistance.

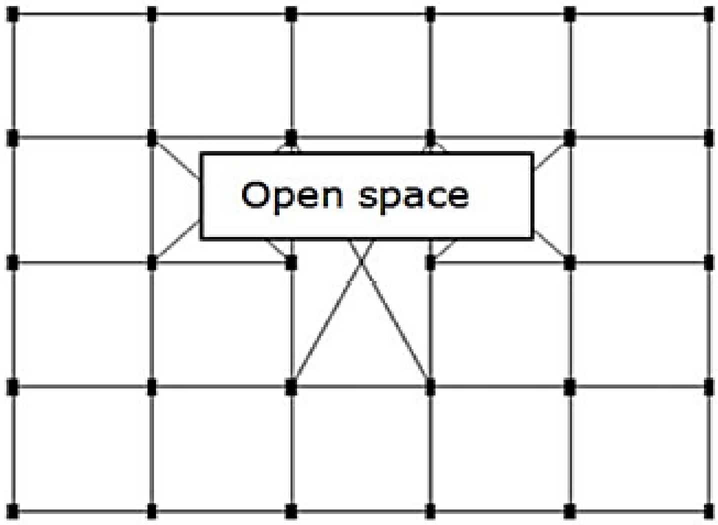

Fig 1.1 Diaphragm Discontinuity in the Frame

1.2 Significance of Study

Diaphragms in buildings should not have anything that is bigger than, say, a pipe with a diameter of 300 millimeters (mm) through them, according to the ideal world of the structural engineer. In addition, diaphragms are planar and have a flat surface across the whole floor plan. However, in the actual world of design, things are very different since the majority of structures need rather significant penetrations for vertical circulation, such as stairways and elevators. This means that the ideal world of architecture is extremely different from the real world of architecture. Building services, such as air ducts and pipes, must also travel through floor slabs, which might result in the formation of possible vulnerabilities in diaphragms.

It compares diaphragms to horizontal beams that resist and transmit horizontal inertia forces to their supports, which in this instance are vertical structural systems such as shear walls or moment frames. In this way, diaphragms resist and transfer horizontal inertia forces to their supports. It shows why structural penetrations are permissible as long as they adhere to the shear force and bending moment diagrams of a diaphragm. It is important to keep in mind that the web of a diaphragm is responsible for resisting shear pressures, whereas the perimeter diaphragm chords, whether working in tension or compression, resist bending moments.
The size of a penetration has the potential to be significant enough to completely compromise the structural integrity of a diaphragm. Take for example a straightforward rectangular diaphragm that acts as a bridging element between two shear walls that move in the y direction. In the event that a slot of the whole width is necessary, what structural choices are available? Because of the slot, the diaphragm can no longer reach all the way to the right wall of the chamber. If the opening is there for the purpose of allowing light or services to pass through the diaphragm, then one of the options available to you is to bridge the gap by inserting a segment of steel bracing. It is possible for it to restore the diaphragm's original spanning capacity, provided that it is constructed and attached with sufficient strength. Alternately, if the aesthetics of the geometry of the diagonal members aren't acceptable, a horizontal Vierendeel frame may be installed to restore the structural function. This frame has much bigger member sizes than the diagonal members. Light and services are able to travel between the structural components in each of these solutions.
If the purpose of the penetration is to build a staircase, then none of the two possibilities presented before are viable alternatives. The diaphragm can no longer transmit forces to the right-hand shear wall as a result of this development. The

only choice available is to stop thinking of that wall as a shear wall and instead construct a new shear wall to the left of the penetration. This is the only feasible solution. Now that the diaphragm has been shrunk, it may cross between the shear walls without any problems. The route taken by the troops has been restored. To finish the design, all that is left to do is stabilize the right-hand wall so that it can withstand stresses in the x direction by linking it back to the newly added Y-X-Y-X.

2. STRUCTURAL DETAILING

An existing building plan is designated for public used as the basis for this research project. For the purposes of the research, this region is considered to be in seismic zone III, despite the fact that it is located in zone II.

Table 2.1 Structural Details

2.1 DESIGN PERIOD

Press Club Building – 100 years.

2.2 BASIC DESIGN CONCEPT FOR RCC STRUCTURES:

PARAMETRE	VALUES
Plan Size	39.200 X 40.200
Location	Press Club Karam Toli Chowk, Ranchi
Building Type	Public Building
Height	20 m (G+5)
Steel Grade	Fe415
Grade of Concrete	M-20
Seismic Zone	Zone III
Column	300 X 600, 350 X 750
Beam	2500 X 400, 250 X 500, 300 X 600
Thickness of Slab	125 mmX150mm
Thickness of Outer Wall	250mm
Thickness of Partition Wall	125mm
Live Load	3 kN/m^2 for slab and 2 kN/m^2 for roof

The building is a R.C framed structure. Brickworks are used as infill wall. Small part of the structure is made of structural steel and it has been designed as isolated from R.C part. Individual members of the frame are being designed for the worst combination of forces such as bending moment, axial force, shear force, torsion, etc. Criticality of erection/maintenance loads are also being checked separately in combination with other simultaneously occurring loads for possible design loadings.

2.3 SUPERSTRUCTURE

Concrete work shall secure a dense, homogeneous, smooth mass including required finishes, possessing required strength and resistance to weathering and abrasion for the structures and foundations. Unless otherwise specified for special type of structures and foundations, minimum grades of concrete to be used has been considered as specified in IS: 456 (latest revision) for different exposure conditions.

2.4 FOUNDATION

Foundations for structures and equipment are considered to be proportioned to resist the worst conditions of loading and generally designed as per the provisions of IS: 1904 and IS: 456.

The depth of foundation is determined based on loading on foundation, safe earing capacity at the founding level, constructional, technological and different functional requirements. The allowable bearing pressure for design of foundation corresponds to values confirmed by results of detailed soil investigation taking into account limits of allowable settlement considered for design of structures.
Safe bearing capacity as recommended in the geotechnical investigation report is considered for design of shallow foundations. However, depending upon the type and intensity of load the foundations may be designed as pile cap supported over piles (depth of pile would vary from 10.0m to 15.0m as per requirement).

2.5 UNDERGROUND WATER RETAINING STRUCTURE

For design of walls of basement, trenches, channels etc. below ground, lateral pressure due to a vertical surcharge loading shall be considered in addition to earth and ground water pressure (if any) etc. In case of heavy wheel loads, lateral surcharge due to actual wheel loads shall be substituted. When a portion or whole of the adjacent soil is below free water surface, computations are done based on submerged weight of soil plus full hydrostatic pressure.
The structures are also checked for stability and factor of safety not be less than those specified under applicable Clause of IS: 456 against overturning and sliding. Proper considerations in design are also to be taken to prevent any possibility of floatation due to upward thrust caused by underground water. Factor of safety against uplift considered at least 1.2 as per IS 3370 (Part-1). Pressure release valves of any form or type that allow ingress of water into the structure are not considered.

2.6 LOADING, LOAD COMBINATIONS AND STRESSES:

LOADS COMING FROM SELFWEIGHT OF STRUCTURE

The following unit weights shall be used and applicable in the design to assess dead loads, i.e. permanent loads due to self-weight as applicable for steel or concrete members.

Table 2.2: Load Combination

Mass concrete	24.00KN/m^3
Reinforced Concrete	25.00KN/m^3
Structural Steel	78.50KN/m^3
Water	10.00KN/m^3
Soil (terrace garden)	18.00KN/m^3

2.7 DEAD LOADS

Dead loads shall include the weight of structure complete with finishes, fixtures and partition and shall be taken as per IS: 875 (Part – I). Any additional permanent / fixed loads will be considered as Superimposed Dead Load (SIDL). The quantum and limitations thereof will be applicable as per relevant Indian Standards.

2.8 IMPOSED/LIVE LOADS

Imposed loads in different areas shall include live, construction, erection, operation and maintenance loads. Equipment loads (which constitute all loads of equipment to be supported on the building frame) are not included in the imposed loads furnished below and shall be considered in addition to imposed loads.

For consideration of imposed loads on structures, IS: 875 (Part – II) “Code of practice for design loads (other than earthquake) for buildings and structures” are to be followed.

If actual expected load is more than the specified minimum load, then actual load will be considered.

2.9 EARTH PRESSURE

Lateral Earth pressure	=	As per Rankine/Coulomb's theory
Surcharge due to vertical load on retaining wall	=	Considered as per IRC : 78-2014, i.e. equivalent to 1.2 m height of earth fill on road side, otherwise 5 KN/m^2.

2.10 WIND LOAD

For design of all structures, the wind loads will be considered as per IS: 875 (Part – 3).

a) The basic wind speed “V_b” = 39 meters / second

b) The risk co-efficient “K_1” = 1.00

c) Terrain Category = Category – 2, Class – B, Height 20M to 50.0M

d) Terrain, Height and Structure Size factor, “K_2” = 1.1

e) Topography Factor, “K_3” = 1.0

2.11 SEISMIC LOAD

Earthquake load is computed as per IS: 1893 taking into consideration soil foundation system, importance factor appropriate to the type of structure, basic horizontal seismic coefficient/ seismic zone factor & average acceleration coefficient as applicable as per relevant clauses of the Standard. Ranchi falls under Zone II.

Importance factor shall be taken as per Table 8 (Clause 7.2.3) of IS: 1893 (Part I) – 2016. The soil foundation system coefficient shall be considered as per relevant clause of IS: 1893 (Part I) - 2016.

Importance Factor	I	1
Zone Factor	Z	0.16
Response reduction factor	R	5

As per IS 1893: 2016, static/dynamic analysis of buildings.

Natural time period is considered as per IS 1893-2016.

2.12 LOAD COMBINATIONS

For the design of individual structural elements, realistic load combinations as per relevant Indian Standards considered. Generally, the following load combinations considered for designing civil &structural work

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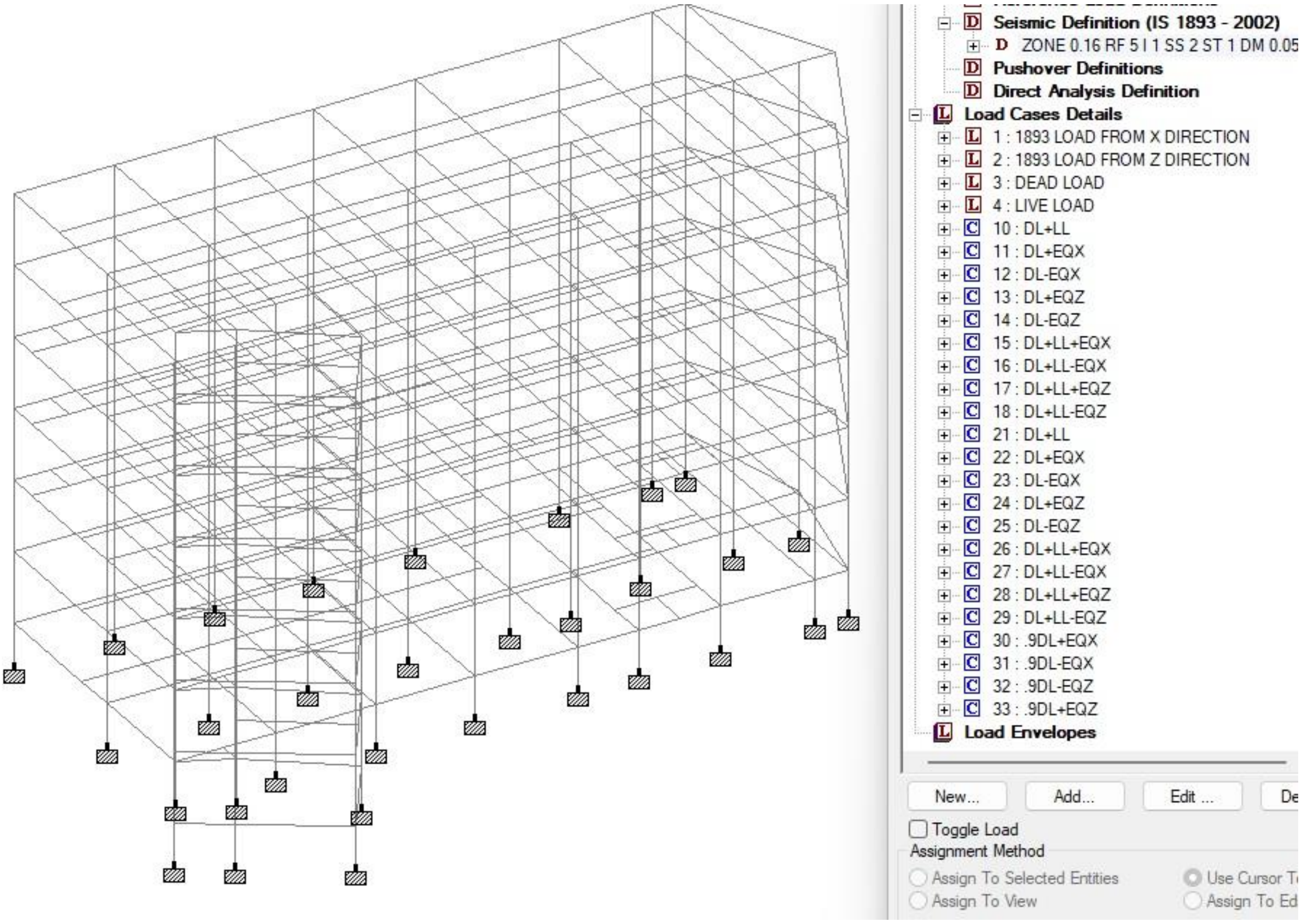

2.13 METHODOLOGY FOR ANALYSIS & DESIGN

Analysis using computer software STAAD Pro (V8i – Select series 6). Analysis results give element and member forces (Bending moments, shear forces, axial forces and displacements).
The structures is regular geometry frames. Separation joint provided at required locations.
Structure is idealized as 3 dimensional structures with linear beam elements using STAAD-Pro (V8i – Select series 6) software. As per IS 1893: 2016, static/dynamic analysis of buildings.
Seismic detailing as per IS: 13920-2016.

2.14 METHODOLOGY OF DESIGN

The structural members are designed for

(i) Strength Criteria

(ii) Serviceability Criteria

2.15 STRENGHT CRITERIA

All members designed by limit state method as per provision of IS 456 – 2000. Columns and beams designed in STAAD Pro(V8i – Select series 5) Software. Slab, stair & foundation designed using Excel Spreadsheet / hand calculation as applicable.

2.16 SERVICEABILITY CRITERIA

(i) Deflection – dimensions of structural members guide by the limiting values of span to depth ratio as per provision of IS: 456-2000.
(ii) Cracking – spacing of reinforcement regulate as per IS: 456-2000 to control flexural cracking.

2.17 POISSON'S RATIO

Poisson's ratio for all concrete: 0.15.

2.18 REINFORCEMENT STEEL

Only thermo-mechanically treated reinforcement bars of grade Fe-500D, conforming to IS: 1786-2008.

Young's Modulus : E = 200,000 MPA

Yield Stress : FY = 500 MPA

Diameters : Normally, 8, 10, 12, 16, 20, 25 & 32 mm dia. Used

2.19 NOMINAL COVER TO REINFORCEMENT:

Pile/Retaining Walls/Water Tank - 50 mm

Pile caps - 75 mm

Footings / Tie Beam - 50 mm

Floor Beams/ RCC Walls - 30 mm

Columns - 40 mm

Slabs - 25 mm

2.20 Minimum Thickness/Depth

(i) Floor slabs including roof slabs, walkways, canopy slabs : 150 mm
(ii) Wall of cables / pipe trenches, underground pits, Pre-Cast trench : 150 mm
(iii) Column footings / Pile caps : 300 mm / 500mm
(iv) Parapets, Chajja : 100 mm
(v) Beams, columns : 250 mm as applicable

2.21 MATERIALS & STANDARDS

Grade of Concrete for Substructure	M30
Grade of Concrete for Superstructure	M30

2.22 LIST OF RELEVANT INDIAN STANDARDS AND CODES

Table 2.4: List of IS code

IS: 456-2000	Code of practice for plain and reinforced concrete
IS: 800-1984/2007	Code of practice for General construction of steel
IS: 1893-2016	Part 1-Criteria for Earthquake Resistant Design of structures
IS 13920 : 2016	Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces – Code of Practice
IS: 875 (Part I to V)	Codes of practice for design Loads
IS: 2911-2010	Part1/Sec 1-Code of Practice for Design and Construction of pile foundations-Driven Cast In-Situ piles
IS: 2911-2010	Part1/Sec 2-Code of Practice for Design and Construction of pile foundations-Bored Cast In-Situ piles
IS SP: 7	National Building Code of India
IS-1080 : 1985	Shallow foundations – Design & construction
IS-2911 : 1985 (Part –4)	Load test on piles
IS-1786 : 2008	High yield strength deformed bar (Grade Fe 500)
IS-2062 : 2006	Structural steel
IS-3370 - Part-II/2009	Code of Practice for concrete structures for the storage of Liquids-General requirements, reinforced concrete structures
IS-2502 : 1963	Code of Practice for Bending & fixing of bars for concrete reinforcement
SP-34 : 1987	Hand book of concrete Reinforcement and detailing
SP: 16	Design aids to IS: 456

2.23 Structural Design

Initially, the structure was modelled and developed in STAAD-Pro, and then the specifics of the reinforcements were taken from that. After that, the building is modelled in SAP2000 using the specifics of the reinforcing materials that were gathered before. The different load configurations are shown down below:

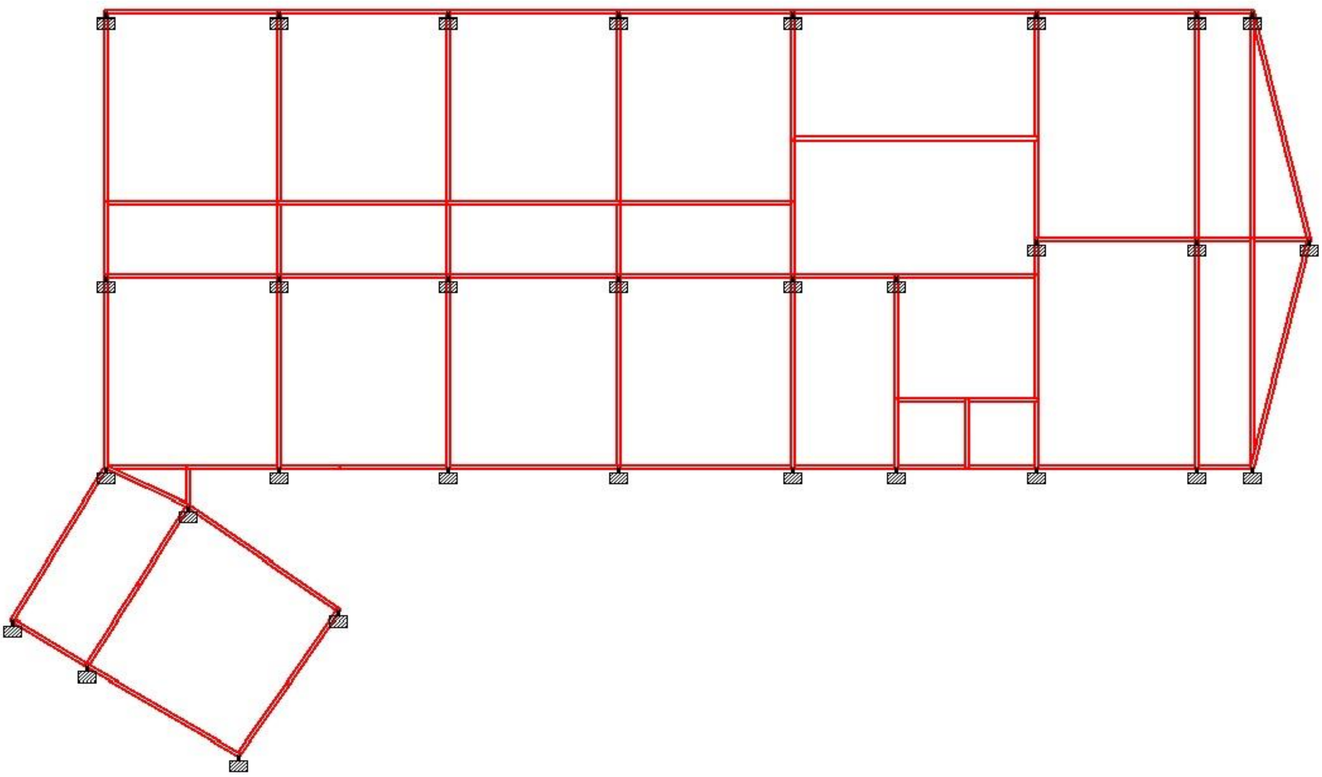

Fig 2.1 Structural Plan

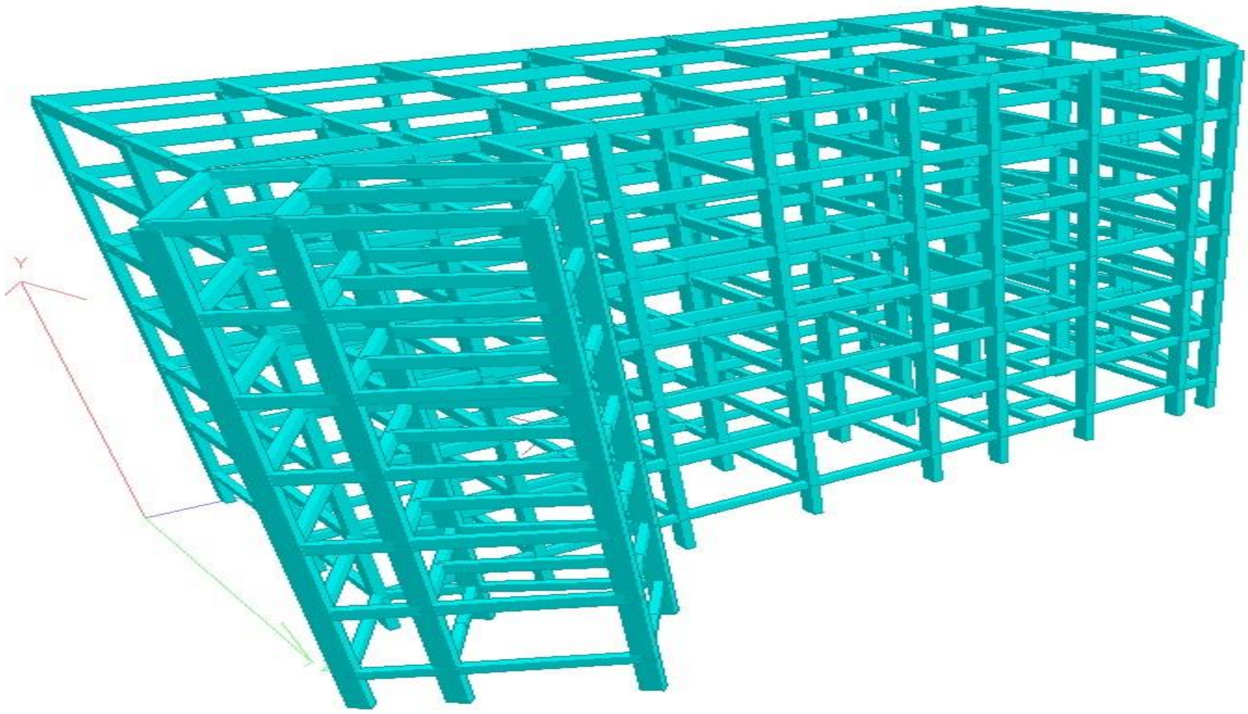

Fig 2.2 Structural Frame Layout

3. SESMIC DETAILING OF RC STRUCTURE

Ductility collapse mechanism Adequate ductility at locations likely to form hinge collapse mechanism need sufficient member ductility to ensure adequate structural ductility. Prevent brittle failure mechanisms to take place prior to ductile yielding.

3.1 Collapse Mechanism

Storey Mechanism Columns require too much ductility Columns are difficult to make ductile Beam – Hinge Mechanism (Sway Mechanism

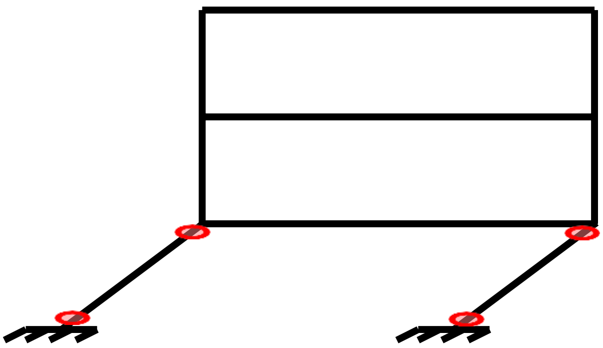

Fig 3.1 Beam Hinge Mechanism (sway mechanism)

Preferred mechanism Ensure that beams yield before columns do Strong Column –Weak Beam Design

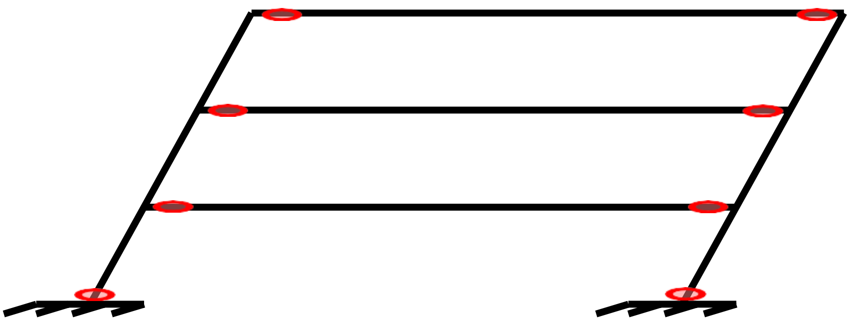

Fig 3.2 Weak Beam Design

3.2 R C Members

Bond Failure: Brittle Shear Failure: Brittle Flexural Failure Brittle: if over- reinforced section (compression failure) Ductile: if under-reinforced section (tension failure) Hence, ensure that Bond failure does not take place. Shear failure does not precede flexural yielding Beam is under-reinforced.

3.3 Failure of RC Section

Yielding of tension bars Ductile Tension failure Under-reinforced section Crushing of compression concrete Brittle Compression failure Over-reinforced section Tension failure more likely if: Less tension reinforcement More compression reinforcement Higher grade of concrete Lower grade of steel Lower value of axial compression

The chain has both ductile and brittle elements. To ensure ductile failure, we must ensure that the ductile link yields before any of the brittle links fails. Assess required strength of chain from code Apply suitable safety factors on load and material Design/detail ductile element(s). Assess upper-bound strength of the ductile element. Design brittle elements for upper-bound load Ensures that brittle elements are elastic when the ductile elements yield.

For instance, in a RC member Shear failure is brittle Flexural failure can be made ductile

Element must yield in flexure and not fail in shear Choose yield mechanism Locate desirable hinge locations Estimate reasonable design seismic force on the building Design the members at hinge locations (upper bound type). Assess the member forces at other locations under the action of "capacity" force Design other locations for that force; need not detail these for high ductility.

CONCLUSION

a. The building's continuous diaphragm contributes to its adaptability. It was discovered that the fundamental period of a structure with a diaphragm discontinuity had a longer duration than an analogous one that had a continuous diaphragm.
b. When constructing with a diaphragm that is continuous, the empirical equation that is provided in design regulations (such as IS 1893:2002) is appropriate. When used to a structure that has a discontinuous diaphragm, this equation may provide very conservative results.
c. The findings of the Modal Analysis reveal the presence of many peculiar modes when the diaphragm discontinuity was modelled. It has been determined, however, that the involvement of the general public in such forms is almost non-existent. As a result, the reaction of the structure will not be greatly altered as a result of these modes.
d. The results of this research suggest that modelling a discontinuous diaphragm may not make a substantial difference in the seismic behaviour of framed buildings.

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