(STC)

BEHAVIOUR OF SOFT STOREY BUILDINGS AGAINST BASE SHEAR

A PROJECT REPORT

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EXTERNAL EXAMINER

ABSTRACT

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One of the observed common reinforced concrete (RC) structural failures during recent earthquakes is column shear failure. When the stiffness and associated strength are abruptly reduced in a structure along the height, earthquake-induced deformations tend to concentrate at the flexible and/or weak storey. The concentration of damage in a storey leads to large deformation in vertical members. The excessive deformation in vertical members often leads to the failure of these members and the collapse of the storey. Soft/weak first stories are especially common in multi-story residential buildings in urban areas, where the first storey often is used for open space, commercial facilities or garages.

For example, structural walls that separate residential units in levels above may be discontinued in the first storey to meet the change in use. The first-story columns during strong earthquake shaking must resist a large base shear, inevitably leading to large storey drift concentrated in that storey. Experimental investigation was planned and conducted to study the influence of brick masonry infill against the lateral loading. In this study, one third scaled frame, with centrally brick infill in the frame along loading direction has been taken for experimental investigation and the available methods of theoretical analysis for the frames has been carried out.

Totally two frames with two columns, along loading direction and one beam with and without infill were constructed in the frame. Until the cracks developed in infill and beam column joints, the contribution of lateral loading is being carried out, the change in lateral stiffness, strength, ductility and of the framed structure due to the presence of in fills and bare frame against lateral loading is investigated. The object of this study was to investigate the behavior of such in filled frame and bared frames against

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Success cannot be spelled with out the having enough experience. In the field of structural engineering, one way or another nature and every one in this universe depends upon other to learn the developing activities.

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CHAPTER - I

CHAPTER -1

INTRODUCTION

1.1 GENERAL

From the engineering points of view, the effects on a building of wind loads and lateral loads arising from seismic excitation become more significant as the height of the building is increased. Under lateral loads, the requirements for lateral stiffness and stability usually dominate over the requirements for vertical load and conventional framed-structures satisfying these requirements would require members both impractical and uneconomical. New systems must therefore be developed in order include the shear wall/core wall system, the tube-in-tube system, and the infilled frame system which is applicable to tall buildings with frame skeleton.

In framed-type structures, walls and partitions are usually considered non-structural and it is a common practice to neglect the effects of infilling walls during structural analysis and design. However, the contributions of the infilling walls to the lateral stiffness and stability of framed structures have long been recognized. Back to the 40's, experiments had been performed to investigate the behavior of infilled frames under lateral loads and the contributions of the in fills to the lateral stiffness and load-carrying capacity of the structures confirmed. Theories were also proposed in the later stages.

The object of the early researches was the possibility of taking into consideration the effects of the existing infilling walls to resist lateral load, which implied that the experimental models had to be conformable with practice and the infills used were usually made of brickwork or precast concrete units not bonded to the frame. This category of infilled frames has the disadvantage of being unstable and highly indeterminate, mainly due to the existence of slip between the frame and the infills when the structure is loaded laterally; and in addition, to the great variation in the properties and behaviour of the brick infills. Experimental evidences also indicated that under lateral load.

Separations existed between the frame and the infills, which opened to an appreciable extent long before the ultimate strength of the structure was reached. To the occupants, the structure is at its ultimate state when the widths of the separations become intolerable, hence although the ultimate strength and the lateral stiffness of the frame is greatly increased by the presence of the infills, this amount of increase can only be considered as a 'reserve' of the structure which cannot be fully relied upon to resist lateral loads.

1.2 INFILLED FRAMES

The idea of utilizing the in filled frame as a structural element was arise recently. To take full advantage of the infills, separations must be limited to tolerable values before the ultimate strength of the structure is reached. In this thesis, test results of one bay single storey with filled and infilled frame models are presented to examine the behaviour of the two categories of multistory infilled frame under lateral static loads. For each category of infilled frames, a theoretical method is also derived and the experimental results are compared with results obtained from theoretical analysis, which are summarized as follows:

- The Equivalent Diagonal Strut method is used to analyse infilled frames without connectors. The infills are replaced by diagonal bracing struts whose equivalent cross-sectional areas are found by a strain energy method. The resulting frame is then analysed by a suitable method of frame analysis.

The merits of the two categories of infilled frames as lateral load resisting elements are compared and conclusions are drawn on the lateral stiffness, strength and failure modes of infilled frames under static lateral loadings.

CHAPTER – II

CHAPTER 2

REVIEW OF PAST RESEARCHES

2.1 EARLY RESEARCHES

In 1948, .**Polyakov** started the first major investigation in Moscow to study the behaviour of infilled frames subjected to lateral loads. Pin-connected steel infilled frames with and without openings were tested and analysed theoretically by an approximate 'theory-of-elasticity' method, using a stress function to express the distribution of stresses around the Boundary. Empirical formulae were derived to predict the deformation and strength of the infills with openings. Polyakov reported that in all the tests, separations between the frame and the infills were observed except at the two compressive corners, which led to his proposal to replace the infills by diagonal bracing struts.

In 1949, a project was initiated at the **Massachusetts Institute of Technology** by the United States Department of the Army to study the atomic blast resistance of infilled frames. Based on the results, **Whitney, Anderson and Cohen** in **1955** presented a paper in which the structure was analysed as a vertical cantilever with flanges and web representing the columns and the infills. This proposal allowed for the inclusion of vertical loads in the analysis.

From 1951, **Benjamin and Williams** continued the project at Stanford. University, California. Large-scale and concrete, concrete block and brickwork. The modes of model infilled frames were tested, with and without openings. Both reinforced concrete and steel frames were used with various types of infills including concrete, reinforced failure they observed were tension in the windward column, shearing of the leeward column, cracking around the perimeter of the infills, and cracking parallel to the compression diagonal of the infill. Theoretical methods proposed by them were either based on simple strength of materials analysis or simply empirical.

In 1953, **Thomas** illustrated the stiffening effect of the infill on the lateral stiffness of frames by the test results of encased rectangular steel frames with various types of weak and strong infill. He found that even with a weak infill, the stiffness of the frame was increased considerably, and pointed out the possibility of allowing for the racking strength of walls and partitions to avoid the necessity for providing special connections or bracing in structural frameworks for resisting lateral forces.

2.2 THE EQUIVALENT DIAGONAL STRUT METHOD

More interest on the topic was used in the 60's. Based on the phenomenon of separation between the frame and the infills, which could usually be observed at very low load, it was pointed out that the effects of the infills on the frame could be replaced by equivalent diagonal bracing struts with appropriate cross-sectional area. This is a macroscopic representation of the system, provided that the cross-sectional area of the equivalent diagonal strut is suitably chosen.

Holmes in 1960 proposed a straight-forward theory of elasticity method to derive the cross-sectional area of the equivalent diagonal strut. He assumed a linear variation of contact stress between the frame and the infill, with a maximum value at the loaded corners and zero intensity at the other corners. After an elastic analysis of the structure, he arrived at the conclusion that the effective width of the infill acting as equivalent diagonal strut was equal to one-third the diagonal length of the infill. The maximum deflection and load was then obtained from the shortening of the equivalent diagonal strut based on the maximum concrete strain at failure. Holmes also reported the results from 13 tests, including several full size models.

K.B.Girish and H.Achyutha have done on experimental and analytical investigation on the response of R.C.C bare frame and non-integral brick masonry infilled frame under lateral reversed cyclic loads and concluded that the infill frame interaction is found to enhance the base shear capacity, improve the hysteretic behaviors and alter the failure mode of the bare frame. And he concludes that large brick size gives more strength and economy.

Yaw-Jeng chiou, et.al Jyh-cherng Tzeng and Yuh-Wehn Liou, et.al (1999) have done an experimental investigation on experimental and analytical study of masonry infilled frames and concluded that the partially infilled masonry wall induces a short column effect and leads to severe failure of the column; on the other hand, the completely filled masonry wall increases the stiffness of the structure.

Shan-Hua Xu and et.al Di-Tao Niu, et.al (2003) have done on seismic behavior of Reinforced concrete braced frame and concluded that, in braced frame structures that combine frame structures and the braces, a high degree of rigidity is secured and an effective energy dissipation mechanism is formed in which braces with elastic-plastic restoring force characteristics can dissipate a greater degree of the energy exerted by earthquake.

Armin B. Mehrabi have done experimental and analytical studies have been carried out to investigate the performance of masonry- infilled RC frames under in-plane lateral loadings and concluded that the finite element models are able to simulate the failure mechanisms exhibited by infilled frames including the crushing and cracking of the concrete frames and masonry panels, and the sliding and separation of the mortar joints.

CHAPTER – III

CHAPTER -3

SEISMIC RESPONSE OF RC FRAME BUILDINGS WITH SOFT FIRST STOREYS

3.1 INTRODUCTION

Many urban multistorey buildings in India today have open first storey as an Unavoidable feature. This is primarily being adopted to accommodate parking or reception lobbies in the first storeys. The upper storeys have brick infilled wall panels. The draft Indian seismic code classifies a soft storey as one whose lateral stiffness is less than 50% of the storey above or below [Draft IS:1893, 1997]. Interestingly, this classification renders most Indian buildings, with no masonry infill walls in the first storey, to be "buildings with soft first storey."

Many earthquakes in the past, e.g., San Fernando 1971, Northridge 1994, Kobe 1995, have demonstrated the potential hazard associated with such buildings. Major damage to many reinforced concrete and steel buildings in the Hyogoken-Nanbu earthquake of January 17, 1995 , and to critical hospital facilities in the San Fernando earthquake of 1971, were attributed to the soft first storey. Alarming amount of damage to the buildings with open basements for parking has been reported during the Northridge earthquake of January 17, 1994 . The recent Jabalpur earthquake of 22 May 1997 also illustrated the handicap of Indian buildings with soft first storey.

Open first storey is a typical feature in the modern multistory constructions in urban India. Such features are highly undesirable in buildings built in seismically active areas; this has been verified in numerous experiences of strong shaking during the past earthquakes. The importance of explicitly recognizing the presence of the open first storey is analyzed. The error involved in modeling such buildings as complete bare frames, neglecting the presence of infills in the upper storeys, is brought out through the study of an experimental works with different models. This thesis argues for immediate measures to prevent the indiscriminate use of soft first storeys in buildings, which are designed without regard to the increased displacement, ductility and force demands in the first storey columns. Alternate measures, involving stiffness balance of the open first storey and the storey above, are proposed to reduce the irregularity introduced by the open first storey.

3.2GROUND ACCELERATION AND BUILDING DAMAGE

Comparatively speaking, the absolute movement of the ground and buildings during an earthquake is not actually all that large, even during a major earthquake. That is, they do not usually undergo displacements that are large relative to the building's own dimensions. So, it is not the distance that a building moves which alone causes damage. Rather, it is because a building is suddenly forced to move very quickly that it suffers damage during an earthquake. Think of someone pulling a rug from beneath you. If they pull it quickly (i.e., accelerate it a great deal), then they needn't pull it very far to throw you off balance.

On the other hand, if they pull the rug slowly and only gradually increase the speed of the rug, they can move (displace) it a great distance without that same unfortunate result .In other words, the damage that a building suffers primarily depends not upon its displacement, but upon acceleration. Whereas displacement is the actual distance the ground and the building may move during an earthquake, acceleration is a measure of how quickly they change speed as they move. During an earthquake, the speed at which both the ground and building are moving will reach some maximum. The more quickly they reach this maximum, the greater their acceleration. It's worthwhile mentioning here that in order to study the earthquake responses of buildings; many buildings in earthquake-prone regions of the world have been equipped with strong motion accelerometers. These are special instruments which are capable of recording the accelerations of either the ground or building, depending upon their placement.

The recording of the motion itself is known as an accelerogram. shows an accelerogram recorded in a hospital building parking lot during the Northridge, California earthquake of January 17, 1994.In addition to providing valuable information about the characteristics of the particular earthquake recorded or the building where the accelerogram was recorded, accelerograms recorded in the past are also often used in the earthquake response analysis and earthquake design of buildings yet to be constructed.

3.3 NEWTON'S LAW

Acceleration has this important influence on damage, because, as an object in movement, the building obeys Newton' famous Second Law of Dynamics. The simplest form of the equation which expresses the Second Law of Motion is

F = MA.

This states the ***F***orce acting on the building is equal to the ***M***ass of the building times the ***A***cceleration. So, as the acceleration of the ground, and in turn, of the building, increase, so does the force which affects the building, since the mass of the building doesn't change. Of course, the greater the force affecting a building, the more damage it will suffer; decreasing ***F*** is an important goal of earthquake resistant design. When designing a new building, for example, it is desirable to make it as light as possible, which means, of course, that ***M***, and in turn, ***F*** will be lessened.

3.4 INERTIAL FORCES

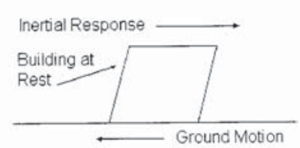


Figure 1: Acceleration, Inertial Forces

It is important to note that ***F*** is actually what's known as an **inertial** force, that is, the force is created by the building's tendency to remain at rest, and in its original position, even though the ground beneath it is moving. This is in accordance with another important physical law known as **D'Alembert's Principle,** which states that a mass acted upon by an acceleration tends to oppose that acceleration in an opposite direction and proportionally to the magnitude of the acceleration .

This inertial force ***F*** imposes **strains** upon the building's structural elements. These structural elements primarily include the building's beams, columns, load-bearing walls, floors, as well as the connecting elements that tie these various structural elements together. If these strains are large enough, the building's structural elements suffer damage of various kinds.

To illustrate the process of inertia generated strains within a structure, we can consider the simplest kind of structure imaginable--a simple, perfectly rigid block of stone. (Figure 2.) During an earthquake, if this block is simply sitting on the ground without any attachment to it, the block will move freely in a direction opposite to that of the ground motion, and with a force proportional to the mass and acceleration of the block.

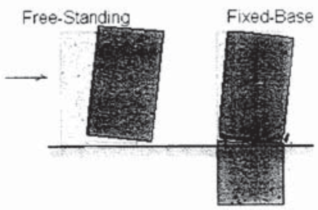


fig 2 : stability of structures

If the same block, however, is solidly founded in the ground and no longer able to move freely, it must in some way absorb the inertial force ***internally.*** In Figure 2, this internal uptake of force is shown to result in cracking near the base of the block.

Of course, real buildings do not respond as simply as described above. There are a number of important characteristics common to all buildings which further affect and complicate a building's response in terms of the accelerations it undergoes, and the deformations and damages that it suffers.

3.5 BUILDING FREQUENCY AND PERIOD

The magnitude of the building response – that is, the accelerations which it undergoes – depends primarily upon the frequencies of the input ground motion and the building's natural frequency. When these are near or equal to one another, the building's response reaches a peak level.

In some circumstances, this **dynamic amplification** effect can increase the building acceleration to a value two times or more that of the ground acceleration at the base of the building. Generally, buildings with higher natural frequencies, and a short natural period, tend to suffer higher accelerations but smaller displacement. In the case of buildings with lower natural frequencies, and a long natural period, this is reversed as the buildings will experience lower accelerations but larger displacements.

3.6 BUILDING STIFFNESS

The taller a building, the longer its natural period tends to be. But the height of a building is also related to another important structural characteristic: the building flexibility. Taller buildings tend to be more flexible than short buildings. (Only consider a thin metal rod. If it is very short, it is difficulty to bend it in your hand. If the rod is somewhat longer, and of the same diameter, it becomes much easier to bend. Buildings behave similarly.) We say that a short building is stiff, while a taller building is flexible. (Obviously, flexibility and stiffness are really just the two sides of the same coin. If something is stiff, it isn't flexible and vice-versa.)

Stiffness greatly affects the building's uptake of earthquake generated force. Reconsider our first example above, of the rigid stone block deeply founded in the soil. The rigid block of stone is very stiff; as a result it responds in a simple, dramatic manner. Real buildings, of course, are more inherently flexible, being composed of many different parts.

Furthermore, not only is the block stiff, it is brittle; and because of this, it cracks during the earthquake. This leads us to the next important structural characteristic affecting a building's earthquake response and performance— ductility.

3.7 DUCTILITY

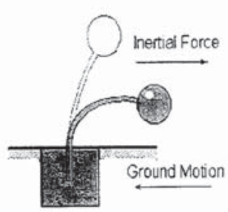


Figure 3: Metal Rod Ductility

Ductility is the ability to undergo distortion or deformation – bending, for example – without resulting in complete breakage or failure. To take once again the example of the rigid block in Figure 3 the block is an example of a structure with extremely low ductility. To see how ductility can improve a building's performance during an earthquake, consider Figure 3 for the block, we have substituted a combination of a metal rod and a weight. In response to the ground motion, the rod bends but does not break. (Of course, metals in general are more ductile than materials such as stone, brick and concrete.) Obviously, it is far more desirable for a building to sustain a limited amount of deformation than for it to suffer a complete breakage failure.

The ductility of a structure is in fact one of the most important factors affecting its earthquake performance. One of the primary tasks of an engineer designing a building to be earthquake resistant is to ensure that the building will possess enough ductility to withstand the size and types of earthquakes it is likely to experience during its lifetime.

3.8 THE BASIC PRINCIPLES OF EARTHQUAKE RESISTANT DESIGN

Earthquake forces are generated by the dynamic response of the building to earthquake induced ground motion. This makes earthquake actions fundamentally different from any other imposed loads. Thus the earthquake forces imposed are directly influenced by the dynamic inelastic characteristics of the structure itself. While this is a complication, it provides an opportunity for the designer to heavily influence the earthquake forces imposed on the building. Through the careful selection of appropriate, well distributed lateral load resisting systems, and by ensuring the building is reasonably regular in both plan and elevation, the influence of many second order effects, such as torsional effects, can be minimised and significant simplifications can be made to model the dynamic building response.

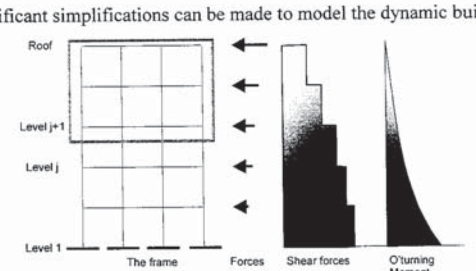


Figure 4: Loading Pattern and Resulting Internal Structural Actions

Most buildings can be reasonably considered as behaving as a laterally loaded vertical cantilever. The inertia generated earthquake forces are generally considered to act as lumped masses at each floor (or level). The magnitudes of these earthquake forces are usually assessed as being the product of seismic mass (dead load plus long-term live load) present at each level and the seismic acceleration generated at that level. The design process involves ensuring that the resistance provided at each level is sufficient to reliably sustain the sum of the lateral shear forces generated above that level.

3.9 THE 'CONVENTIONAL' EARTHQUAKE DESIGN PROCEDURE

The conventional engineering design approach is to use the actions for members derived from the above elastic analysis as the basis for determining the dimensions and structural capacity. Significant changes in dimension will affect the building stiffness and may require re-analysis. The resulting sizes are then checked against those assumed during the analysis and provided a reasonable match is attained, the design verification process is considered complete. Earthquake design has three important distinctions from other loadings. Firstly there is the acceptance that damage to both non-structural and some structural elements will occur, but collapse is to be avoided (refer to section **Error! Reference source not found.**). Secondly earthquakes are highly variable dynamic events which designers tend to simplify into a set of quasi-static lateral loads. This approach enables relatively simple analysis and design, but noticeably departs from reality. It is therefore important to build into the structure a degree of toughness or robustness which will avoid the development of undesirable collapse mechanisms. Thirdly, although there is geological and seismological understanding of how earthquakes are initiated and how the energy release mechanisms translate into surface ground motion, earthquakes still inherently contain a higher level of uncertainty than do other forms of loading.

3.10 THE IMPORTANCE & IMPLICATIONS OF STRUCTURAL REGULARITY

Most Standards outline certain provisions relating to both the vertical regularity of the structure and also the plan regularity. These usually apply to the appropriateness of several assumptions implicit in the distribution pattern of the loading or the torsional effects

3.11VERTICAL REGULARITY

Ideally the capacity of the structure should follow the shear and bending moment pattern of the structure shown in Figure . Substantial departures from this ideal typically result in the onset of premature post-elastic deformations often concentrated at over one level. When this occurs, elements within the one level degrade, attract additional (post-elastic) deformation and a soft-storey mechanism develops with collapse often being the inevitable result. The vertical regularity check is intended to avoid abrupt changes in overall strength or stiffness at any particular level. Where such provisions are not met, then a more detailed analysis will be required to ensure that post-elastic deformation capacity at each level can be met without unacceptable loss of strength or post-elastic deformation demands in excess of their capacity. It is wise to avoid abrupt curtailment of reinforcing steel at one level of a reinforced concrete frame or substantive changes in a column section. It is better to introduce such changes gradually, over several floors, thereby allowing a smooth transition between sections to develop. Obviously it is undesirable to curtail shear walls above their base as this also induces a very real potential for soft-storey development.

3.12 HORIZONTAL REGULARITY.

The random, three dimensional motions generated by earthquakes is usually simplified into two transverse orthogonal components with the vertical response typically being ignored. The transverse dynamic response may also introduce twisting and torsional effects into the response, either directly as a function of the input ground motion, or because of variations in the spatial distribution of seismic mass, or because of the structure being irregular in plan.

Measures employed to counter these effects typically involve distributing the lateral load resisting systems about the building plan and attempting to limit the plan profile to being reasonably regular and compact. Most modern earthquake loading standards require the designer to assess the Centre of Rigidity (CoR) of the structural system, and the centre of mass (CoM) of the uniformly distributed seismic mass. The eccentricity is typically increased by 10% of the building width to allow for unexpected variations in torsional effects with the magnitude of the resulting torsional action (being the product of mass and linear eccentricity between CoR and CoM) increasing accordingly. Such approximations tend to be based upon the response of the structure within the elastic response domain and may provide little security against collapse once the deformations have progressed into the inelastic domain. Paulay has recently proposed an elegant means of directly addressing post-elastic torsional effects. He postulates that the post elastic torsional demand can be met, satisfied and indeed controlled by rigorous detailing of lateral load resisting elements so as to ensure their displacement ductility demands are met. Provided this is achieved the effects of torsion are readily accommodated.

The preferred method of minimising torsional effects is to select floor plans which are regular and reasonably compact. Wide separation of horizontal lateral load resisting systems is encouraged. Plan forms with re-entrant corners such as 'L' and 'T' plan layouts should be avoided, or, where these plan forms are dictated by other constraints, seismic separation joints should be introduced between rectangular blocks. Such joints must be designed to accept the post-elastic dynamic response of the building parts, which may be responding with disparate phases. Contact and hammering between blocks is to be avoided.

CHAPTER – IV

CHAPTER 4

SOFT STOREY

The *soft storey* concept is very dangerous in earthquakes. A soft storey may be conveniently defined as one where the stiffness is less than 70% of the storey above it. This commonly occurs in multi-storey offices and hotels due to the desire for higher ceilings and more open spaces on the ground floor. Several design strategies are available for dealing with this situation.

4.1 EFFECTIVE EARTHQUAKE RESISTANCE MATERIALS:

Desirable features of structural materials for earthquake resistance are:

- high ductility
- high strength-to-weight ratio
- homogeneity

4.2 SOFT-STOREY BUILDINGS:

An open-ground-storey building, having only columns in the ground storey and both partition walls and columns in the upper storey, has two distinct characteristics:

- It is relatively flexible in the ground storey, i.e., the relative horizontal displacement it undergoes in the ground storey is much larger than what each of the storey above it does. This flexible ground storey is also called soft storey.
- It is relatively weak in ground storey, i.e., the total horizontal earthquake force it can carry in the ground storey is significantly smaller than what each of the storey above it can carry. Thus, the open ground storey may also be a weak storey.

Often, open-ground-storey buildings are called soft-storey buildings, even though their ground storey may be soft and weak. Generally, the soft or weak storey usually exists at the ground storey level, but it could be at any other storey level too.

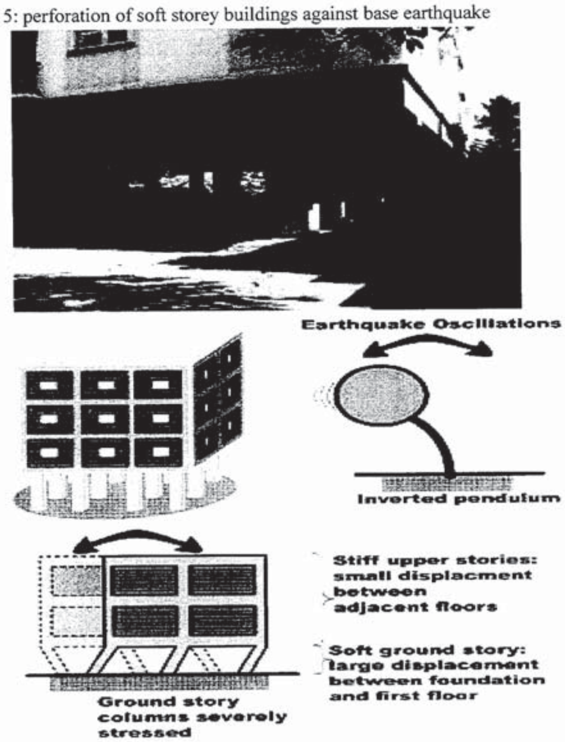
4.3 BEHAVIOR

Open-ground-storey buildings have consistently performed poorly during earthquakes across the world, a large number of buildings with open ground storey have been built in India in recent years. Further, a large number of similarly designed and constructed buildings exist in the various towns and cities situated in moderate to severe seismic zones (namely III, IV and V) of the country.

The presence of walls in upper storey makes them much stiffer than the open ground storey. Thus, the upper storey move almost together as a single block and most of the horizontal displacement of the building occurs in the soft ground storey itself. In common language, this type of building can be explained as one on chopsticks. Thus, such buildings swing back-and-forth like inverted pendulum during earthquake shaking, and the columns in the open ground storey are severely

strength to resist these high stresses they do not have adequate ductility, they may be severely damaged, which may even lead to their collapse of the building.

Fig 5: perforation of soft storey buildings against base earthquake



4.4 EFFECT OF SOFT STOREY BUILDINGS EARTHQUAKE

A typical RC building is made of horizontal members (beams and Slabs) and vertical members (columns and walls), and supported by foundations that rest on ground .The system comprising of RC columns and connecting beams is called a RC Frame. The RC frame participates in resisting the earthquake forces. Earthquake shaking generates inertia forces in the building. Which are proportional to the building mass. Since most, of the building mass is present at floor levels, earthquake-Induced inertia forces primarily develop at the floor levels. These forces travel downwards – through, slab and beams to columns and walls, and then to the foundations from where they are dispersed to the ground as inertia forces accumulate downwards from the top of the building, the columns and walls at lower storeys experience higher earthquake-induced forces (figure 1) and are therefore designed to be stronger than those in storey above.

4.5 ROLES OF FLOOR SLABS AND MASONRY WALLS

Floor slabs are horizontal plate-like elements, which facilitate functional use of buildings. Usually, beams and slabs at one storey level are cast together. In residential multi-store buildings, thickness of slabs is only about 110-150mm when beams bend in the vertical direction during earthquakes, these thin slabs bend along with them. And, when beams move with columns in the horizontal direction, the slab usually forces the beams to move together with it.

In most buildings, the geometric distortion of the slab is negligible in the horizontal plane; this behaviour is known as the rigid diaphragm action.

After columns and floors in a RC building are cast and the concrete hardens, vertical spaces between columns and floors are usually fined-in with masonry wails to demarcate a floor area into functional spaces (rooms). Normally, these masonry, walls, also called infills walls, are not connected to surrounding RC columns and beams when column receive horizontal forces at floor levels, they try to move in the horizontal direction, but masonry walls tend to resist this movement. Due to their heavy weight and thickness, these walls attract rather large horizontal forces. However since masonry is a brittle material, these walls develop cracks once their ability to carry horizontal load is exceeded, thus infills walls act like sacrificial fuses in building. They develop cracks under severe ground shaking but help share the load of the beams and column until cracking .earthquake performance of infill wall is enhanced by mortars of good strength making proper masonry courses, and proper packing of gaps between RC frame and masonry infill walls. However, an infill wall that is unduly tall or long incomparsion to its thickness can fall out-of-plane (i.e., along its thin direction) , which can be life threatening . Also, placing infills irregularly in the building causes ill effects like short column effects and torsion.

CHAPTER – V

CHAPTER 5
EXPERIMENTAL INVESTIGATION

5.1 INTRODUCTION

Model testing technique was employed in the Experimental investigation to examine the actual behaviour of infilled frames under lateral static load. Frame models were fabricated, and tested to rupture. The models consisted of two categories of frames with and without infills

5.2 PROPERTIES OF THE MODELS

The models were grouped into two categories.

The first category which consists of scaled model with out any infills, and another model with infills which are capable of resisting the lateral forces. The models comprised of the scaled sizes with respect to prototype with the scale of 1: 3.The storey heights were one meter for all the models.

5.3. DESIGN OF THE MODELS

5.3.1 THE FRAME

The models were tested as vertical cantilevers. To provide full end fixity at the foundation, base slab is made to bolted to the loading frame slab, the frame is modeled for the ratio of 1: 3, uniform dimensions are maintained through out the frame.

5.3.2 THE INFILLS

The infills were 100 mm in thickness, In order to study the behavior of the in filled frame up to rupture, it was necessary to use brick as the infilling material. Mortar of 1: 5 is prepared to act as a bonding agent in between the bricks. Care is taken to have good bonding between the frames and brick along the corners.

5.3.3 REINFORCEMENTS

It is not the object of the present research to look into the detailing of the reinforcements in the frames and infills. However, secondary reinforcements including temperature and shrinkage reinforcements are always necessary for concrete members. Care has been taken to ensure that the placement of reinforcement is uniform in both the frames.

5.3.4 MATERIAL PROPERTIES

The concrete strength was equivalent to average 28-day cube strength of 20mpa. in laboratory tests where accurate control of materials and test conditions was possible. Since the aggregates were scaled down, trial mixes were tested to obtain mixes that attained the required strength with good compaction and workability. At the age of 28 days, the model was ready for test.

5.3.5 REINFORCEMENT DETAILS OF R.C.FRAME

For two columns of both frames, 4 nos. of 6mm diameter HYSD bars were provided. The reinforcements were curtailed in storeys due to reduction in axial force and bending moments.. The two legged stirrups of 6 mm diameter bars were used with the spacing of 80 mm c/c near supports and 100 mm c/c in the middle portion. For the same storey level beams, 4 nos. of 6 mm diameter bars were provided. The stirrups of 6 mm diameter bars were provided with 80 mm c/c near ends and 100 mm c/c in the middle portion.

5.3.6 LOADING ARRANGEMENTS

A horizontal loading system was adopted to eliminate the effects of the self-weight of the models. The model was hence tested as a vertical cantilever. In order that the model could be in a vertical plane, adjustments were made using a plumb line in the setting up of the model.

Load was applied laterally through 10-ton hydraulic jacks under a central oil pressure and distributed into equal point loads.

5.3.7 INSTRUMENTATION

To perform the effective usage of the loading system the following equipment are used to calibrate the results

1. **Load cell** – load cells are employed to transmit the load from the jack to the model, and to have an effective calibration of load values.

2. **LVDT** – linearly varying deflectometer are employed at the salient locations to ensure the deflection at the specified locations.

3. **DEMEC**- Demountable mechanical strain gauges are fixed at the specified location to ensure the occurrence of strains.

4. **Hydralic jack** – mechanically operated hydralic jack are used to transmit the load to the structure, these jacks are connected to the load cell, to calibrate accurate transmission of load.

The frame is subjected to equivalent static lateral cyclic loading. The load is applied in Increments of 5 KN base shear for each cycle till the ultimate load is reached. The deflections at all storey levels are measured at each increment of the load. The strain in concrete and infill are monitored at maximum load of each cycle and at unloading conditions of frame (i.e. when the load is released fully) during all cycles of loading. The formation and propagation of cracks, hinge formation and failure pattern are recorded. The deflectometer readings for calculating error due to rigid body rotation of foundation block are also recorded.

The complete test took 1-2 hours.

CHAPTER VI

CHAPTER -6
THEORETICAL ANALYSIS

6.1 THE EQUIVALENT DIAGONAL STRUT METHOD

6.1.1 INTRODUCTION

When an infilled frame without connectors is subjected to lateral load, a large portion of the load is transmitted to the infills through the joints of the enclosing frame. The effects of the infills are similar to the action of diagonal struts bracing the enclosing frame. This analogy is further justified by the phenomenon of slip and separation between the frame and the infills, which can always be detected in infilled frames without connectors at a certain stage of the loading range; contact between the frame and the infills are then achieved through only a certain portion of the sides of the infills known as the length contact α , It is hence proposed that an analogous model representing actual structure can be established in which the effects of the infills are replaced by diagonal bracing struts whose cross-sectional areas are to be found by some other means. Various methods had been proposed by investigators to calculate the cross-sectional area of the equivalent diagonal strut, the most rational method being due to Smith who based the calculations on the actual length of contact, assuming both triangular and parabolic contact stress distribution However, the results obtained from this analysis were much greater than experimental values and this discrepancy was mainly attributed to the assumed contact stress distribution, which was though-to be more concentrated at the compression corners than either the assumed triangular or parabolic distribution.

For an infilled frame with λH_f equal to 3.0, the length of contact is equal to 50 % of the storey height. If a triangular contact stress distribution is

0.15H from the compression corners, revealing their mutual proximity. As accounted for in the preceding paragraph, this value should be reduced, which implies that the resultant of the contact stresses is closer to the compression corners. Generally, λH_f is greater than 3.0 and the resultant of the contact stresses is very close to the compression corners. A good approximation can therefore be made by assuming the contact forces to be represented by point loads acting at the compression corners. This assumption is used in deriving the cross-sectional area of the equivalent diagonal strut in the following.

6.1.2 EFFECTIVE WIDTH OF THE EQUIVALENT DIAGONAL STRUCT

A strain energy method is used to find the cross-sectional area of the equalent diagonal struct. The following assumptions are made:

1. The material of the infills is homogeneous, isotropic and elastic.
2. The infills are not bonded intentionally to the frames by any connectors.
3. Full separation between the frame and the infi l ls can be achieved except at the compression corners.
4. The contact forces are concentrated at the two compression corners of each infill.

The following analysis gives the cross-sectional area of the equalent diagonal strut after separation has occurred, which should be used in all analyses for infilled frames because the greater stiffness available before slip and separation occur is unreliable.

The spans of the equivalent beam and column are given by:

$$L_1 = 1/2.\,(B+b) + C_1$$

$$L_2 = h + C_2$$

Where C_1 and C_2 are corrections to account for the local deformations due to local stress concentrated at the joint, and are dependent on the dimensions of depth of the beam was suitable in general. In order to investigate the effects of this local deformation, values for C_1 equal to 0, 1/8, 1/4, and ½ of the beam depth have been used in the computations and the results discussed earlier.

the length L_2 as calculated from Equation .usually exceeds the distance from the bottom of the wall to the Controidal axis of the beam. In this circumstance, the latter value should be used instead. The strain energy of the infill due to bending deformation is given by:

$$U_b = \tfrac{1}{2} EI_1 \int_0^{L1} p^2 (x.\sin\theta - h\,\tfrac{1}{2}.\cos\theta)^2\,dx$$

$$+ \tfrac{1}{2} EI_2 \int_0^{L2} p^2 (x.\cos\theta - b_2/2.\cos\theta)^{2\,dx}$$

While the strain energy due to shear deformation is

$$U_s = \frac{(p.\sin\theta)2L_1h^2_1}{20G_iI_1} + \frac{(p.\cos\theta)^2L_2b^2_2}{20G_iI_2}$$

and the strain energy due to axial deformation is

$$U_p = \frac{(p.\cos\theta)^2L_1}{20E_iA_1} + \frac{(p.\sin\theta)^2L_2}{20E_iA_2}$$

The total strain energy stored in the infill is thus

$$U = U_b + U_s + U_p$$

Minimizing the strain energy yields the deflection in the direction of the load, i.e. the diagonal deflection:-

$$\delta_d = P/3E_i\,\{\sin^2\theta/I_1\,[(L_1 - m_1)^3 + m^3_1] +$$

$$\cos^2\theta/I_2\,[(L_2 - m_2)^3 + m^3_2]\}$$

$$+ \frac{1.2E_i}{G_1}\,[\frac{L_1\,\sin^2\theta}{A_1} + \frac{L_2\,\cos^2\theta}{A_2}]$$

$$+ [L_1\,\cos^2\theta + L_2\,\sin^2\theta]$$

Where

$M_1 = h_1/2.\cot\theta$

$M_2 = b_1/2.\tan\theta$

The diagonal stiffness of the infill is equal to the reciprocal of the diagonal deflection when p=1, i.e.

$$k_d = 1/\delta_d$$

If the infill is replaced by an. equivalent diagonal strut of length L_d, the stiffness of the strut is given by:-

$$K_d = \frac{E_i A_e}{L_d}$$

Comparing the above Equations, the effective cross sectional area of the equivalent diagonal strut is obtained as:

$$A_e = \frac{L_d}{E_i\,\delta_d}$$

6.1.3 STIFFNESS OF INFILLED FRAMES WITHOUT CONNECTORS

In order to calculate the stiffness of the structure, The infills are replacec by diagonal bracing struts whose Cross-sectional areas are given by abov equation. This analogous frame as illustrated can be analysed by the stiffnes method of structural analysis indicated to give the deflections at the require levels, from which the lateral stiffness of the structure can be computed.

6.1.4 STRENGTH OF INFILLED FRAMES WITHOUT CONNECTORS

An estimate on the ultimate strength of the structure can be achieved with the following assumptions:

1. Premature failure of the bare frame by joint- opening will not occur.
2. Similar to the assumption made in Art.4.1.4, tension cracking of the infill will not result in collapse of the structure.
3. The ultimate strength of the structure is governed by the two possible modes of failure:

a. Shear failure of the infill, usually at the lintel beam in the case of infills with a high opening.

b. Compression failure of the infill, usually at the compression corners where the compressive stress is greatest.

With the above assumptions, the end actions on the most severely stressed infill - usually the lowest infill - are calculated according to Equation A.1.4 and the ultimate strength of the infill computed based on the following

Criteria:

1. When the opening is below the compression diagonal, the force in the infill is mainly compressive, which is similar in nature to the force in the equivalent diagonal strut. Thus the equivalent diagonal strut can replace the infill in Strength calculations; and the diagonal compression will be divided by A to give the average compressive stress in the equivalent diagonal strut.

If the ultimate compressive Strength of the infilling material is known, the ultimate Strength of the structure can be obtained by proportion.

2. When there is an opening which extends above the compression diagonal, the infill is under bending shear and comparison. The action of the infill is hence difference in nature from the action of the equivalent diagonal

infill. It is then necessary to analyse the strength of the actual infill subjected to the calculated end forces. With the application or assumption 2, the lintel beam will not fail under bending, although tension cracks will develop. However, the shear resistance of the lintel beam has to be investigated; where the lintel beam is shallow, its shear resistance is usually the controlling factor for the ultimate strength of the structure.

CHAPTER VII

CHAPTER -7
ANALYTICAL STUDY

7.1 INTRODUCTION

Reinforced concrete framed buildings with infill walls are usually analyzed and designed as bare frame, without considering the strength and stiffness contributions of the infill. However, during earthquake, these infill walls contribute to the response of the structure and the behavior of infilled framed buildings is different from that predicted for bare frame structures. The present study aims to evaluate the response of framed reinforced concrete buildings with bricks as infill.

7.2 OBJECTIVES

The objective of present work is to analyse the behaviour of quarter size, two bay, five storey RC frame with and without infills under lateral load with respect to their load carrying capacity, load deflection behaviour, and failure modes by using finite element analysis software ANSYS. These characteristics are essential for the design and analysis of seismic resistant structures.

7.3 SCOPE OF THE STUDY

The development in model material and model testing technique hav established the reliability of model analysis and testing as a design tool. It i with in view, the current investigation to study the influence of infills in th lateral load resistance of building frames is proposed on the lines of mode design and testing. The model proposed is a 1:3 scale model of a prototype. Th dimensions of the model are given in the figure. The salient aspects wit regards to material, reinforcement and bonding similitude have been strictl

7.4 FINITE ELEMENT MODELING- USING ANSYS SOFTWARE

There are several methods are available for analysis the RC infills frames, the methods are fail to model the stiffening action between the frame and infills and therefore fail to model the interactive forces in the members. Majority of the finite element studies have been carried out to study the elastic behavior. The studies on RC frame with infills have been lacking and there is need to study the monotonic and hysteretic behaviour under lateral loads.

The analytical investigation adopted in the paper consists of finite element analysis, treating the infilled frame subjected to in plane loads as plane stress problem. The frame and the infills are idealized using solid in 3D non-linear finite element analysis based on the following assumptions

- The concrete material is assumed to be initially isotropic
- The infill material is isotropic
- The reinforcement is assumed to be "smeared" through out the element
- The tensile strength of brick masonry and concrete are taken to be 0.1 times their corresponding compressive strengths.

The frame is modeled in 3D using **eight noded** solid element. T reinforced cement concrete member of the frame namely, beams and colum have been modeled using SOLID 65 element which has 3DOF'S at each no available in the element library of ANSYS software. The above elements ha as shown in fig.5 masonry have been modeled using SOLID 45 element which has 3DOF'S at each available in the element library of ANSYS software. Link element with 2 DOF at each node was used to model the behaviour at the infilled frame interface.

Non-linear material properties have been assigned to the elements. Additional concrete material data, such as transfer coefficient, tensile stresses, and compressive stresses are input in the data table typical shear transfer coefficient range from 0.0 to 1.0, with 0.0 representing a smooth crack and 1.0 representing a rough crack (no loss of shear transfer). This specification may be made for both the closed and open crack. In the present analysis the shear transfer coefficient for an open crack is assumed as 0.3 and that for closed crack is assumed as 0.7 if cracking occurs at an integration point, the cracking is modeled through an adjustment of material properties, which effectively treats the cracking as a "smeared band" of cracks, rather than discrete cracks.

7.5 PROPERTIES OF ELEMENT

SOLID65 is used for the 3D modeling of solids with or without reinforcing bars (rebars). The solid is capable of cracking in tension and crushing in compression with the following assumption and restrictions:

- The element must have eight nodes
- The element is nonlinear and requires an iterative solution
- The beam must not have a zero length, area, or moment of inertia
- The following two options are not recommended if cracking or crushing nonlinearities are present
- Large strain and large deflection. Results may not converge or may be incorrect, especially if significantly large rotation is involved.

SOLID45 is used for the 3-D modeling of solid structures. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities.

7.6 FALIURE MODES

The possible failure modes as shown in figures 6 to 9 are considered in the analysis.

1) Frame members

- Cracking in tension
- Compressive failure
- Yielding of reinforcement

2) Infills

- Crushing of strut
- Diagonal cracking
- Shear bond failure
- Tensile failure of tie member

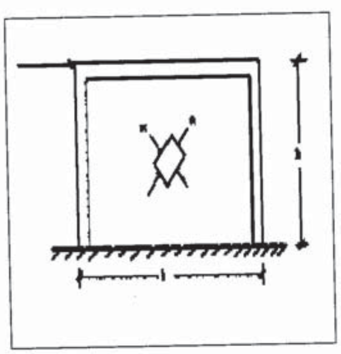


Fig: 6 Lateral load on the frame

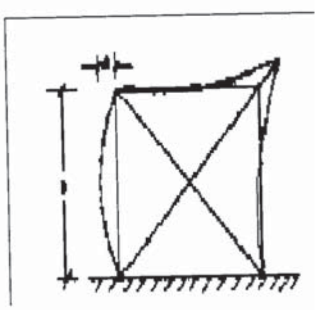


fig: 7 inter face cracking

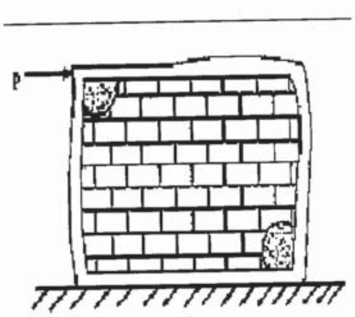


Fig: 8 Corner crushing mode

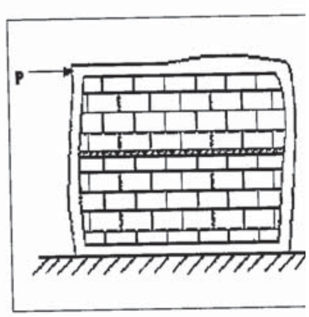


fig: 9 Diagonal cracking modes

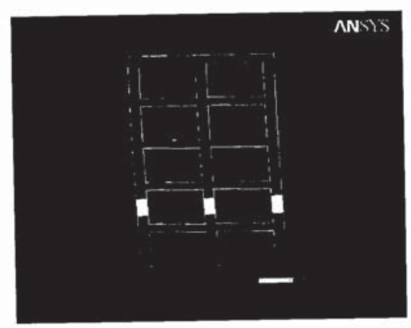


Fig: 10 Stress flow diagram for the bared frame

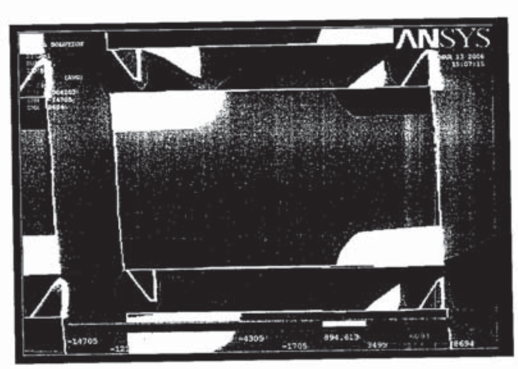


Fig: 11 Stress flow diagram for the infilled frame

CHAPTER VIII

CHAPTER EIGHT

RESULTS AND DISCUSSION

8.1 INTRODUCTION

The lateral stiffness, strength and failure mode of the models recorded in the experimental investigations. For convenience in comparison, the stiffness of a model is measured by the magnitude of the total lateral load in KN required to produce a lateral displacement at the beam column joint and the ultimate load of the model is given by the Magnitude of the largest total lateral load the model could sustain immediately before failure. In order to examine the merits of the infills, the test results of a bare frame are also included. The general behaviour of infilled frames under lateral static load will be discussed in the following Articles with respect to load-deflection characteristics, Length of contact, crack propagation and failure model, strain Distribution, stiffness and strength.

8.1.1LOAD-DEFLECTION CURVES

The load-deflection curves for the single bay frame models up t(approximately 90% of the ultimate load. At the ultimate stage, due t(excessive creeping, the deflections were increasing continuously until th(models failed. Hence the ultimate deflections are obtained using LVDT, whicl records the deflection at the each level. The load deflection curves of the tw(categories of models are discussed in the following articles.

8.2 LOADS-DEFLECTION CHARACTERISTICS OF INFILLED FRAMES

The models in this category consisted of the modeled frame with infi ...vided in between the two frames of the column and the beams. The curve

with models with infill, but the ultimate deflections are much lower than the bared frames. Slip may be due to the loss of bond between the frame and the infills, and re-adjustment of the infills within the frame, Causes additional deflections to occur under constant load. Since solid infills have larger area of contact and better fit within the frames. It is interesting to note that no slip had been recorded in models.

The existence of slip in infilled frames renders the initial higher stiffness of the structure unsafe to be utilized in practice. Hence the stiffness of the structure is to be considered as the stiffness available after slip has occurred. This criterion is recommended in all stiffness calculations. The length of contact is defined as the remaining contact length between the frame and the infills after separation has occurred, and is hence a parameter specific to infilled frames.

8.3 LENGTH OF CONTACT

In the testing of series, almost immediately after the model was loaded, boundary cracks indicating relative movement between the frame and the infills could be observed to develop around the infills in frames, which propagated around the boundary ,as the load was increased until slip occurred, when the cracks were found to surround the entire boundary of the infills be observed after slip had occurred; the first visible separation could usually be observed immediately after slip and the gap widened with further increase in load. The variations with load of the observed length of contact for the interfaces of the infills with the windward columns and the beam are observed. In this model, separations were detected between the windward columns and the infills.

8.4 CRACK PROPAGATION AND FAILURE MODE:

The propagation of cracks observed in the experiments, in which the loads are acting from right to left and the values indicate the magnitude of the total lateral load in KN. It can be observed that, in general, the crack pattern in models were characterised by considerably large number of cracks in the beam column joints , propagation of cracks in much high in the windward column, where else the formation of hinges failure are also much higher in the windward column, this resulted in the earlier failure of the column. Leeward column are affected by means of the failure in the beam column joints, and with few crack pattern in the column joints.

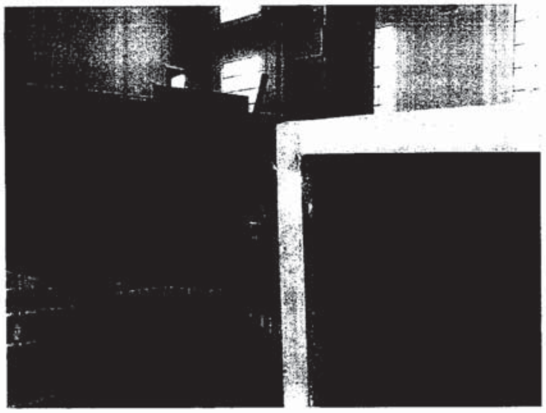
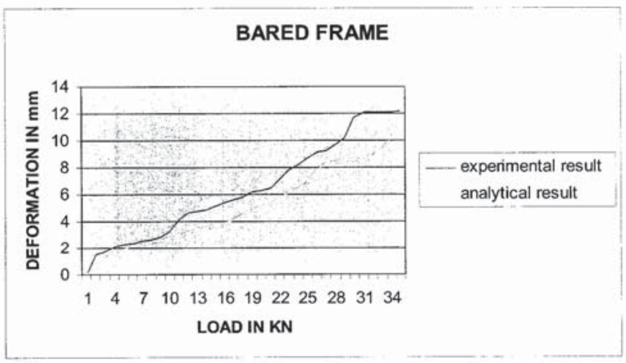


Fig: 12 loading arrangement set up (infilled frames)



8.5 LOAD-DEFLECTION CHARACTERISTICS OF BARE FRAMES:

The models in this category consisted of the modeled frame without infill provided in between the two frames of the column and the beams. The curves are characterized by the phenomenon of slips and slopes are smaller compared with models without infill, but the ultimate deflections are much larger than the infilled frames. Failure of the beam column joints, due to the lateral application of load, causes additional deflections to occur under constant load. Since there is no lateral resistance of the structure against the lateral loading, it is interesting to note that slip had been recorded in models.

As a result of larger unbalance lateral loading, failure of the hinges of the column have occurred, this causes the failure of the total bared frames

8.6 STRAIN DISTRIBUTION

The distribution of strains in the infill of the models due to the total load approximately equal to 80% of the ultimate load. Strains are calibrated using the DEMECH. In general, the strains were very small in the initial stage and increased slowly with load increments. At the intermediate stage the compressive strains increased at a quicker rate while the tensile strains increased abruptly, resulting in continuous cracking throughout the infills. This phenomenon persisted to the ultimate stage when the phenomenon of creep was very prominent and the strains were growing continuously until the models failed.

8.6.1 STRAIN DISTRIBUTION IN INFILLED FRAMES

In this type of model, the combined action of all the infills as a lateral load resisting element can be observed from the strain distribution. Along the compression diagonal which extended from the leeward column/beam, junction at 45° to the horizontal to intersect the windward column, a 'compression band' could be figured out from the strain distribution which was characterised by enormous magnitude in both tensile and compressive strains compared with other locations. Although the strains along the compression diagonal are not prominent at low load the abrupt increase at high load is obvious. In model with infills the maximum tensile strain recorded at the bottom infill after cracking had occurred.

8.6.2 STRAIN DISTRIBUTION IN BARED FRAMES

In case of this type of model, the effect of bare frame against lateral loading can be examined using the strain distribution. As a result of the lateral loading, leeward column experience the hinges failure at the beam column joint, these results in the formation of the tensile strains in the leeward column, and compressive strains at windward column, these results in the failure of the frame.

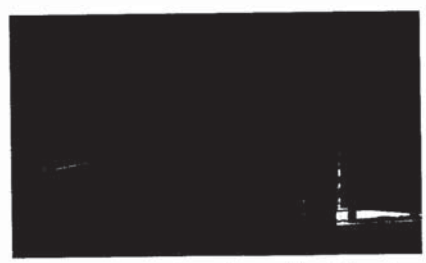


Fig : 14 experimental setup (bared frame)

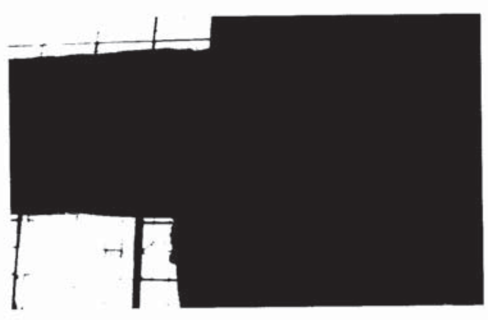
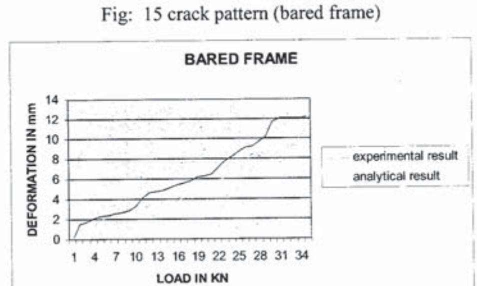


Fig: 15 crack pattern (bared frame)



8.7 COMPARISON OF RESULTS:

The experimental results were compared with analytical results and graphs were plotted load vs. deflection. In the initial stage the analytical result are more or less equal to the experimental value compare to analytical value. In the later stage the deflection from analytical work is lesser than the deflection obtained from experimental work. The ANSYS results are more accurate as compared to the experimental results because in ANSYS the whole frame is divided into elements for analysis. So the deflection in analytical is lesser than experimental.

Comparison of deformations

1. The ultimate base shear is reached in the sixteenth cycle of loading. After reaching the ultimate load, post ultimate cycles are performed to study the behavior of the RC frame until final collapse. The base shear versus storey deflection in each cycle till failure is obtained.
2. It is observed that at a maximum base shear, cracks are initiated at the junction of the loaded and middle end of the beam and column storey where the moment and shear forces are maximum. The crack patterr indicated a combined effect of flexure and shear failure.
3. Separation of infill occurred at the tension corners and the high stres: concentration at the loaded diagonal ends lead to early crushing of th loaded corners.
4. Cracks developed in the leeward column (opposite to the loaded end) c the bottom

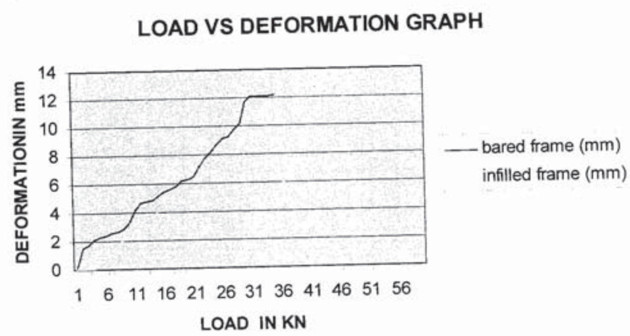


Fig: 17 comparison of bared frame and infilled

CHAPTER – IX

CHAPTER -9
CONCLUSION

9.1 BEHAVIOUR OF INFILLED FRAMES UNDER LATERAL STATIC LOAD

With reference to the behaviour of the infilled frame and bared frame models under lateral static, the following conclusions can be drawn for the performation of the soft storey frames: -

1. When the breadth/height ratio of the infill is increased, the lateral stiffness and strength of the structure is also increased.
2. The strains are comparatively high along the compression diagonals, but still significant at other locations.
3. Cracks in infilled frames are generally extensive and numerous and in form of diagonal shear cracks. Usually, tension cracks will not result in total collapse of the structure.
4. Secondary reinforcements contribute to the stiffness and strength of the structure by increasing the modulus of elasticity of the infilling material They also serve to limit the widening of cracks and spalling of the infill: after cracks have developed.
5. The contributions of the infills to the lateral stiffness and strength of th structure is usually great when the action in the infills is compressive..
6. Tension cracks in the infills will not result in total collapse of th structure, cracks in infilled frames are generally not extensive and fe in number. They usually develop along the compression diagonals an in the lintel. Beams in form of tension cracks. Infilled frames with soli infills usually fail by compression at the two compression corners.

7. When the Equivalent Diagonal Strut method is used, to analyse infilled frames without connectors, a correction value equal to half the beam depth is suitable to account for the local deformation at the joints. Between the frame and the infills other than at the two compression corners.

8. The load carrying capacity of infilled RC frame is more than that of RC frame with out infill

9. Non-linear finite element analysis is very effective in predicting the behaviour of bare RC frame and RC frame with infills

9.1.1 SCOPE FOR FURTHER STUDIES:

To go on with the further development in this study, experimental program can be carried out with the presence of the connectors in between the frames and infill can be provided. Merits of providing the connectors to the frame could result in the following merits.

9.1.1.1 MERITS OF CONNECTORS

It has been revealed in the investigations that both the lateral stiffness and strength of the bare frame are increased tremendously by the presence of the infills. However, in infilled frames with connectors, higher stiffness, strength and stability had been recorded. The merits of the connectors can be, readily' seen from the follows:

1. The contributions of the infills to the lateral stiffness and strength of the bare frame are increased by the introduction of connectors.
2. Increase in lateral stiffness and strength is readily achieved by increasing the breadth/height ratio of the infills in infilled frames with connectors, but adverse effects may occur in infilled frames without connectors if openings are present.
3. The lateral stiffness and strength of infilled frames with connectors can be predicted more accurately by virtue of its high stability whereas in infilled 'frames without connectors, the presence of slip and separations and the large number of indeterminacy render its behaviour highly unstable.

Thus it is recommended that connectors be introduced wherever possible in order to improve the behaviour of infilled frames without connectors.

9.2 HOW TO AVOID SOFT STOREY BUILDINGS:

The effects of lateral loading against soft storey buildings can be reduced With the following building components

- shear walls
- braced frames
- Moment resisting (or rigid) frames

The main horizontal resisting system for earthquakes is the floor acting as a diaphragm.

9.2.1DIAPHRAGMS

The diaphragm transfers and distributes the horizontal forces of the earthquake to the various vertical elements or systems in accordance with rigidity of the building or portion thereof. This latter determinant has to do with torsional effects. Penetrations are commonplace in floor slabs. The designer must understand the action of the diaphragm to appreciate the effects of such penetrations.

9.2.2 SHEAR WALLS AND BRACED FRAMES

These systems act as vertical cantilevers. Their lateral load-carrying function is to transfer the horizontal diaphragm loads to the foundations.

Braced frames act similarly to shear walls. The most common material for braced-frame construction is steel in the form of rolled sections or tubes. Where diagonal bracing is used, the braces in compression are sometimes ignored because of buckling. Where the bracing is in one direction only (within the plane of the braced frame) the diagonal member must be proportioned to prevent buckling when in compression.

9.2.3 MOMENT FRAMES

Moment-resisting frames counteract the horizontal forces of earthquakes through the bending strengths of the beams and columns connected rigidly at their junctions with one another. Of course, this bending is accompanied by shear forces. From an architectural standpoint, moment resisting frames have positive and negative implications:

- They allow greater flexibility than shear walls and braced frames in the functional planning of the building – positive.
- They exhibit greater deflections than shear walls and braced frames so that the detailing of non-structural elements becomes more problematic - negative.

9.2.4 NON-STRUCTURAL COMPONENTS

It is commonplace for engineers to ignore the "structural" effect of these elements. In some cases the non-structural elements provide accidental strength to the building. They may, however, interfere adversely with the structural behaviour of the essential load-carrying structure. This could lead to unanticipated overstressing of essential load-carrying members.

When the number of panels in the ground story level that can be filled with masonry walls is insufficient to offer adequate lateral stiffness and resistance in the ground story level, a ductile frame is not an adequate choice. In such cases an alternative system, like a RC shear wall, is required to provide earthquake resistance.

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