TO INVESTIGATE THE CONTRIBUTION OF STEEL REINFORCEMENT IN CORNER REGION OF MASORNY WALL FOR IMPROVING KEY CONNECTION

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Under the supervision of

Mr. Bibhas Paul

(Assistant Professor)

By

Sandeep Verma

(152659)

to

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WAKNAGHAT SOLAN – 173234

HIMACHAL PRADESH, INDIA

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CERTIFICATE

This is to certify that work titled "**To investigae the contribution of steel reinforcement in corner region of masorny wall for improving key connection**", submitted by **Sandeep Verma** in the fulfilment for the degree of Masters in Technology in Structural Engineering to Jaypee University of Information Technology, Waknaghat, Solan has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Date:

Dr. Ashok Gupta	Bibhas Paul	External Examiner
Professor & Head of Department	Assistant Professor	
Department of Civil Engineering,	Department of Civil Engineering,	
JUIT Waknaghat	JUIT Waknaghat	

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Date:

Name: Sandeep Verma

(152659)

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ABSTRACT

The use of masonry infill wall for load bearing structures is quite common practice in low seismic zones, while in the high seismic zones these are used for partition walls only. The lack of seismic standards of masonry and lack of proper design parameters has made this construction practices totally empirical based. To give aesthetic architectural look, requirements of structural system increases and for the masonry construction this lacks with limited available data of design. Studies in the past have shown during earthquake masonry structures fail more in number than framed structures due to rough design and bad execution techniques.

To improve the performance of masonry structures during earthquake, we need to understand its expected failures and reasons behind them. One of the failure i.e. in-plane and out of plane failure of masonry walls are very common. To improve this failure one need to improve the connection between the long wall and short wall. So that when earthquake comes, a rigid connection between walls can be introduced. This connection will help in providing resistance in between junction of long walls and short walls. When long wall faces earthquake, it comes under action of shear force and force tends to move its top portion away from its original position. If there is not properly designed key connection between these two walls, the long wall will fail at early age of loading. In case if there is strong key connection between these two walls, short wall will provide support to the long wall and increase its capacity to withstand an earthquake and vice versa. To ensure this rigid connection bricks are laid in bonds. But to make it more strong, one can use steel reinforcement in layer wise having L-shaped in plan at suitable vertical center to center distance also keeping economy in design. Resulted shear force, displacements, shear stresses and energy dissipation parameters satisfy the improved behavior of connection due to presence of steel reinforcement at suitable c/c vertical distance.

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CHAPTER-1

INRODUCTION

1.1 General

Masonry consists of building structures by laying individual masonry units (brick, concrete block, stone, etc). Normally cement mortar is used vertically and horizontally to bond the masonry units together. From the architectural point of view masonry construction can provide beautiful walls and floors at economical prices. As the construction of masonry includes placing it in position one by one as individual units, masonry construction tends to be quite intensive. The basic materials of masonry like cement, brick, stone and masonry construction tends to be durable construction and often requires little maintenance.

Brick masonry is commonly used in the building construction as infill also in the foundation work in the case of load bearing walls. Masonry is commonly used in the walls of buildings, retaining walls, foundations, architectural purposes and many more purposes in the buildings. Brick block are also the most common type of masonry in used in industry and may be either used in the load bearing system or in the framed system. Inspite of all this, the advancement in the masonry design for earthquake resistant has little bit knowledge.

Masonry construction results good in compressive strength but has shown weak strength characteristic in tension. So they are used mainly where compressive forces are domination and light transverse (tensile loading) loading. Masonry structures are commonly designed for the gravity loading only. Adding reinforcement in the masonry adds its tensile strength. Due to the normal axial load it causes uniform pressure distribution on the section. But in actuality an inclined load, axially load has a horizontal component which causes shear stresses along the joint. Also eccentric load may rise to the bending stresses due to eccentricity rise up.

1.2 Steel Reinforcement presence in the Masonry

As discussed earlier that brick masonry has good compressive strength but weak in the tension, so to increase its tensile capacity, steel is introduced as the reinforcement in the masonry. Cement mortar to join the bricks units together also provides good compressive strength. To ensure the durability requirement it is necessary to have good strength in both the compression and in the tension. Rebar can be added in masonry wall either horizontally or vertically to increase the ductility.

1.3 Mortar in the Masonry

Mortar binds the masonry units together by providing bond between the masonry units. Mortar generally consists of Cement/ lime, sand and water. Like concrete, mortar can be prepared at site or can be off-site. When mortar is prepared off-site, it needs to place in its position with limited time otherwise cause effects like setting. In case of site mixing of mortar, it's important that the standards of defined mortar ratio along with all its ingredients could follow. Achieving these ratio standards at site becomes difficult due to many physical reasons. The responsibility of mixing and achieving design radio as per specified totally falls to the contractor. Also it is the prime responsibility of construction supervisor that he should check quality standards with time to time and proceed in a quality fashion.

Storing condition also effects on the performance of mortar. So storing of materials in good conditions is followed. If the quality seems to be haphazard from one batch to the next, or if the consistency of the mortar seems to vary for the same specified work, the Construction Supervisor may head off future problems by noticing these issues and discussing with the Masonry Contractor.

Strength of the brickwork also is a function of the quality of the mortar used.. Thus there is an optimum relationship between masonry unit strength and the mortar strength.

1.4 Grout in the Masonry

Grouts are either fine grouts (Portland cement, lime and sand) or coarse grout (Portland cement, lime, sand and coarse aggregate). Typically the Structural Drawings and the need of bond define the type of grout needed and the respective parameter properties: strength, maximum aggregate size, etc. For grouting two basic methods i.e. low lift grouting or high lift grouting are used. Low lift grouting is the simple and best method to place scaffold height (prior to building the next lift of scaffold) or bond beam height. Vertical rebar, if required, are often placed in the cores after grouting and stirred to help consolidate the grout. The lap of bar for the vertical rebar is often a minimum of 30 bar diameters. One major disadvantage of lift grouting is that concrete masonry units courses upon the reinforcement dowels that were placed after each grout.

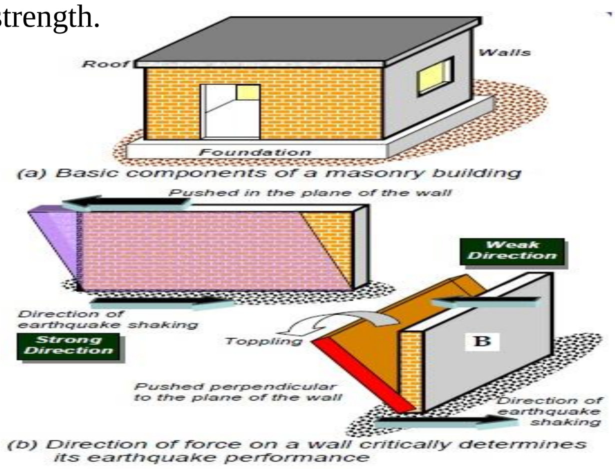
1.5 Brick Masonry

1.5.1 Introduction:

Masonry buildings are common practice in a structure. Due to strong in compression and weak in tension, structures built with masonry also known as brittle structures up to standards of designs are achieved. These are most susceptible to damage and most vulnerable part of the structure which cause deformation under earthquakes. Most of the masonry buildings in India are made up of fired clay bricks. These construction practices are engineered or non - engineered. But most of the construction practices of masonry building are non – engineered. Construction is set up with bricks without considering need of any technical aspect. Mostly these are constructed to take compressive loads during their life period. But structural integrity of these becomes major issue during earthquake. In earthquake, it causes lateral forces in the structure which finally leads to the different failure modes of the structure due to its bad characteristics for lateral loading. So finally we need to strengthen up these building to push its performance. Also we can ensure this need at the construction time by improving its ductility and tensile behavior.

1.5.2 Behavior:

Ground vibrations during earthquake cause inertial forces at the location of the mass location in the building. The force of earthquake and generated inertial forces are opposite in direction so it causes shear stresses in in-plane walls and bending stresses in out of plane walls. The generated inertial forces at the roof level travel to the foundation through the roof – wall – foundation system. Out of these three components, walls are most vulnerable to damage by the horizontal forces due to its low shear and tensile strength. The existing masonry buildings are mostly unreinforced, that is, they don't have embedded the reinforcing bars. The vulnerability of unreinforced masonry to seismic forces arises due to its low shear and tensile strength.

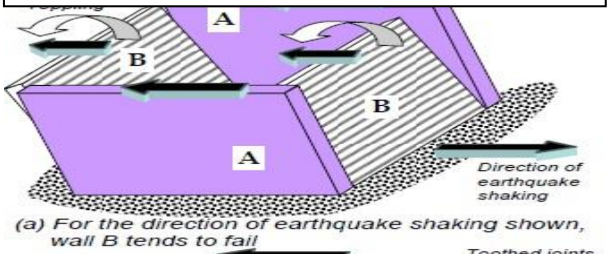


Wall lying in the direction of earthquake is called strong wall and wall perpendicular to the direction of earthquake is called weak walls.

A wall topples down if pushed laterally at the top (walls in the weak direction).

The wall B is failing due to toppling which is result of bending stresses. This bending stresses is resulted by the inertial forces due to earthquake which acts at the mass center and try to push the diaphragm opposite to the earthquake direction. So the connection between walls and diaphragm leads wall along with it. Finally bending stresses in the weak walls resulted.

Figure 1: Failures in the Masonry walls.



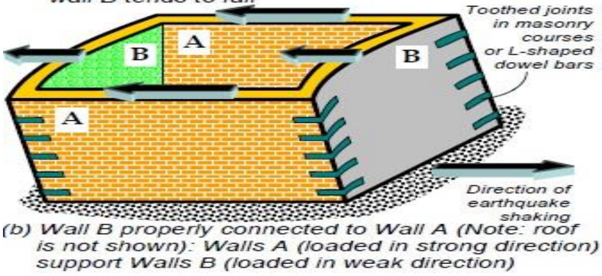


Figure 2: Box type connection.

To avoid deformation, walls need to behave as a single unit so as to provide better structural integrity. This configuration will help to take advantage of good lateral resistance of strong walls to the weak walls.

1.5.3 Behavior Improvement:

The basic need to improve behavior of masonry structure is proper interlocking box connection at junctions, roof level, lintel level and plinth level. So major challenge is how to ensure this connection?

A number of techniques with application of different material help in achieving this need. In this paper concern is given to improve the box connection at the junction.

1.6 Infill Masonry Presence Effects

In any structure presence of infill leads to the-

I. During the time of earthquake it leads to unequal distribution of earthquake forces.
II. Irregular distribution of vertical strength and stiffness leads to the soft story with high drifts and hence requiring a bit high demand of ductility as other floor frames.

III. Torsional forces set up due to the presence of the horizontal irregularities. As the concentration of more stiffness in a part of structure leads centre of rigidity to shift towards edge having more stiffness.

IV. In case of the short column which generally rises in sloppy terrain problems, the heights of some columns are smaller than columns in the other story level or in the same story. Failure causes due to over shear force and bending moment on this short column hence requiring more demand of steel.

V. During the time of earthquake in-plane and out of plane failure comes up in infill frames due to the strong wall and weak wall introduction in system. Both the failure causes causalities to be more drastic. Many earthquakes in the last some years have resulted drastic due to this failure.

The basic problem is that in the design of a R.C.C structure designer generally ignore the stiffness of infill wall performance. Researches hold in the last few years have shown that stiffness effect of the infill wall have significant role in the distribution of lateral forces. In the design stage, designer has to make clear role of infill in the structure and defining its performance capacity for the dynamic behavior. Having load resisting property of infill, the execution of work will follow –

I. Axial load.

II. Axial and lateral load: - For improving the lateral load to the wall provide frictional and mechanical anchorage at top.

III. Lateral load: - Wall connection built fixed with columns and at the top a movement joint and along sides no axial and lateral movement joints. Also must be sufficiently strong to stand effects of the inter story drift, floor movement and differential settlements. This type of wall is known as partition wall.

Leaned 6 story building in

Measuring of column drift

Joint damage

Figure 3: Formation of soft and weak story

Both windows abrupt

Infill on one side abruption

Partial infill in both sides

Figure 4: Short column effect during Wenchuan earthquake

Partial collapse of 2-story RC frame building with infill walls

In-plane failure

Out of plane failure of the masonry

Figure 5: In-plane and out plane failure

1.7 Failure modes in the Masonry

Due to the earthquake loading, it causes inertial forces set up in the structure at mass level in the opposite direction to the earthquake direction. Due to opposite direction of these two forces results in the shear stresses and bending stresses in the in-plane and out of plane walls respectively. Different failure modes are-

1. Sliding Shear failure:

It occurs in an infill wall due to sliding off the brick and mortar joint at its interface. It is caused by the low vertical load, poor mortar quality and seismic loads due to the earthquake causes shearing in the wall and resulting in the sliding. If building is jointed accurately to the foundation, next aim is for adequate resistance of the foundation itself in the form of some combination of horizontal sliding friction and in its lateral earth capacity. Most common failure of this type can be seen in the walls having poor shear strength, loaded with heavily load and load reversals may occur occasionally with along with horizontal forces.

2. Diagonal Cracks:

These cracks result due to combine action of the coming vertical load and generated shear and tensile stress from the earthquake.

3. Non structural failures:

Every structural member of the building is capable of carrying the vertical load safely also to withstand safely up to some limits is other most requirement. Non-structural members like walls, suspended ceilings, window frames should make secured against movement during the earthquake shaking. Failure of these members not cause to the building collapse but it still might cause danger for occupants and requires costly replacements or repair. Due to less resisting strength of structural members like interior partitions, windows and similar building elements are often subjected shear stresses during earthquake.

4. Failure due to overturning:

A wall having its thickness is less than its height and length is particularly vulnerable to shaking in its weak direction. For avoiding toppling of a masonry wall, its length-to-thickness and height-to-thickness ratios should be designed carefully.

The failure of infill frame is not simple to find out it depends on number of factors like its strength, stiffness, interaction between brick and mortar, openings and shear connectors etc.

Many experimental and FEM modelling researches have performed in last decades over infill frame behavior and its failure mechanism. A number of studies hold considering a single storey. Mostly the connection between infill and structural frame affects at early stage of earthquake loading and forms two compression ends. And the deformation is the function of the stiffness provided. This failure also depends on the quality of material and workmanship. As in the last it was conclude that it is not necessary to know prediction of separation because it does not affect modulus of rigidity of the whole structure (Thomas Telford, 1996) [9].

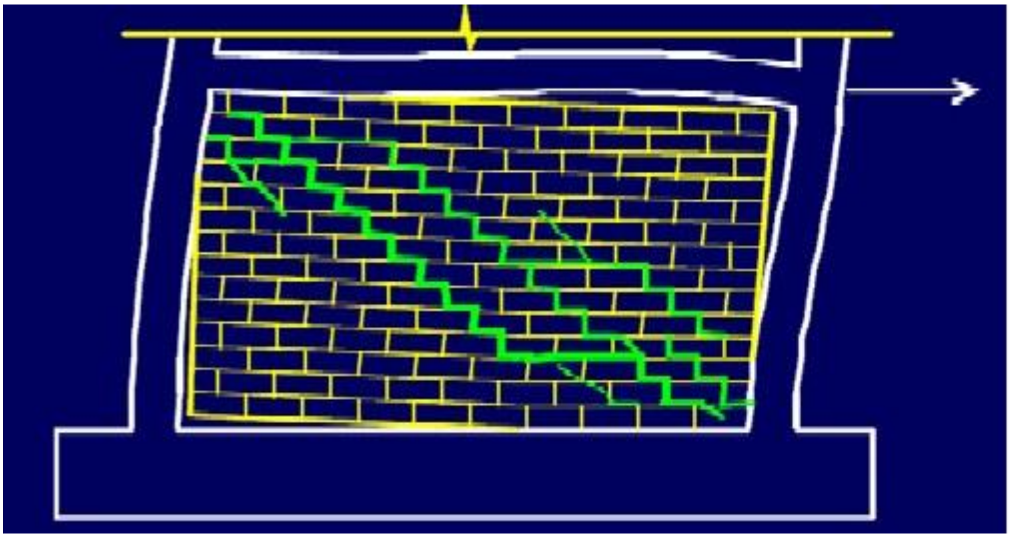
Staffor Smith (1996)[10], he conclude that the weak frame cannot transmit the forces causing due to earthquake loading to the equivalent diagonal strut of infill and finally tends to fail by crushing failure at the ends of the diagonal strut. Also he conclude that the strong frame can transmit high forces to the diagonal strut resulting which cracking initiate in the strut from central region and the crack spread towards the ends of strut If the weaker masonry is used along with strong frame system failure, failure of the masonry occurs by horizontal sliding along bed joints of brick and mortar. It means that infill frame fails at early and not able to use its full strength.

In most cases brick mortar joint is considered plane of weakness/ plane of failure due to its low shear resistance. Also cracks may be appear in the connections i.e. column and infill, beam and infill which reduces the strength capacity and finally put negative impression on performance. The failure of shear resistance between mortar joint and brick has been carried out along with weakness planes.

Merabi (2002) [11] while investigation infill frame structure (strong infill frame and weak frame system) he concluded brittle shear failure of column on windward side. As failure has taken place, the lateral load carrying capacity was increased at after stage. Also trace of ductility has studied after failure phenomena. In the infill frames with increases in loading hinges formed out also lack of shear resistance leads to failure of brick mortar joint. In the other case with strong frame system and strong frame the failure of infill has taken due to crushing and brittle shear failure of infill doesn't happen here due to heavy shear reinforcement.

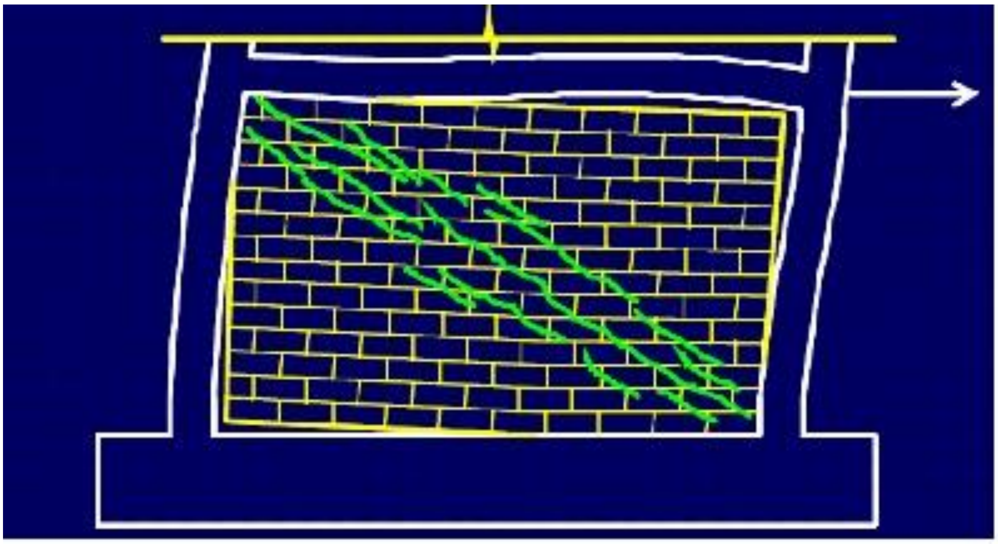
Some of the failure modes of masonry under the linearly increasing loading given by Merabi (2002) [11] as-

Mode-1



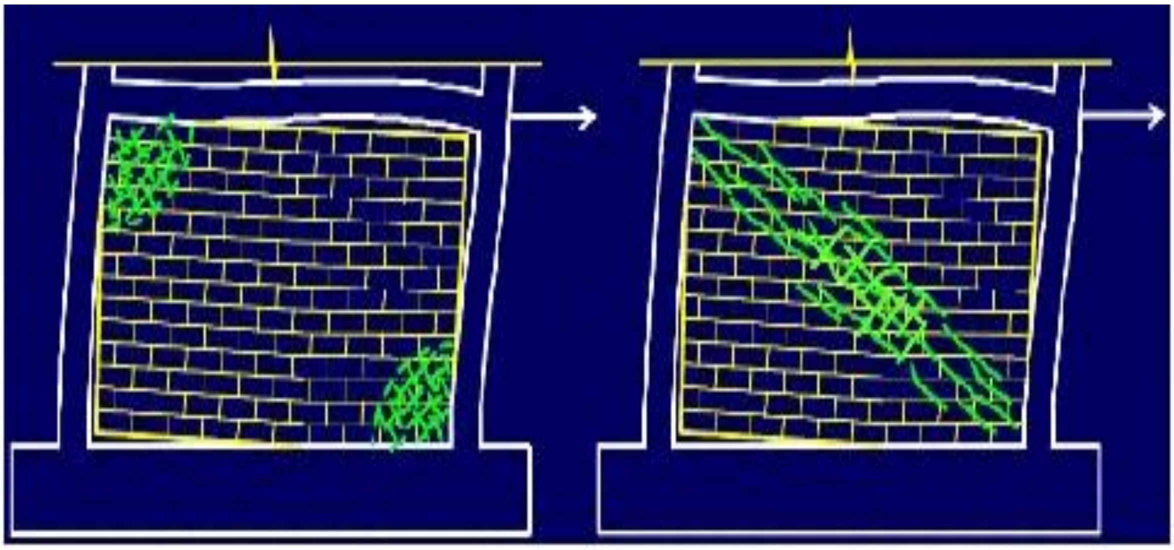
Shear Friction Failure

Mode-2



Diagonal Tension Failure

Mode-3&4



Compressive failure:

1. Crushing of concrete.
2. Failure of the diagonal strut.

Mode-5

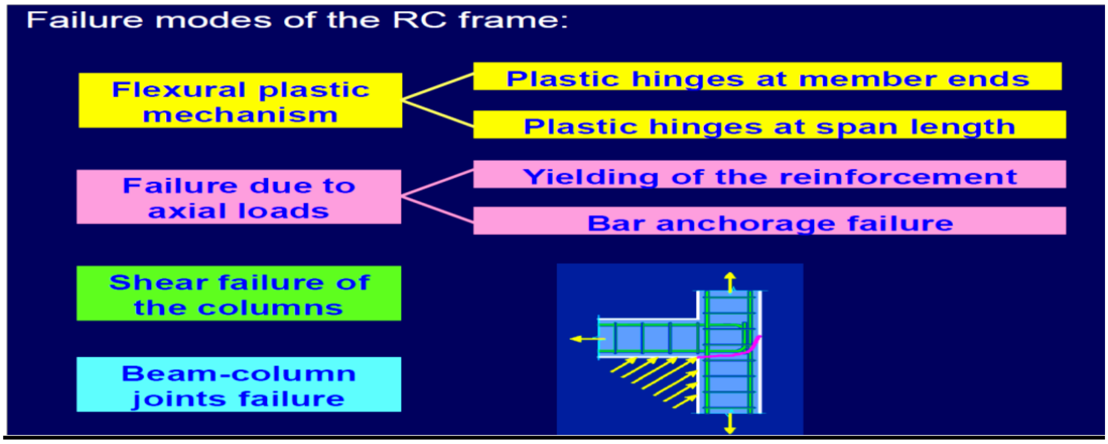


Figure 6: Infill frame failure modes

1.8 Objective

In low earthquake prone areas and even somehow in the earthquake prone areas unreinforced masonry buildings are designed only for the dead loads and live loads. But mostly are constructed without engineering practices. People are not so much aware about the seismic design and gravity loading design for unreinforced masonry buildings. For the Zone-II and Zone-III it is safe upto a limit but for the Zone-IV and Zone-V we need to improve the ductility of the structure. For this, first concern is for masonry structure Box like action. The Box connection may be called for the connection at wall junctions, roof level, lintel level and plinth level. So to investigate the presence of steel reinforcement embedded in the masonry course at junctions.

Chapter-2

LITERATURE REVIEW

1. [1]Torsional behavior of an Asymmetrical Building.

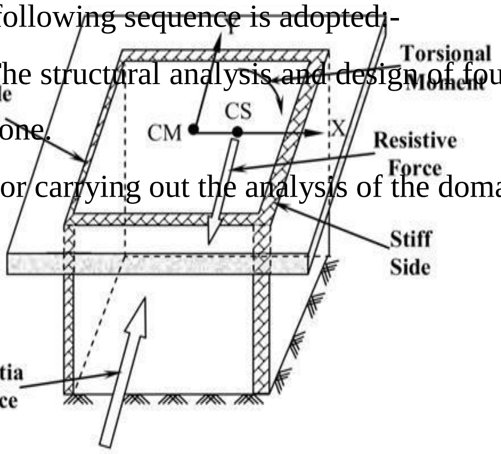
Journal: - International Journal of Modern Engineering Research (IJMER)

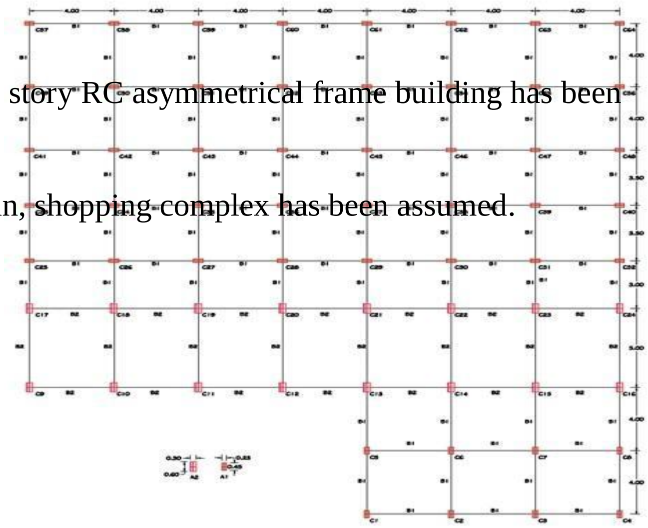
BY: - Sachin G. Maske, DR. P.S. Pajgade

Methodology: -

The following sequence is adopted: -

- The structural analysis and design of four story RC asymmetrical frame building has been done.
- For carrying out the analysis of the domain, shopping complex has been assumed.





a) Isometric view of building b) Building Plan

Figure 7: Torsion concept

- The structure is to be located in seismic zone IV. And the site have soil of medium characteristics.
- For modelling the problem, it has been divided in two cases-

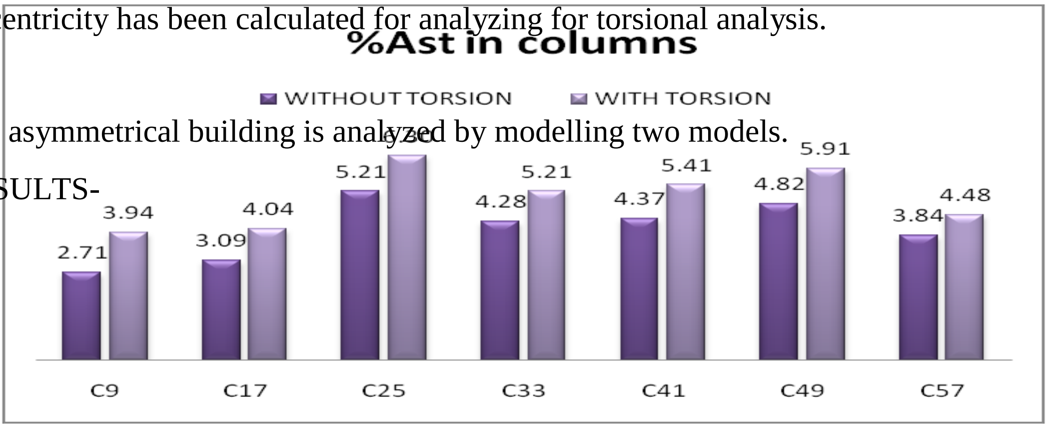
CASE-1

In this case eccentricity is not considered for torsional forces calculation.

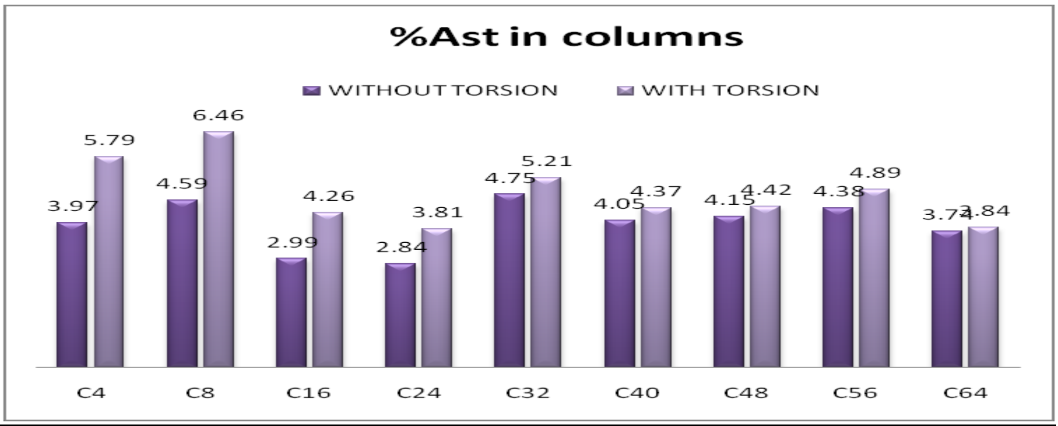
CASE-2

Seismic analysis of the building is done by considering torsion in the structure. For this eccentricity has been calculated for analyzing for torsional analysis.

- The asymmetrical building is analyzed by modelling two models.
- RESULTS-



a) On flexible side coulumns



b) On stiff side columns

Figure 8: %Ast in columns.

Conclusions: -

The main findings of this study are:

- In the asymmetric model no-2, it was observed that columns lying on stiff side faces very less forces and columns that lying on flexible side faces high forces. Column forces around centre of rigidity, there is no significant changes had seen.

- For torsional analysis, majority of designer adopts thumb rule so as to avoid calculation work for eccentricity. However this may be an inaccurate assessment. Using ETAB, which is capable of calculating parameter like the centre of rigidity through which one can able to perform torsional analysis.

2. [2] **Flexural enhancements of RC columns with FRP.**

Journal: - The 14th World Conference on Earthquake Engineering, China 2008

BY: - M Sarafraz, F Danesh

Methodology:-

- For columns maximum strains and moments occurs in the ends. And the damage is limited within the plastic hinge zone.
- To increase flexural capacity of the columns, the technique referred to as Near Surface Mounted (NSM) FRP rod is proposed. For improving the column region confinement a new method called Near Surface Mounted rods is combined with FRP jacketing for providing column end region confinement.
-

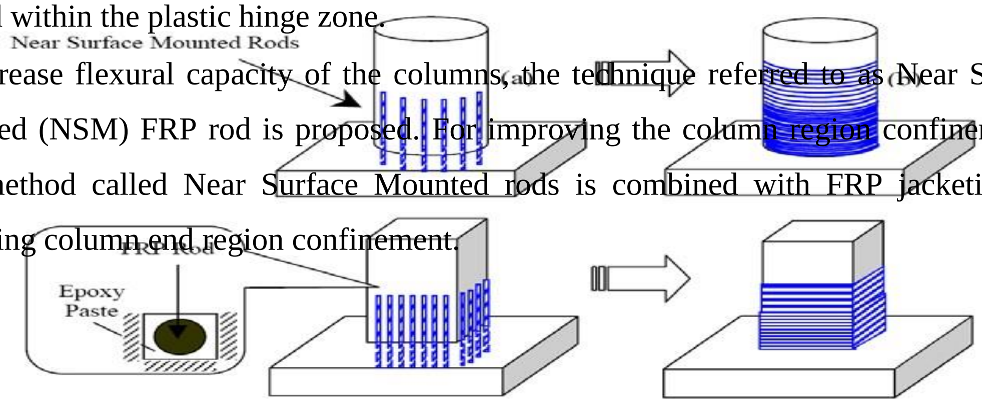
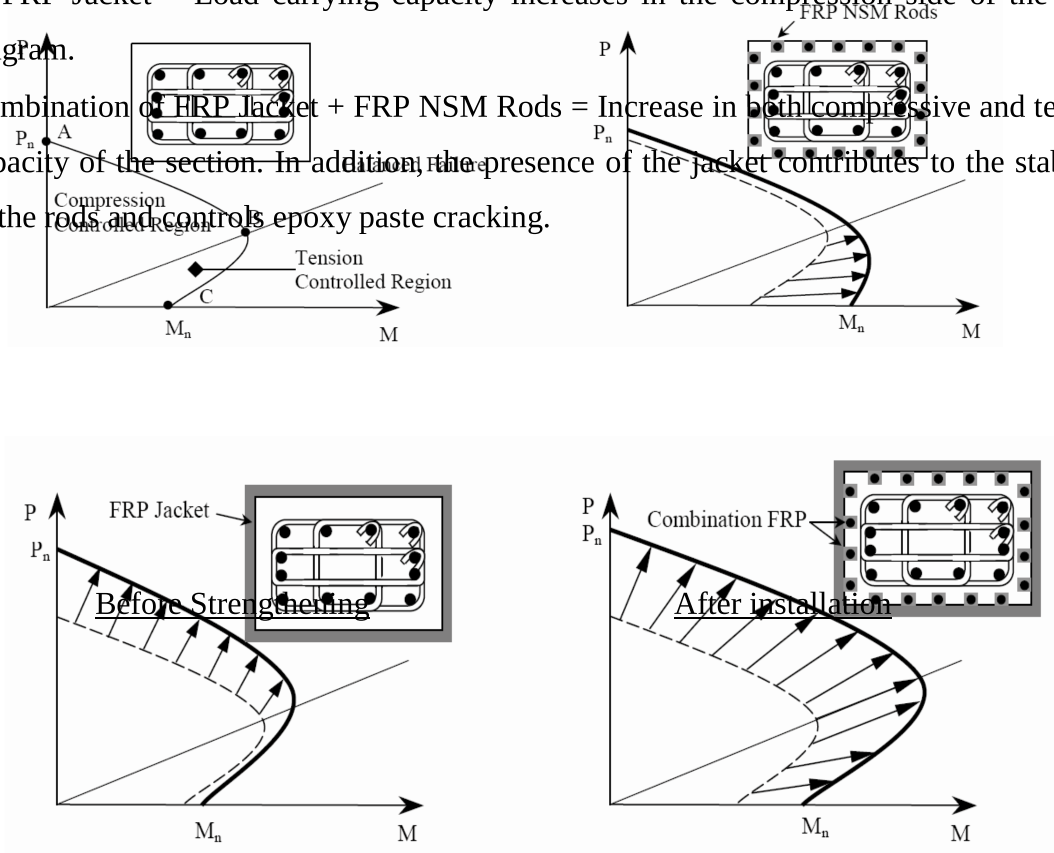


Figure 9: Installation of near surface mounted rods followed by FRP jacketing.

- Use of NSM FRP rods increases the flexural and the shear strength of deficient column and can be more convenient than using externally bonded FRP laminates in the negative moment regions of a deck.
- Load vs. Moment interaction (P-M) diagram of a rectangular column is shown. In the concrete failure results are due to crushing and in tension failure stresses are due to cracking.

- In FRP NSM Rods – Tensile strength capacity increases in the tension side of the P-M diagram resultant of which moment carrying capacity increases.
- In FRP Jacket – Load carrying capacity increases in the compression side of the P-M diagram.
- Combination of FRP Jacket + FRP NSM Rods = Increase in both compressive and tensile capacity of the section. In addition, the presence of the jacket contributes to the stability of the rods and controls epoxy paste cracking.



Before Strengthening

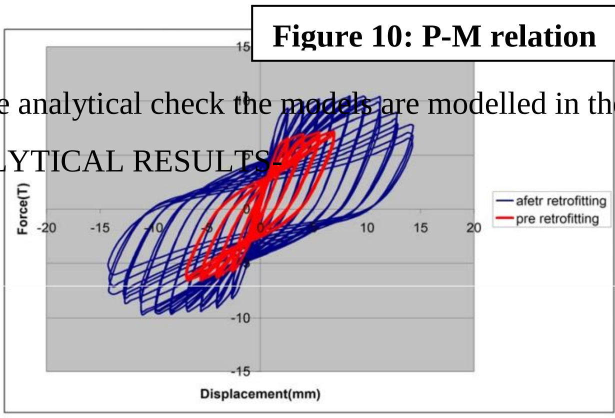
After installation

Composite Jacket

NSM rods + Composite Section

Figure 10: P-M relation

- For the analytical check the models are modelled in the DIANA.
- ANALYTICAL RESULTS-



a) The force displacement curve for before and after retrofitting of columns with FRP. This figure shows that

Figure 11: Force- displacement result

retrofitting of columns with FRP cause to enhancement of flexural capacity of reinforced concrete column.

b) Examining the final failure, the un-strengthened control specimen presented a typical bending failure mode which is preceded by yielding of the steel reinforcement followed by compression failure of the concrete. Failure NSM specimens, occurred through the simultaneous separation of the CFRP reinforcement from the concrete.

Conclusions:-

Performance analyses have been carried out on RC column strengthened with NSM system. The following conclusions derived from the experimental results.

- It has been seen that the NSM rod can increase the flexural capacity of RC column.
- It has observed that the NSM specimen utilized the CFRP reinforcement more efficiently than the externally bounded strengthening specimens.
- Combination of the FRP jacketing and NSM rods could be used for improving the flexural capacity of the damaged or undamaged columns.

3. [3]Effect of infill wall on the ductility and behavior of high strength reinforced concrete frame.

Journal: - Housing and Building National Research Center (HBRC Journal)

BY: - Ahmed Sayed Ahmed Tawfik Essa, Mohamed Ragai Kotp Badr, Ashraf Hasan El-Zanaty

Objective: -

- The aim of this paper is to check prime changes and effects of infill wall on the behavior of high strength reinforced concrete (H.S.R.C).

Methodology: -

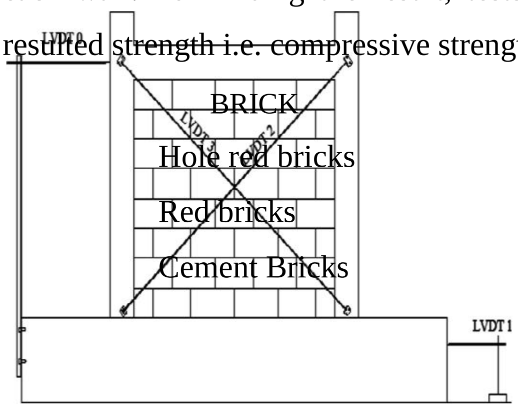
- EXPERIMENTAL PROGRAM-
  a) The experiment contains four specimens.

F1	Bare Frame
F2	Frame with infill wall thickness 12cm of hole red bricks
F3	Frame with infill wall thickness 6cm with red bricks
F4	Frame with infill wall thickness 12cm of cement bricks

  b) The mix used for the base of the frame is of M-30 and frame is of M-65.

  c) For carrying out study three infill wall prisms were taken from the same bricks for each wall. For finding the result, tests were performed on initial samples. The resulted strength i.e. compressive strength of the sample was find out then.

BRICK	STRENGTH (N/ mm^2)
Hole red bricks	3.99
Red bricks	3.45
Cement Bricks	1.95

  d) 

Four Linear Voltage Displacement Transducers (LDTVs) were used to measure the various types of deformation on frame.

Figure 12: The setup of LVDT on specimen.

LDTV 0	Loading was controlled by the displacement of LDTV
LDTV 1	Used to measure the base horizontal displacement
LDTV 2, 3	used to measure the diagonal deformation of the specimen .

e) Ultimate lateral load and relative displacement for all specimens were calculated.

f) For finding the results ductility factor and accumulated displacement ductility parameters were too calculated.

Conclusions:-

- In fill wall samples namely F2, F3 and F4 resulted in low ductility factor than the bare frame F1.
- For F2, F3 and F4 with infill wall samples, ultimate lateral load resistance parameter was greater than the bare frame F1.
- As there was decrease in the thickness of infill wall on one side and on the other side it was observed decrease in resistance of frame for lateral load. This all resulted because infill wall with small thickness takes over more buckling. The final failure occurred within small lateral load.
- For infill frames resulted energy dissipation was higher than what for bare frames.

4. [4]Out–of-plane strengthening of unreinforced masonry walls using near surface mounted fibre reinforced polymer strips.

Journal: - ELSEVIER

BY: - Dmytro Dizhur, Michael Griffith, Jason Ingham

Methodology:-

EXPERIMENTAL VALIDIFICATION

- Material properties based on the past experiments done.
- **Beam tests**
  a) For starting the project, first stage of study consisted of testing nine single leaf masonry beams. Curing had been done for 28 days.
  b) Every beam were retrofitted using a single CRPF strip inserted into a groove geometry. Allow for 7 days curing, allowing the epoxy filled around CRFP strip to reach its full strength and white coating is done on face to trace the cracking pattern.

Test setup

a) Initially four points loading was setup but due to B1 sample showed shear failure so load changed to three point loading.

b) 2 LDTV's (Linear variable displacement transducers) were used. One at the middle of the beam to catch mid span displacement. Beam B3, B7 and B9 were given semi cyclic loading.

c) Beam B4, B6 and B7 were set up with strain gauges to measure weakening effect on bond strength.

Experimental Results

a) Applied force with mid span moment presented on a secondary axis is plotted against the mid span displacement response. The flexural strength of beams with CFRP rods varied between 1.84-3.53 KNm while the flexural strength of unreinforced beams were unable to support their self weight.

b) Failure modes inspected were-

Debonding of the CFRP strips from the masonry sub trace, masonry crushing and flexure shear failure of the masonry.

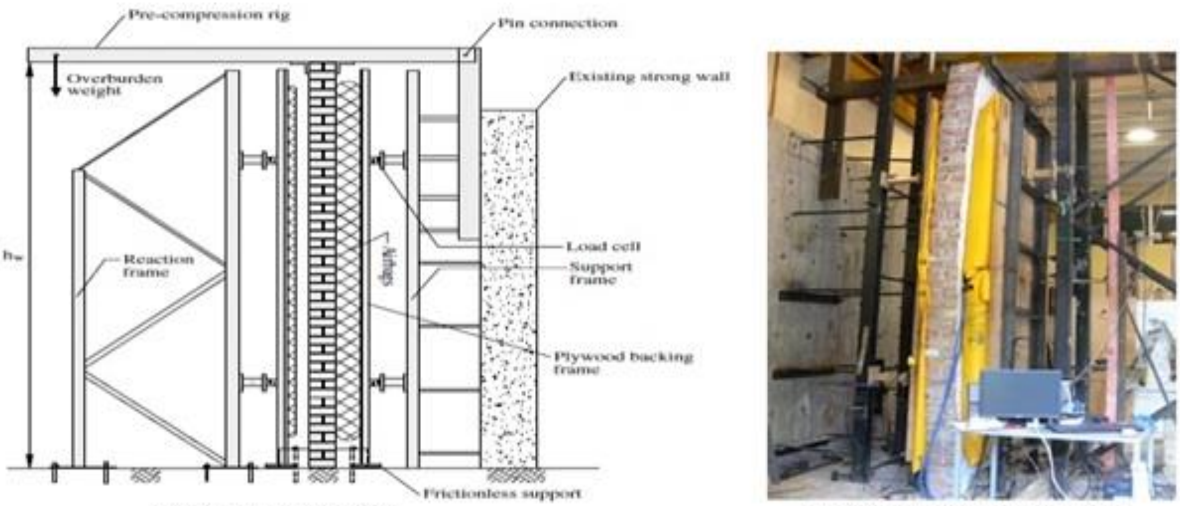
- **<u>Wall Tests</u>**

a) Second stage of experimental study consisted of testing five walls built with either two leaf or three leaf wall thicknesses. Curing of 28 was done. The wall sample was first tested in the as built condition first and then retrofitted using the NSM CFRP retrofit techniques.

b) All walls were tested using pseudo-static loading, with walls W1, W2 and W3 were tested using reversed loading cycle.

c) Wall W2 have an imposed load. The wall W3 contains its top brick courses were installed after 1 week of assembly due to delay by the mason. Wall W3 contains two CFRP rods on +ve side and one CFRP rod on –ve side. Wall W5 had two CFRP strips terminated 450mm above the base of the wall.

Test setup



(a) Schematic side view (b) Photograph of a typical test

Figure 13: Out of plane wall test.

a) In the test setup, airbags are provided to apply uniform reversed cyclic face pressure.

b) Lateral load was measured using eight 10 KN load cells (four on each side) and the lateral displacement was measured using LVDT located at mid height.

Experimental Results

a) The pressure by the airbag was increases progressively until the cracks appear. The horizontal flexural cracking through the mortar joints was seen as the mid span lateral displacement increased.

b) Wall W1 and W3 failed due to lack of top wall support in a sliding shear failure mode and wall W2 fails due to the debonding. Due to a faulted load cell W4 was not observed properly.

c) Wall 5 failed in the sliding shear at the mortar joint directly below the CFRP strips were terminated.

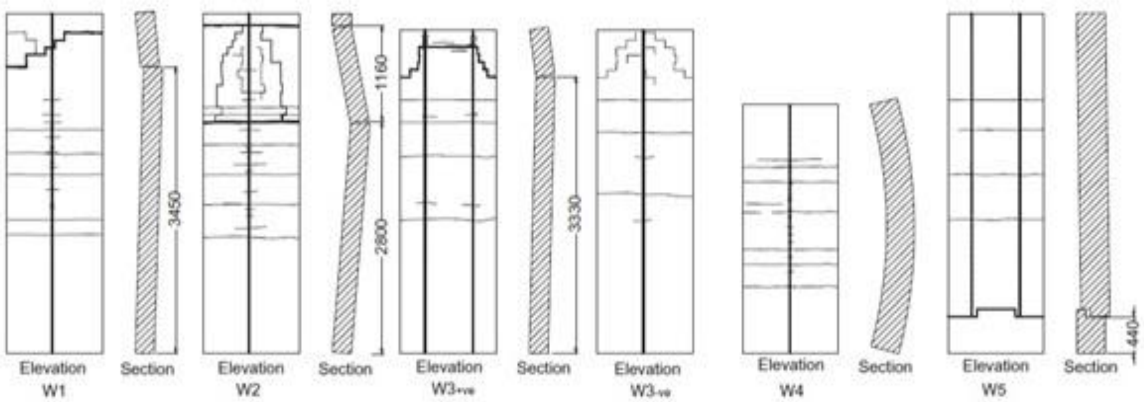


Figure 14: Observed observations and cracking

d) The stiffness degradation of the wall W1, W2 and W3 have large value at low drift ratio due to crack development over the height of the wall.

Conclusions:-

- The experimental result shows that the use of vertically orientated CFRP strips significantly increases both the flexural strength (3.05-6.21 times)and the ductility capacity of the UR walls.

- NSM CFRP strengthening results in increase in flexural strength of walls with overburden loads.
- Due to sliding shear failure of the Wall 5, it is concluded that the termination near the wall support is not safe.
- For all cyclically loaded walls, high stiffness degradation noticed at low drift ratio and progressive degradation (low rate) at high drift ratio.
- Displacement induced debonding was seen in several tested retrofitted beams. Large change in the curvature has been ensured by out of plane debonding.

5. [5]Non linear finite element modelling of RC Frame – Masonry wall interaction under cyclic loading.

Journal: - Tenth U.S. National Conference on Earthquake Engineering Frontiers of Earthquake Engineering (10NCEE).

BY: - R. Allouzi1, A. Irfanoglu, and G. Haikal3

Methodology:-

- New techniques are developed to catch the cyclic response of reinforced concrete (RC) frames in filled with masonry to experimental data.
- A finite element model (FEM) set up using ABAQUS 6.11-1.
- Software has the capability to model strength and stiffness degradation and simulate various types of failure modes in the in plane and out of plane direction.
- Continuum material models are used for concrete, steel, mortar, and bricks elements and cohesive-friction interfaces along mid thickness of mortar bed joints.
-

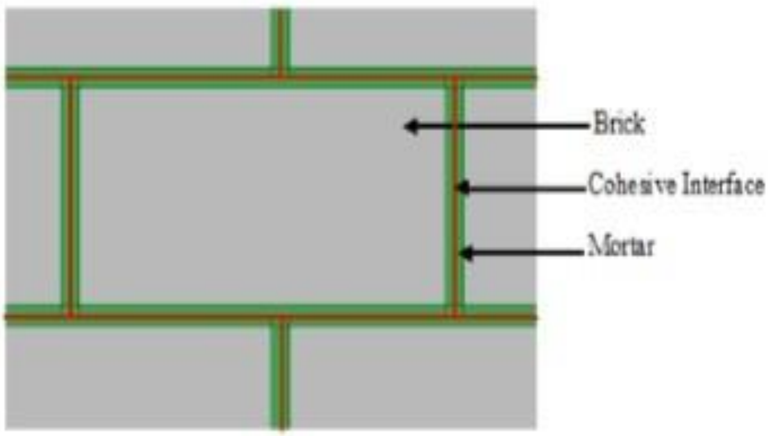


Figure 15: Finite element discretiztion of infill wall.

- The results from the models of integrated RC frame with infill wall are compared with experimental data provided.
- Finally the models are checked to various results.
- Hysteresis models of in filled RC frames are drawn and models failed by shear.

Conclusions:-

- The objective of this study was to investigate the ability of FEM model to predict the behavior of RC frames in filled with masonry wall and comparing the final results with the practical based results.
- Use of Continuum Concrete Damage Plasticity model has been adopted. For this cohesive-friction interface was introduced. It was capable of simulating the behavior of an RC frame with in filled frames under monotonic and cyclic loadings.
- Few geometric and physical properties are needed for these simulations compared to discrete model approaches.
- As in the discrete model, predefining the expected shear cracks (failure) and failure planes needs. Ultimately leading to the time consuming method.
- Develop of hysteresis models resulted from these drawn models of infilled RC frames failed by shear.

6. [6]Stress-Strain characteristics of Clay Brick Masonry under uniaxial compression.

Journal: - Journals of materials in Civil Engineering @ ASCE.

BY: - Hemant B. Kaushik, Durgesh C. Rai and K. Jain, M.ASCE.

Methodology:-

- The behavior of unreinforced masonry and its constituent for uni-axial compressive stress-strain behaviour, and like parameter. Also constituents, i.e., solid clay bricks and mortar, have been studied by laboratory tests.
- For investigating the uni-axial compressive stress-strain behavior of bricks, mortar, and masonry prisms of all samples have been constructed with different combinations of mortar and brick grades. Monotonically increasing loading apply for masonry prisms and mortar cubes and strain controlled at their top which was applied vertically by a 250 KN load and ±125 mm displacement capacity MTS servo-hydraulic actuator. However, brick units were tested in a 2,000 KN universal testing machine under stress- controlled loading.
- For the bricks stress strain curves had plotted.

Brick type	f_b (MPa)	Failure strain
M (10 specimens)	17.7 [0.23][a]	0.0072 [0.18]
B (10 specimens)	16.1 [0.08]	0.0060 [0.19]
O (10 specimens)	28.9 [0.23]	0.0070 [0.39]
S (10 specimens)	20.6 [0.17]	0.0057 [0.28]
Average (40 specimens)	20.8 [0.33]	0.0065 [0.34]

Table 1: Summary of test results on bricks

Average value of $\mathbf{E_b=300f_b}$

- Stress-Strain curve for the mortar cubes had plotted.

f_j (MPa)	Failure strain	E_j (MPa)
	(a) *Weak* mortar—1:0:6 (9 specimens)	
3.1 [0.22]	0.0087 [0.38]	545 [0.30]
	(b) *Strong* mortar—1:0:3 (9 specimens)	
20.6 [0.08]	0.0185 [0.21]	3,750 [0.16]
	(c) *Intermediate* mortar—1:0.5:4.5 (9 specimens)	
15.2 [0.06]	0.0270 [0.36]	3,300 [0.26]

Table 2: Summary of test results on mortar

Average value of $\mathbf{E_j=200f_j}$

- Stress-Strain curve for the masonry prism-

Brick type	f'_m (MPa)	Failure strain	E_m (MPa)
(a) Prisms with *weak* mortar—1:0:6 (4×7 specimens)			
M	4.0 [0.13]	0.0052 [0.53]	2,239 [0.30]
B	2.9 [0.17]	0.0034 [0.45]	1,795 [0.17]
O	5.1 [0.16]	0.0086 [0.15]	2,630 [0.14]
S	4.3 [0.17]	0.0065 [0.14]	2,355 [0.19]
Average	4.1 [0.24]	0.0059 [0.43]	2,300 [0.24]
(b) Prisms with *strong* mortar—1:0:3 (4×7 specimens)			
M	7.4 [0.10]	0.0067 [0.28]	3,585 [0.18]
B	6.5 [0.14]	0.0041 [0.39]	3,592 [0.25]
O	8.5 [0.21]	0.0057 [0.36]	5,219 [0.50]
S	7.6 [0.17]	0.0050 [0.55]	4,250 [0.44]
Average	7.5 [0.18]	0.0053 [0.41]	4,200 [0.38]
(c) Prisms with *intermediate* mortar—1:0.5:4.5 (4×7 specimens)			
M	6.5 [0.19]	0.0102 [0.17]	3,542 [0.27]
B	5.9 [0.23]	0.0062 [0.40]	3,509 [0.49]
O	7.2 [0.24]	0.0092 [0.32]	4,712 [0.33]
S	6.8 [0.23]	0.0066 [0.31]	3,325 [0.26]
Average	6.6 [0.20]	0.0080 [0.34]	3,800 [0.35]

Table 3: Summary of test results on mortar prism

Average value of $\mathbf{E_m=550f_m}$

- Control points defining the stress strains curves of the masonry.
- During the study six control points were identified on the stress-strain curves of masonry in this study, which related to the experimentally observed masonry prism compressive stresses and the corresponding compressive strains.

Conclusions:-

- The relation between brick, mortar, and masonry strengths studied.
- Effects of water absorption, initial rate of absorption, and addition of lime in the mortar on the strength and ductility of masonry were also studied.
- For approximate value of the elastic modulus of bricks, mortar, and masonry 300, 200, and 550 times their compressive strengths parameters are defined respectively.
- Increase in the masonry prism compressive strength was found with increase in compressive strengths of bricks and mortar.
- Compressive behavior of masonry with lime mortar was found to be much better than that of masonry without lime mortar. Failure strain was about 50% greater and prism strength only about 13% less than those for prisms with strong mortar.
- For the performance limit states for masonry material and member control points could be used.
- An analytical model was developed following the defined control points.
- For the accuracy of stress strain data of results, regression analysis provides accuracy in the results.
- The analytical model required only two inputs i.e. brick compressive strength and mortar compressive strength for modelling.
- When compared with several experimental and analytical researches work, resulted stress strain data have found satisfactory with accuracy.

7. [7]Shear modulus and stiffness of brickwork masonry: An experimental perspective.

Methodology:-

- Experiments conducted on six different samples as-

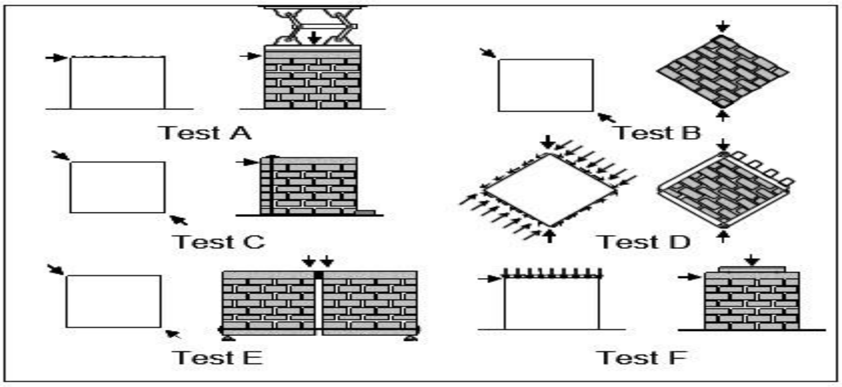


Figure 16: Different tests with different positioning.

-

Test	Test Method Basis	Type of Test Protocol
A	European pre-norm prEN 1052-1	Compressive tests of masonry wallettes with monotonic loading in a displacement-controlled mode (0.3 mm/min)
B	ASTM C1391	Monotonic loading
C	Shear Loading	Not used
D	Nichols (2000)	Square masonry shear walls at different levels of precompression and with dynamically applied shear The shear load was also sinusoidal with various frequencies
E	Shear Loading	No Protocol
F	Shear Tests with Harmonic Seismic Frequency	on the masonry cantilever walls, with the constant level of precompression and with imposed lateral loading history in sinusoidal cyclic manner

Table 4: Test samples, methods and type of test protocol.

Conclusions:-

- This paper aimed at re-evaluating the values for the shear modulus stated in many national codes considering different experimental techniques for its determination.

8. [8]Simulation of brick masonry wall behavior under in-plane lateral loading using applied element method.

Journal: - 13th World Conference on Earthquake Engineering (13WCEE).

BY: - Bishnu Hari Pandey, Kimiro MEGUR.

Methodology:-

- **Applied element method (AEM) -**

  Modelling work as we do in the finite element method is similar in nature what followed in the applied element method. Each object is divided into a number of elements of different shapes and sizes to form mesh.

  Method of joining the elements together is the main difference between AEM and FEM methods. In the AEM elements are connected by a series of non linear springs representing the material behavior. (Wikipedia)

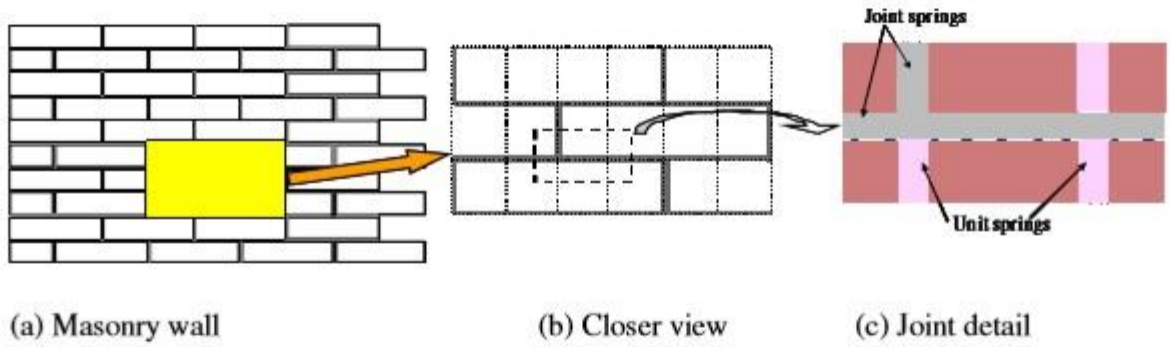


Figure 17: Modelling of bricks with mortar joint.

- **Descritization for brick masonry** is then done to take anisotropy of the material into account. A set of square elements and mortar joints, jointed together had used to define as brick unit.
- **Material** modelling-

  Failure modes noted for the model are:-

  Different cracking pattern of joints, sliding of brick unit mortar bed or head joints, cracking under direct tension, diagonal tensile cracking under high compression and shear, and masonry crushing.
- Wall behavior analysis.

- Results-

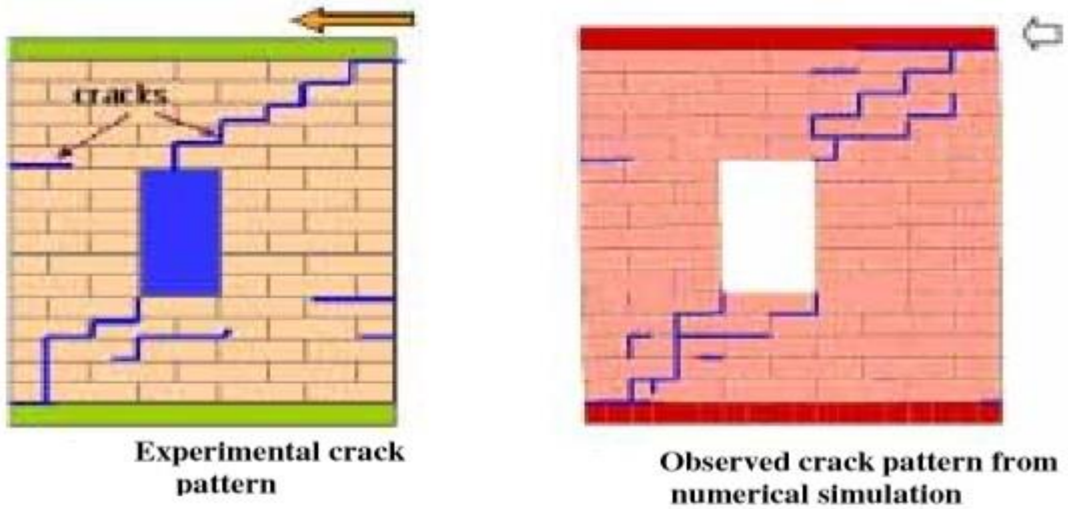


Figure 18: Cracking pattern.

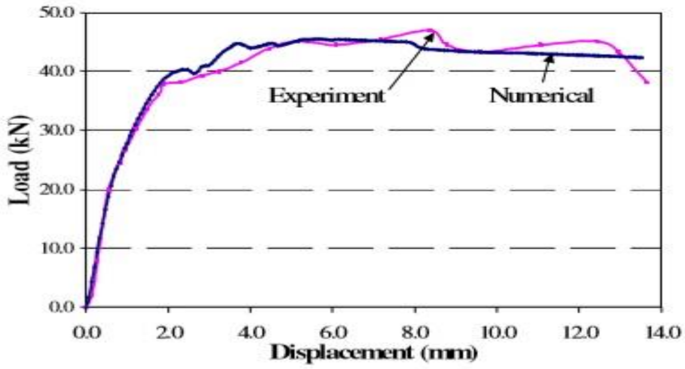


Figure 19: Load vs. Displacement results

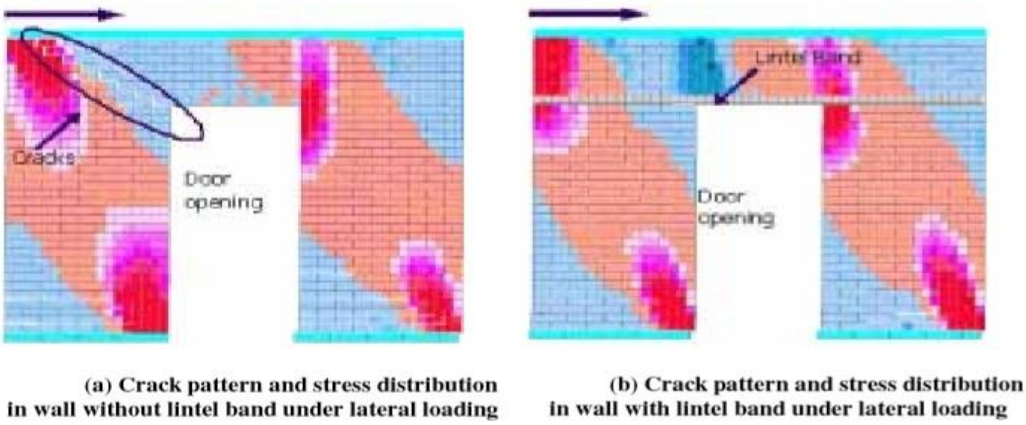


Figure 20: Crack pattern and stress distribution

Conclusions:-

- Here AEM method's capability checked to capture the behavior of the material and its results with the practical study.
- The analysis of this study under different parameters of the material is also capable to set guidelines for the masonry retrofitting.

Chapter-3

FEM MODELLING

3.1 Modelling

Modelling of any structure and getting final results are itself a challenging task. It is very difficult to draw the model as like what the structure in actual is with all the parameters defining the nature of its working. Even if someone is able to stipulate all the conditions/constraints like behavior of brick, mortar and bond between brick and mortar there is not sure that the defined behavior is as like what the structure have.

It is well known from the past researches that the presence of infill walls reduces the horizontal earthquake loading by absorbing its energy. This all results due to increase in the overall stiffness by the masonry. If centre of rigidity of the structure and the resulted inertial forces does not act closely to each other they will cause torsion. When these are closely to each other distribution of lateral forces becomes in the order of their relative stiffness. So consideration of the stiffness of infill walls becomes necessary in the dynamic analysis of structure.

To model behavior of the masonry many studies have carried out using Finite Element Analysis and Theory of Elasticity. To model the behavior between brick and mortar interface, many parameter are there depending the software which one is using. Many of the past study have carried using approximation analysis. The one best example of this is using a Equivalent Diagonal Strut. In this behavior of masonry wall is taken like a diagonal braced frame. Many of the studies have carried out for defining this strut behavior but generally defining its width always follows an approximate approach and varies researcher to researcher.

Asteris, P.G (2008) [12], for modelling behavior of masonry in finite element model two methods have been developed i.e. Micro Model and Macro Model.

Micro Model Method: - This is a method of Finite Element Method in which the each component are modelled separately like contact surface, slippage and separation etc. Results of this method are very conservation with great accuracy but the only disadvantage is that it takes more time to run analysis completely.

Macro Model Method: - This method is also known as Simplified model/Equivalent diagonal strut method. For analyzing overall behavior of masonry a diagonal strut is modelled. The number of struts using for modelling depends on the researcher to researcher. The only disadvantage of this method is that it will lack in defining the behavior of infill if there is any opening.

3.1.1 Micro Model

In Finite Element Method (FEM), problem/domain is discritized into a number of small well defined size components. Following the material properties defined and following the boundary conditions for the domain. As this method is generally used on a small part the problem, it requires a lot time for complete analysis. Some research on infill frame modelling is:-

Madan et. al. (1985) [13], in this study the investigation of the elastic behavior of a single story infill frame with opening had carried out. For defining the parameters like slip, separation and frictional loss and deboning of bond between mortar and brick a link element has adopted. In the link element, control over forces (axial & shear) and moments have achieved for getting actual behavior of the domain. For defining the opening model was achieved by assigning very low value of infill thickness, young's modulus of elasticity and having high value for poison's ratio. It was find out that as there is an increase in the size of opening, lateral stiffness of the structure is reducing. At the corners of the opening principal stresses are maximum. Also he concluded that for the infill wall with opening, equivalent diagonal strut is not applicable.

Bell (1991) [15], in this study the application of FE model to access the cracking effect and the separation of brick and mortar. In the modelling, cracking with its location, separation i.e. slips and stiffness has defined. Results shows that the decrease in the bending moment and in the deflection with increase in the stiffness of the masonry. But the bending moment increased with the crack depth. On the un-cracked section bending moment is increasing when crack size on the cracked section is increasing. The principal stresses also changes as change in the crack depth and related failure property.

3.1.2 Macro Model

The main disadvantage of Macro modelling is that whole structure behavior is analyzed as a whole which doesn't include catching its minor behaviors. The time consumption and problem complexity is less than last method here. Hence for macro modelling of infill wall strut behavior was modelled and its behavior also checked with the experimental results.

Polyakov (1960) [14], the equivalent diagonal strut concept has studied. For this a three story building was analysed. For defining one of the expected behavior of infill i.e. cracks along diagonal length of panel defined. The results show that the stresses from the peripheral members to masonry were transferred by compression corner of the frame infill interface.

3.2 Importance of Finite Element Modelling

To know the behavior of a reinforced concrete building for non linear analysis it becomes very difficult to analyze domain. All this problems has lead researchers to come up with many approximate empirical relations. All theses relations are based on experiments hold with the related aim.

Finite Element Method provides flexibility in analysing and designing with and without consideration of dynamic analysis within a short time. Also it provides user to facilitate with the different methods and parameters of analyzing. For a general R.C.C building, non-linear analysis has become common these days. This analysis provides safety against earthquakes and other natural disasters. This method is also applicable with the pre-stressed concrete.

With the passage of time many advanced technology and computers have come up. Combination of all this with computer programming of methods of analysis helps in analyzing domain within short period of time as compare to the time consumed by the manually.

The FEM has become powerful tool, in which complex domain may be analyzed for the nonlinear response with defined boundary conditions in short time.

The crack initiation, cohesion and many more like parameters has leaded a revolution in the use of finite element method.

3.3 Characterizing Elements

For drawing an element in finite element method, elements are characterized by the following-

1) Family
2) Degree of freedom
3) Number of nodes
4) Formulation
5) Integration

In the Abaqus for every element have a particular name/ identity.

1) Family:

For a stress analysis different elements used are shown here in the diagram.

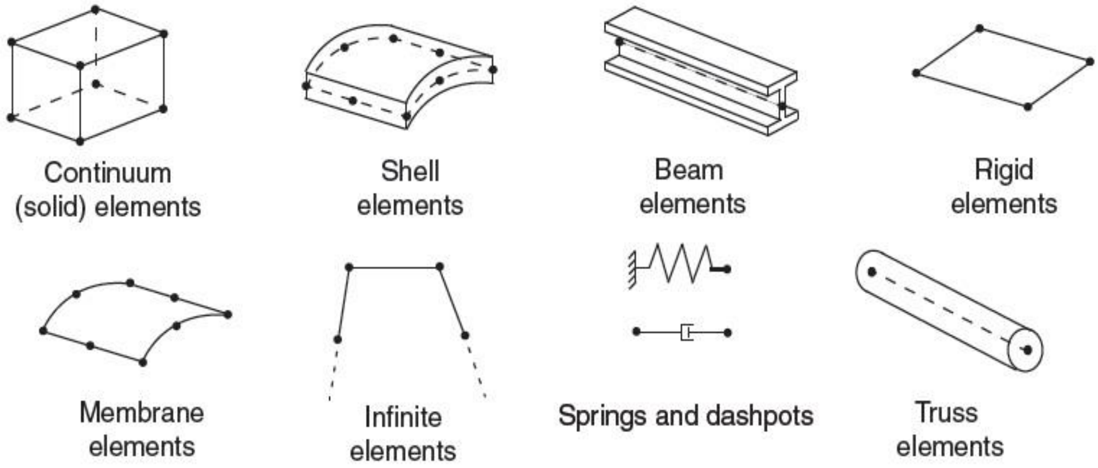


Figure 21: Elements Family

The geometry assume by each family element makes it differ from one another. In Abaqus solid element library includes two dimensional and three dimensional elements of either first order or second order using full or reduced integration. In solid library available two dimensional elements are triangle and quadrilaterals while in the three dimension tetrahedral, triangle wedge and hexahedra (bricks) are provided.

If we assign an identity S4R for an element, the first letter S reveals the family from which it belongs. Here S means the element belongs to the shell family.

2) Degree of freedom:

These are also known as the fundamental variables. For translation at each node it is the stress displacement simulation at that node. Some families have degree of rotations like beam and shell family which some have degree of freedom for temperature in heat transfer system. The generally used degree of freedom in this thesis work are-

Translation in direction-1

Translation in direction-2

Translation in direction-3

Rotation about the 1-axis

Rotation about the 2-axis

Rotation about the 3-axis

Here the numbers 1, 2 and 3 are mentioning the global directions unless there has been defined any local system.

3) Number of Nodes:

All the parameters i.e. displacement, rotation and temperature degree of freedom are defined at a node location. For finding the displacements at any other location, usually it is formed by integrating the displacement defined/analyzed in the nodal position. The order of interpolation depends on the number of nodes in an element.

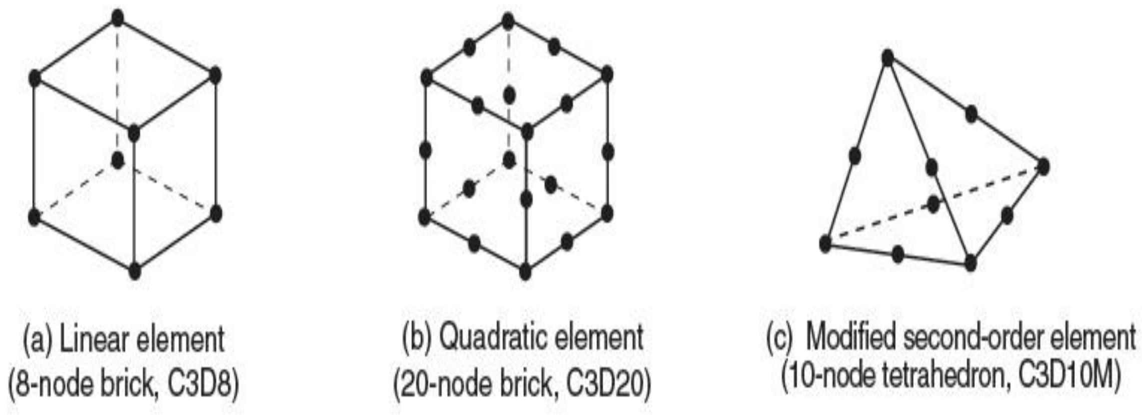


Figure 22: Linear brick, quadratic brick and modified

The linear element (8 node brick) shown above use linear interpolation in each direction. This is also known as first order elements.

The elements having nodes in the mid called second order elements and use quadratic interpolation.

The elements having nodes in the modified triangular or tetrahedral elements (10 nodes) called modified second order elements and use modified quadratic interpolation.

4) Formulation:

For defining the behavior of an element a mathematical formation is needed. Meshing of the elements is based on *Lagrangian* or material behavior i.e. the material assigned for element remains assign during whole analysis, material remain within its boundary condition. Alternate method *Eulerain* or spatial description commonly used in fluid conditions.

Some families have their different standard and alternative formulations. If any element has alternative formulation they are assigned a special character at the end of element name.

5) Integration:

To integrate over the volume .Abaqus uses mathematical formulations. Gaussian drature is used commonly for all elements. At every integration point/node Abaqus calculates the material response. Full integration or reduced integration is used in analysis depending on the choice. The choice will affect results accuracy with significant amount. A letter "R" is used at the end of an element name which signifies the element has reduced integration nature. Abaqus Standard provided both reduced and full integration elements while Abaqus Explict provides only reduced integration elements exception of tetrahedral and triangle elements and having fully integrated for first order brick, membrane and shell element.

3.4 Masonry infill frame modelling data

To investigate the objectives of this study it is necessary to model the domain as like real life case. The domain is checked for a particular earthquake acceleration data of "Trinidad-Offshore Earthquake" occurred in the November 8, 1980. For maintain the real properties of the material and interaction between brick and mortar bed, parameters have considered accordingly results of the past study.

Description of Model: -

1. A box of (3.5m*3m) is modelled for the study.
2. Thickness of the wall is 1 brick wall i.e. 10cm.
3. Mortar bed is of 12mm with mortar ratio of 1:3.
4. Fe-415 steel of 8mm.

Earthquake Data: -

Source: CESMD strong motion data (http://www.strongmotioncenter.org/)

“Trinidad-Offshore Earthquake” acceleration data has used. Earthquake occurred in the November 8, 1980.

ML=6.9 MS=7.2 M**B**=6.2

Peak Acceleration=-326.247m/s^2

3.4.1 Model of Brick-

The concrete damage plasticity may be used for defining brick material like plain concrete. R.Allouzi (2014) [17].

Concrete Damage Plasticity:

As the name is indicating in this technique both damage mechanics and plasticity are given as input for defining the actual behavior of the model. The basic objective is to draw a model which can clearly show the failure characteristics of concrete under multi axial loading. For fulfilling the requirements in modelling, combining an effective stress based plasticity model with a damage model based on plastic and elastic strain. The model response in compression i.e. uni-axial, bi-axial and in trial-axial and in tension is compared to the experimental results. The model defines the increase in strength and displacement capacity for increasing the confirmation levels. Furthermore the model is checked for compressive and tensile behavior for accurate structural analyses.

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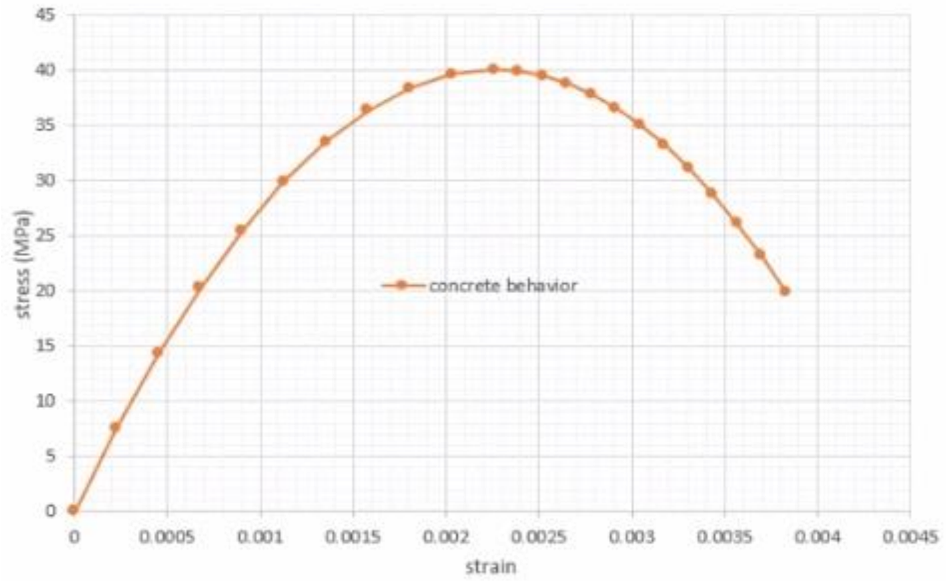
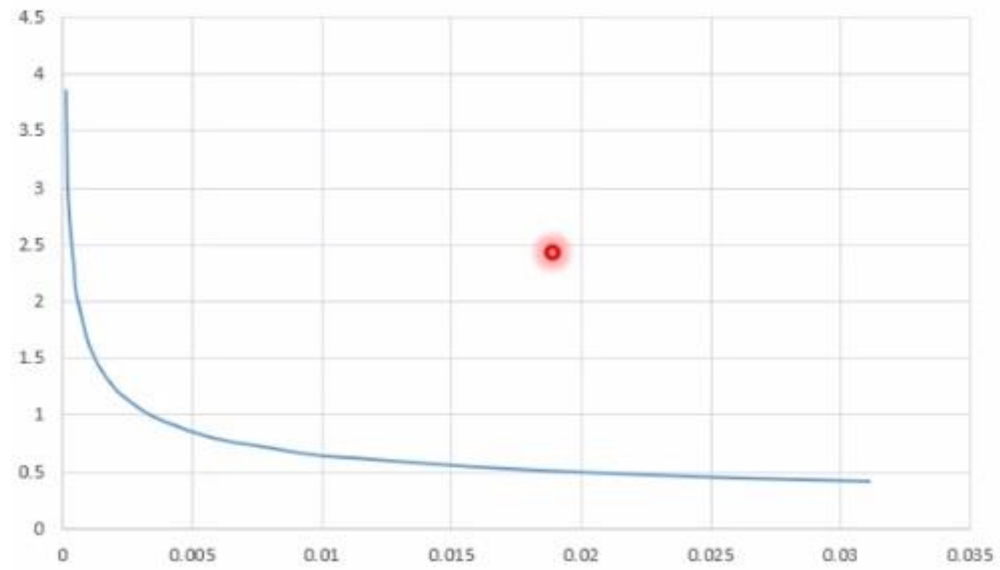


Graph 1: Compression behavior of Concrete

- Concrete in compression failing in crushing.



Graph 2: Tensile behavior of Concrete

- Concrete in tension failing in cracking.

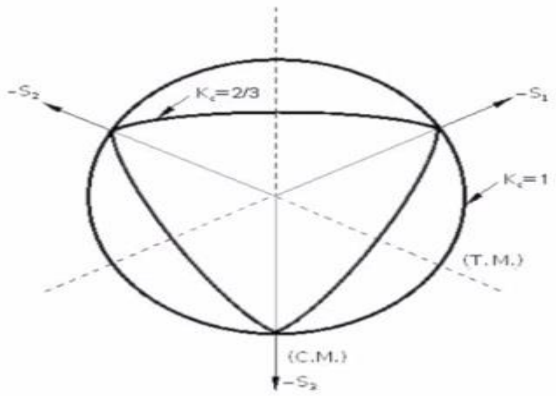


Figure 23: Drucker-Prager strength Hypothesis

The model i.e. CDP (concrete damage plasticity) that we used in the Abaqus software is the modification of Drucker-Prager strength Hypothesis. Afterwards, this drucker prager strength hypothesis many researchers had worked on it and modified the guidelines accordingly. Modifications over the time done by researchers tell that the failure surface in the deviatoric cross section needs not to be a circle and it is governed by the parameter Kc.

Kc: - The distance between the hydrostatic axis / respectively the tension meridian and compression meridian in the deviatoric cross section. Value of this parameter is always greater than .5 and when this value is assumed to be 1, the deviatoric cross section of the failure surface becomes a circle. Lee (1998) [16], reports that according to the experimental results this value for the mean normal stress equal to zero amounts to 0.6 and as there is decrease in the mean stress value it results in increase of value. The CDP model recommends to assume Kc=2/3. This was resulted from the output of triaxial stress test and is a theoretical test.

There are many parameters defining the behaviour of concrete. One of them which tells about the state of the material at which point concrete undergoes failure under biaxial compression σ_{b0}/σ_{c0} (f_{bo}/f_{co}) is the ratio of the strength in the biaxial state to the strength in the uni-axial state. The most reliable in this regard are the experimental results reported on which results were verified. After this approximation with the elliptic equation, uniform biaxial compression strength f_{cc} is equal to 1.16248 f_{c0}. The ABAQUS user's manual defines default of σ_{b0}/σ_{c0} as 1.16.

Dilation angle- In the plastic shearing process it results in plastic volumetric strain which is controlled by the dilation angle. And dilation angle is assumed to be a constant during the plastic yielding. Also to know at which angle the failure surface is intersecting with horizontal i.e. hydrostatic axis it is measured in meridional plane. Often, dilation angle ¥ is represented as a concrete internal friction angle. Usually ¥=36° or 40° are the general values that one assumes in design..

The biggest and reliable advantage of the CDP modelling is that the method is totally reliable on parameters having an explicit physical interpretation. The parameter defining the exact role of patent material and all the mathematical method, that one use in Abaqus for defining a boundary conations to the domain in 3-D are explained in Abaqus user manual. The other parameters describing the performance of concrete are determined for unaxial stresses. Parameters value used are:-

Parameter Name	Value
Dilation angle	36
Eccentricity	0.1
f_{bo}/f_{co}	1.16
K	.667
Viscosity parameter	0

3.4.2 Model of Steel- R.Allouzi (2014) [17].

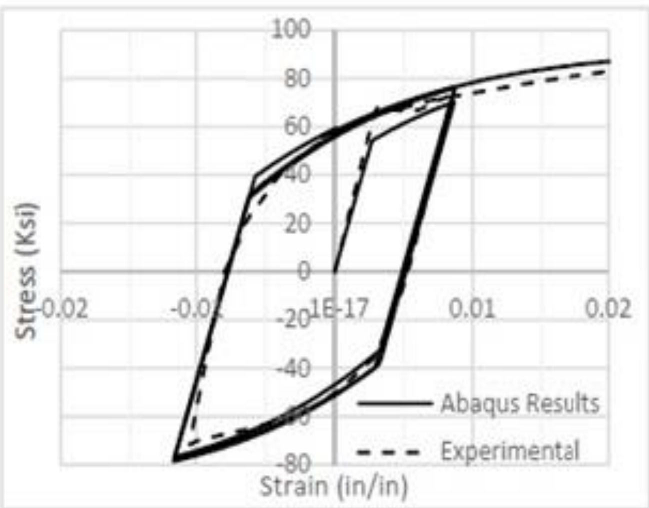


Figure 24: Stress strain curve under cyclic loading

1. For using steel reinforcement in the infill wall elastic plastic model of steel is used for simulation of uniaxial loading.
2. For the modelling of inelastic behavior of steel hardening and stabilized type of data, stress strain diagram present here is used.

3.4.3 Model of brick mortar joint-

As in this thesis work, macro modelling is used so to avoid uncertainties in analysis modelling is done by a solid masonry wall rather doing it in parts. For the micro modelling it's possible to do modelling in parts and this input shear stress- shear displacement behavior at joint is followed by R.Allouzi (2014) [17].

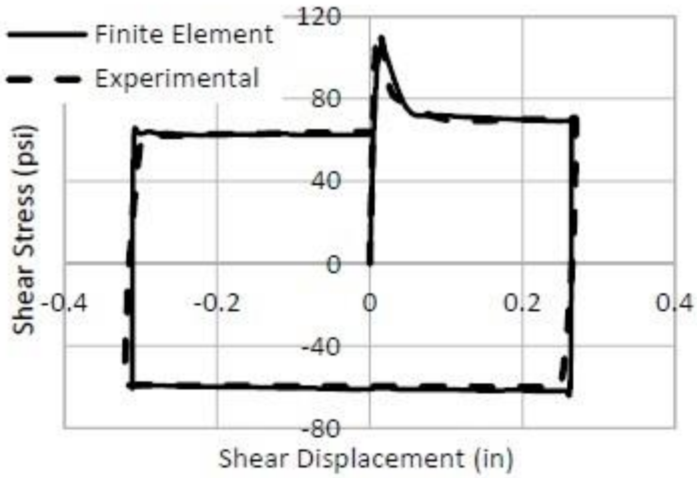


Figure 25: Shear stress, shear displacement curve of mortar joint between two solid concrete bricks under 75 psi normal stresses.

3.5 Modelling of Brick Masonry Box

In this case of modelling, simple analysis of a masonry wall needs to carry up. This is all because to ensure the presence of reinforcement in the corner regions can provide box action strongly and can improve its stresses carrying capacity along with all other advantages over wall without reinforcement in corner region. This modelling also ensures the results by satisfactory macro modelling in both cases. The modelling work is divided into two parts. The part-1 is modelling of masonry infill frame without steel reinforcement in the corner region. And the part-2 is modelling of masonry infill frame with steel reinforcement in the corner region

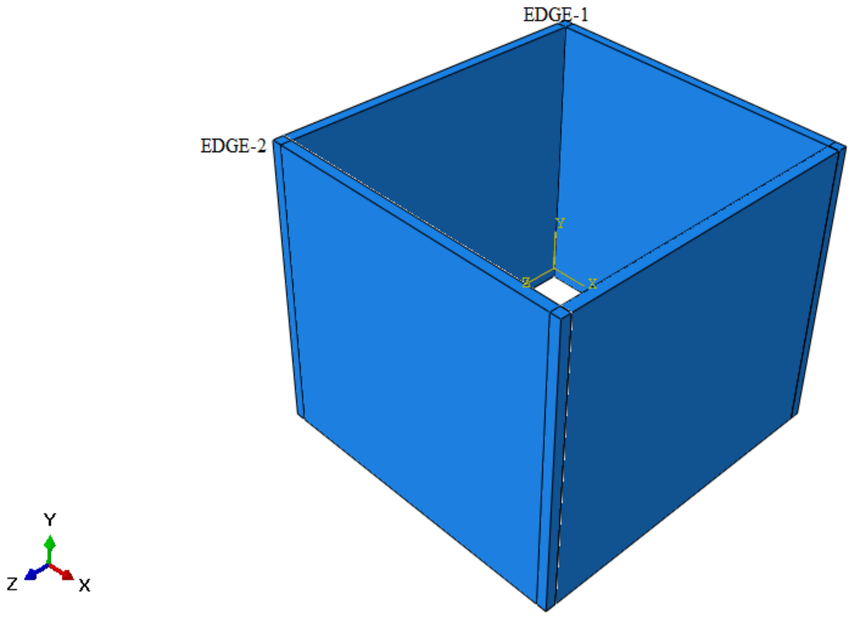


Figure 26: An isometric view of a masonry box.

3.5.1 Modelling of a masonry box

All dimensions are in (N, m).

Case-1: - Steps for modelling masonry infill without reinforcement.

1. Go to the module Part.
   a) Create part-1 named as “half brick masonry”.

2. Go to the module Property.
   a) Create material named as “masonry”, defining its density and elasticity.

      Density=2000kg/m^3 Elasticity=118 MN/m^2

      Poisson’s ratio=.15
   b) Defined concrete damaged plasticity

      Plastic behavior.

      Damaged behavior.
   c) Assign section.

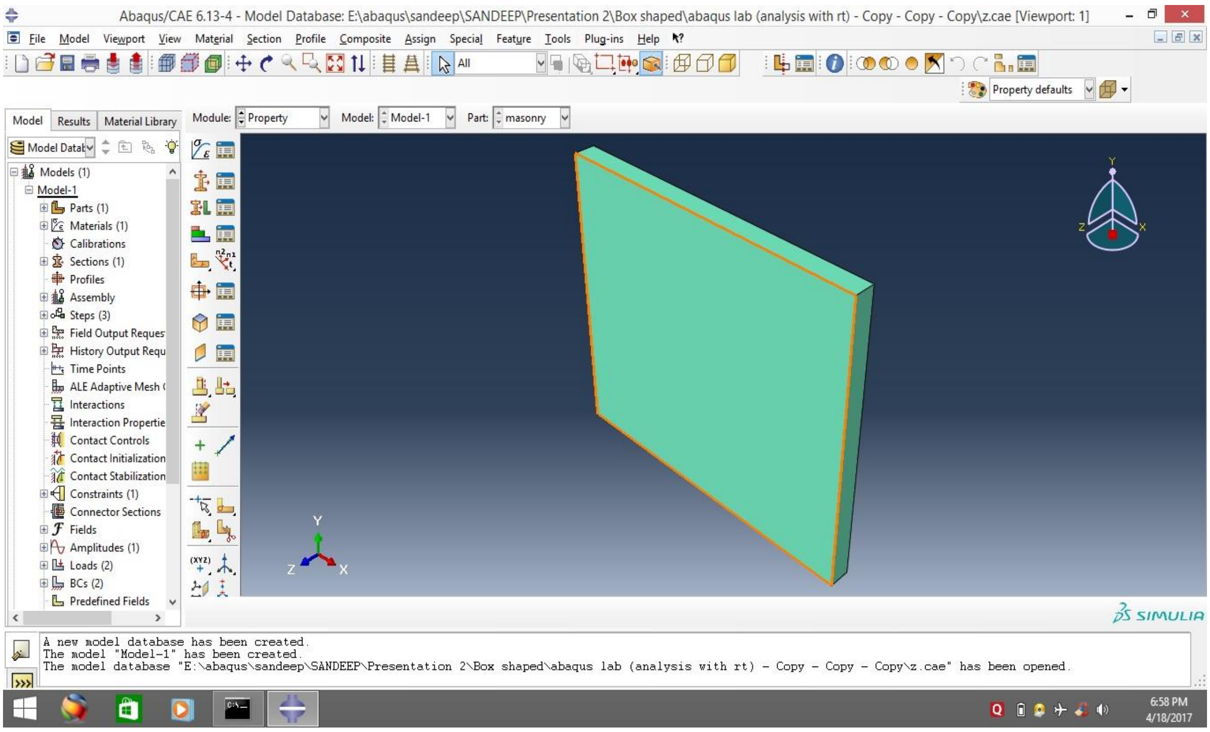


Figure 27: An isometric view of a masonry wall.

3. Go to the module Assembly.

a) Create instance: Here geometry of the object is created. Linear pattern, translate instance and rotate instance helps to set up the geometry.

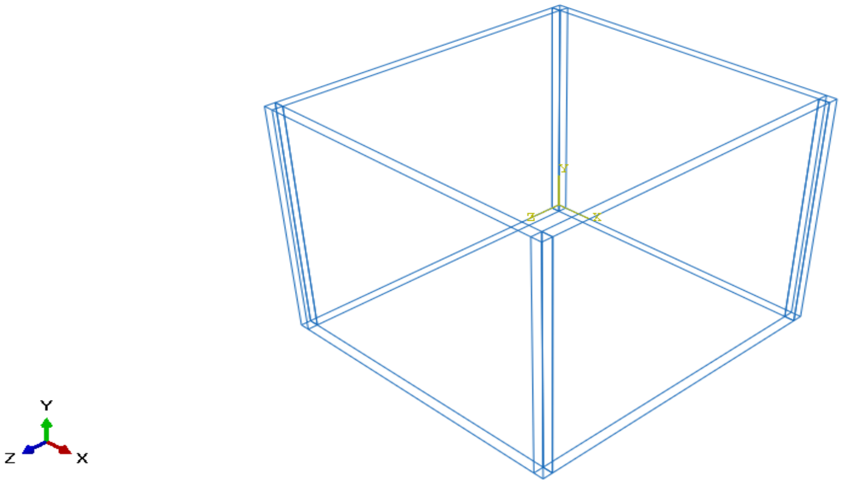


Figure 28: Box without reinforcement.

4. Go to the module “Step”.

   a) In the step manager create steps. Here default initial step automatically generates and two more steps i.e. Eigen and Seismic are defined in this modelling. For the Seismic case earthquake acceleration data has been defined. “Trinidad-Offshore Earthquake” acceleration data has used. Earthquake occurred in the November 8, 1980.

   ML=6.9 MS=7.2 MB=6.2

   Peak Acceleration=-326.247m/s^2

   b) In the field output manager defining the parameters as we want to manipulate the results.

5. Go to the module “Interaction”.

   a) Here we need to define the connection between the wall to wall contacts.

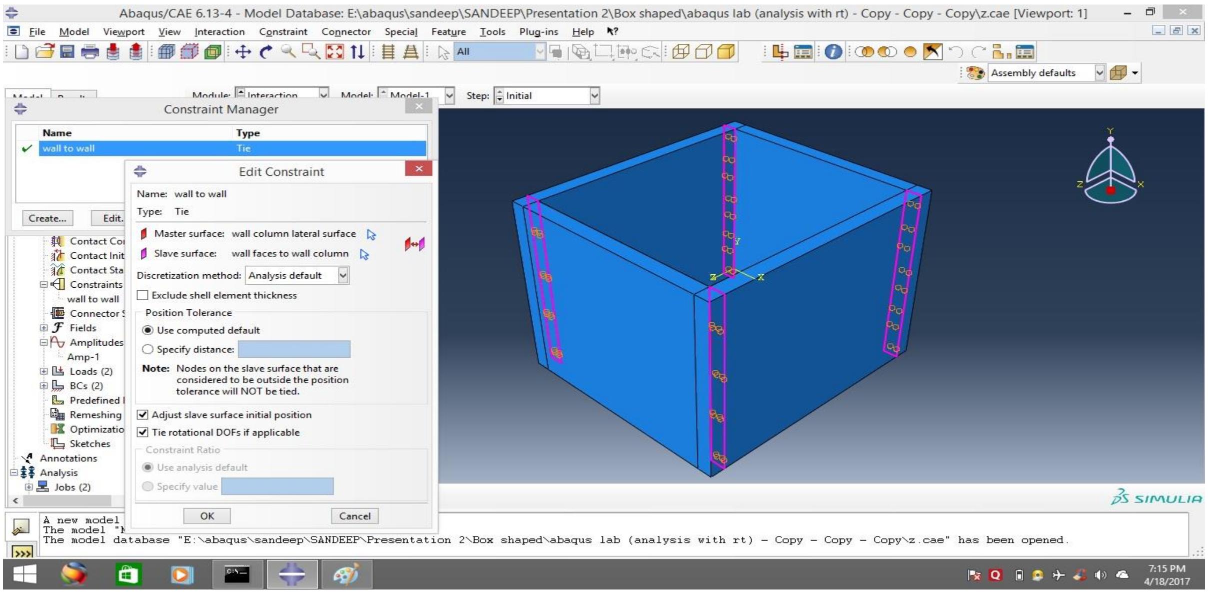


Figure 29: Interaction between connections

6. Go to the module “
   a) Here defining the dead load and live load coming over the box for the seismic load.
   b) Defining the boundary condition for both the steps.
7. Go to the module “Mesh”.
   a) Meshing by parts or assembly depending upon the need as defined in the assembly case.
   b) Defining seeds.
   c) Setting the element type for the meshing.
   d) Creating the instance and controlling the mesh.

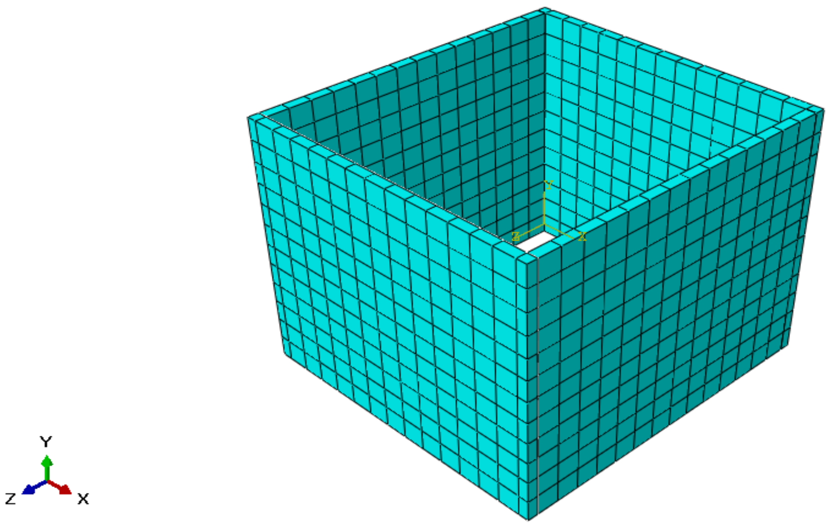


Figure 30: Box with meshing.

8. Go to the module “Job”.

a) Creating job and thus submitting the data.

b) After completion go for the results.

Case-2: - Steps for Modelling of masonry with reinforcement**.**

Steps for Modelling

1. Go to the module Part.

   a) Create part-1 named as “half brick masonry”.

   b) Create part-2 named as “rebar”.

2. Go to the module Property.

   a) Create material named as “masonry”, defining its density and elasticity.

   Density=2000kg/m^3 Elasticity=118 MN/m^2

   Poisson’s ratio=.15

   b) Defined concrete damaged plasticity

   Plastic behavior.

   Damaged behavior.

   c) Create material named as “rebar”, defining its density and elasticity.

   Density=7850kg/m^3 Elasticity=200 GPa

   Poisson’s ratio=.3

   d) Assign sections.

3. Go to the module Assembly.

   a) Create instance.

   Here geometry of the object is created. Linear pattern, translate instance and rotate instance helps to set up the geometry. Also

   Portioning helps in connection walls at the corner regions.

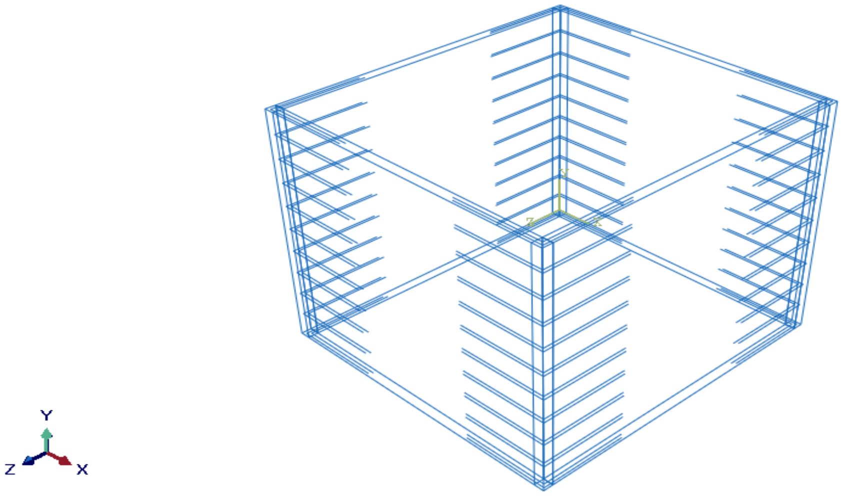


Figure 31: Box with reinforcement at corners

4. Go to the module “Step”.
   a) In the step manager create steps. Here default initial step automatically generates and two more steps i.e. Eigen and Seismic are defined in this modelling. For the Seismic case earthquake acceleration data has been defined. “Trinidad-Offshore Earthquake” acceleration data has used. Earthquake occurred in the November 8, 1980.

      ML=6.9 MS=7.2 M**B**=6.2

      Peak Acceleration=-326.247m/s^2
   b) In the field output manager defining the parameters as we want results.
5. Go to the module “Interaction”.
   a) Here we need to define the connection between the wall to wall contacts and the reinforcement relation with the surrounding material.

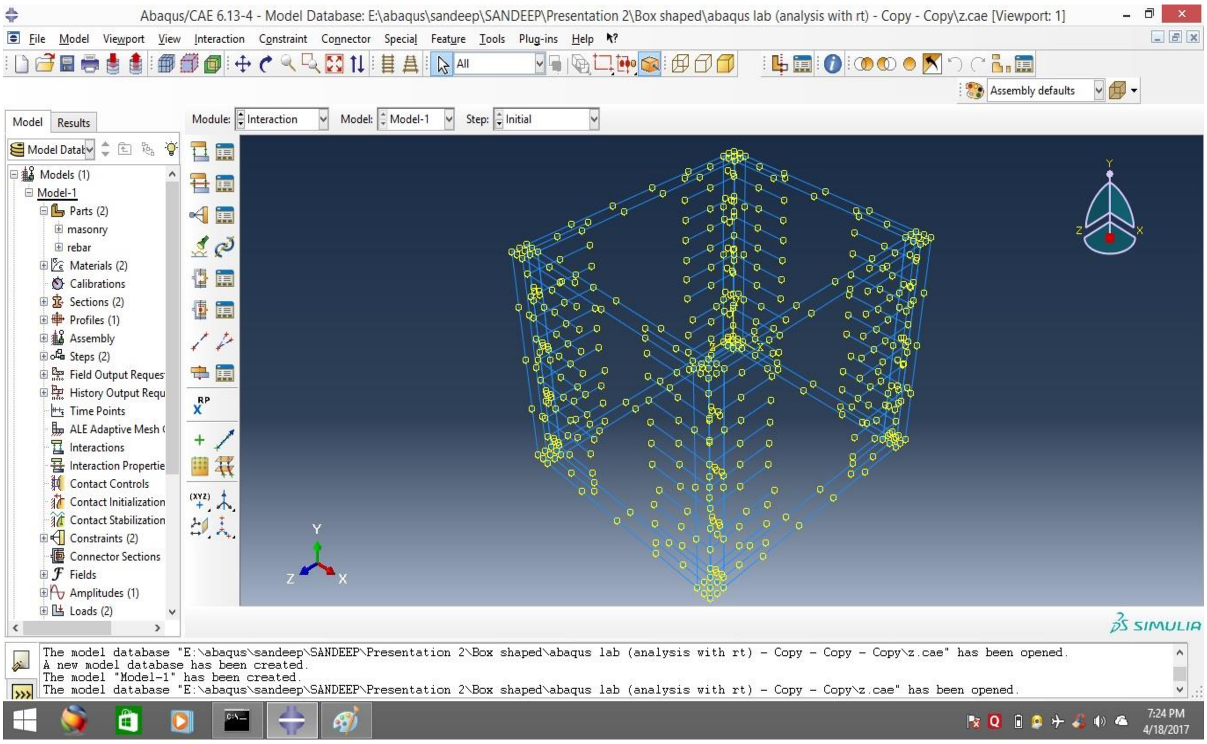


Figure 32: Interaction between walls and steel reinforcement.

6. Go to the module “Loading”.
   a) Here defining the dead load and live load coming over the box for the seismic load.
   b) Defining the boundary condition for both the steps.
7. Go to the module “Mesh”.
   a) Meshing by parts or assembly depending upon the need as defined in the assembly case.
   b) Defining seeds.
   c) Setting the element type for the meshing. Creating the instance and controlling the mesh.

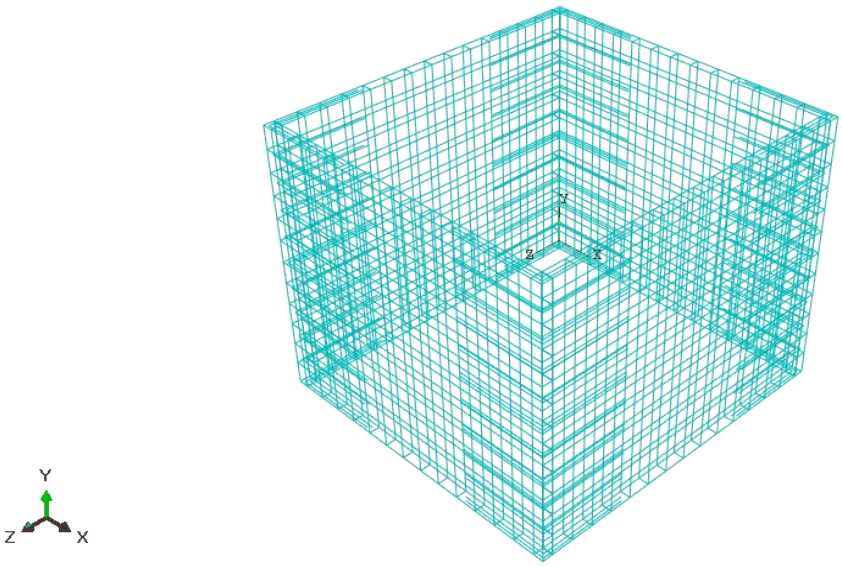


Figure 22: Box with meshing

8. Go to the module “Job”.
   a) Creating job and thus submitting the data.
   b) After completion go for the results.

Chapter-4

RESULTS AND DISCUSSIONS

In this chapter result of the finite element analysis of the masonry infill frame for a particular earthquake acceleration data has been performed using the Abaqus software. Results of the masonry infill frame with and without steel reinforcement in the corner region are compared. After that behavior of stresses, displacement and forces have studied.

4.1 Finite Element Results of Masonry Infill Wall

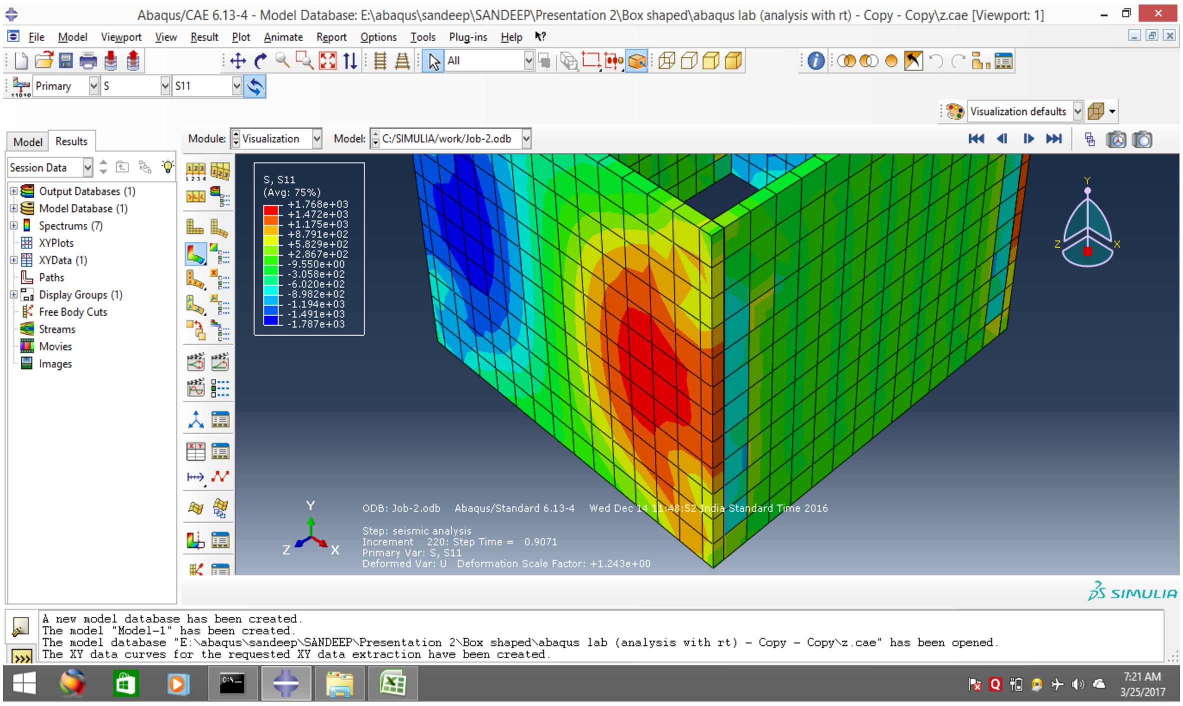
After following all the steps of modelling and analyzing model for a particular defined acceleration data the different parameters have studies here. First of all let’s discuss about the expected failures of the masonry infill walls. This will be shown by contour graphs.

Stress concentration in the masonry infill wall leading to the crack initiation at respective position. When forces are distributed uniformly over an area the object is said to be strong or rigid. In the stress concentration, area for distribution of forces becomes less and cause localized increase in stress. Now when the generating stresses become greater than material’s strength, crack initiation takes place. This zone of stress concentration also leads to fatigue stresses. Sometimes to overcome this defect results in better strength. The main parameter reflected are-

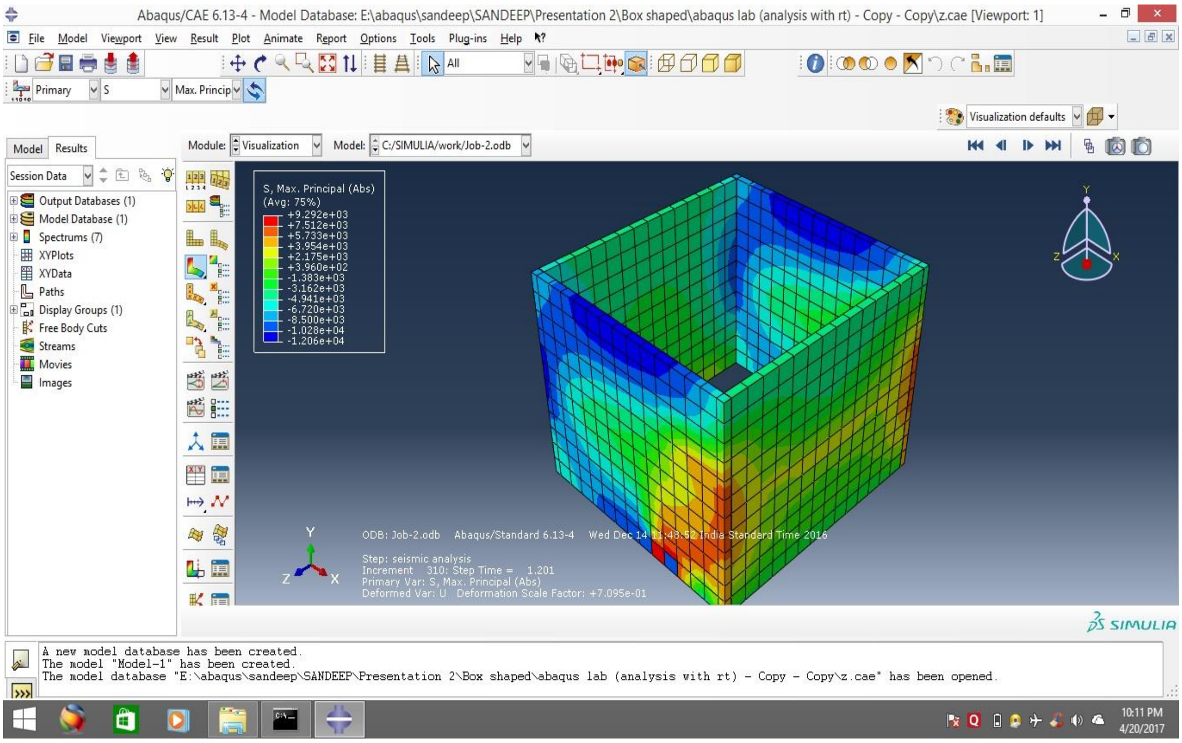
1. Corner crushing in the masonry infill wall.
2. Diagonal cracking.
3. Shear failure.
4. Out of plane failure.
5. In plane failure.

Two contour diagrams are given in next page to show the idea of stress concentration zones:

Contour Graphs: - For showing the stress concentration at some locations here two contour graphs are given.



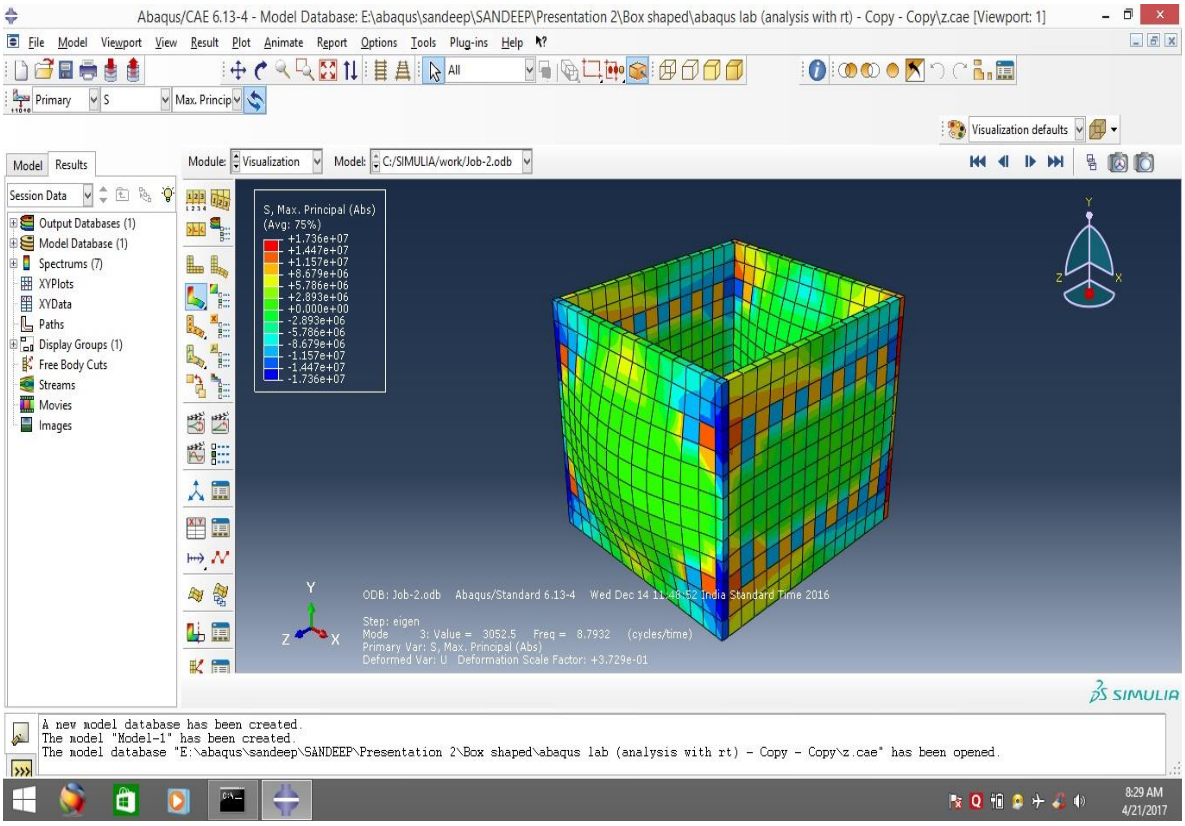
(i)



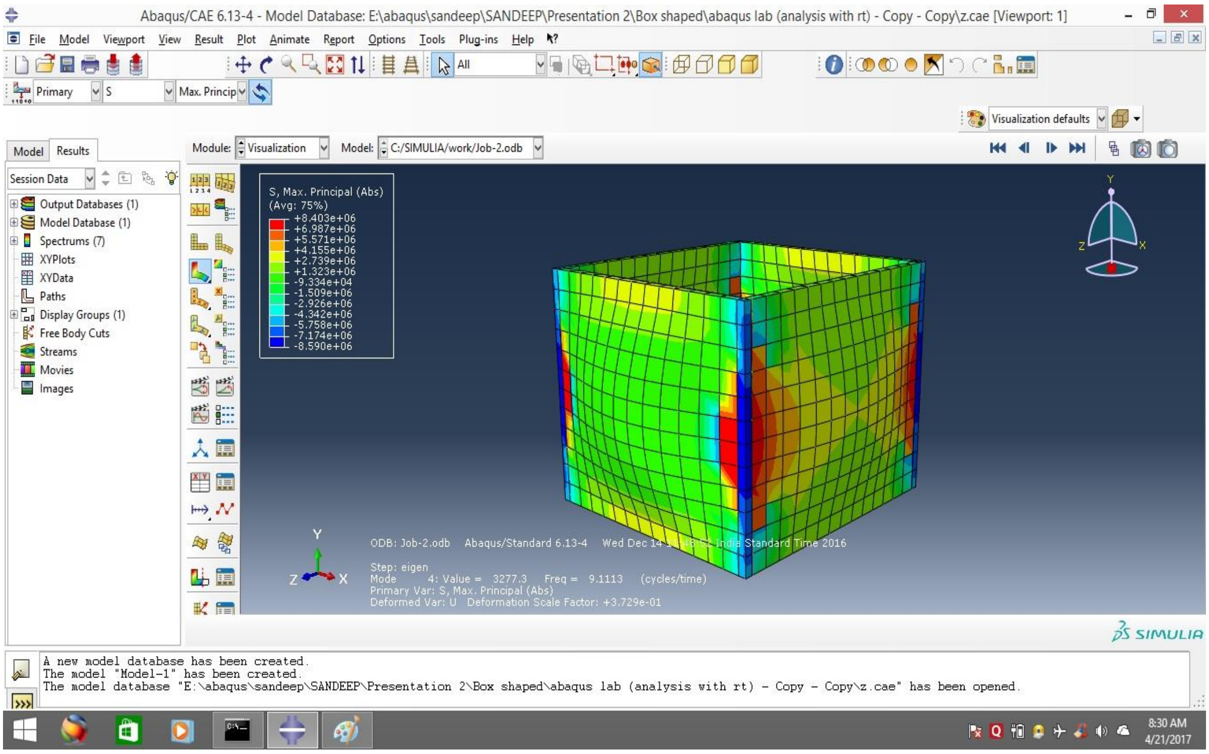
(II)

Graph 3: Contour diagram at different time

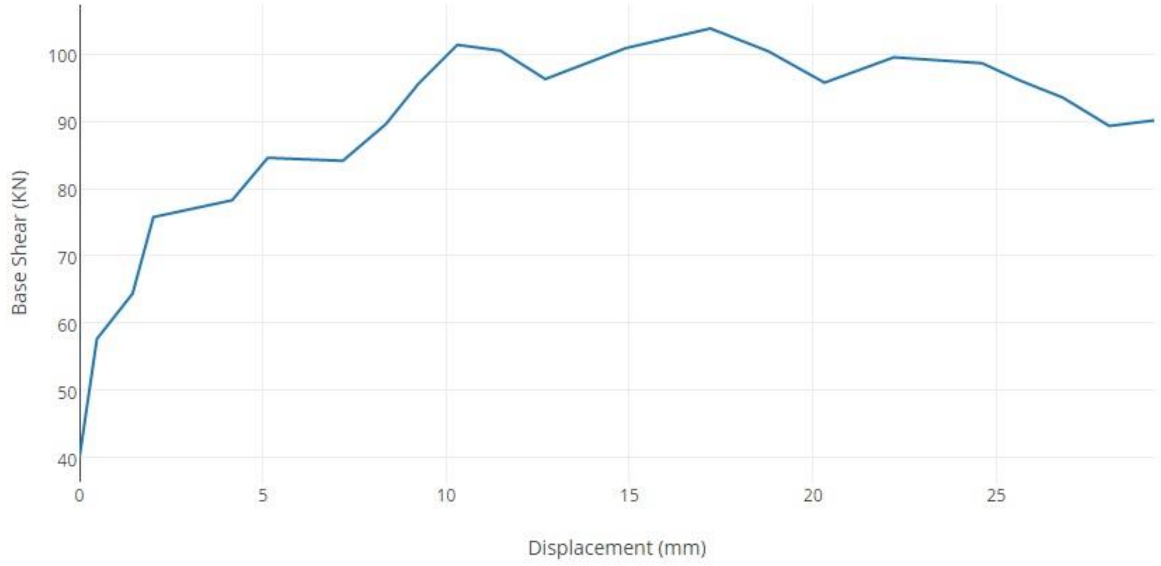
Graph 3: - For showing the out of plane failure of the masonry infill wall.



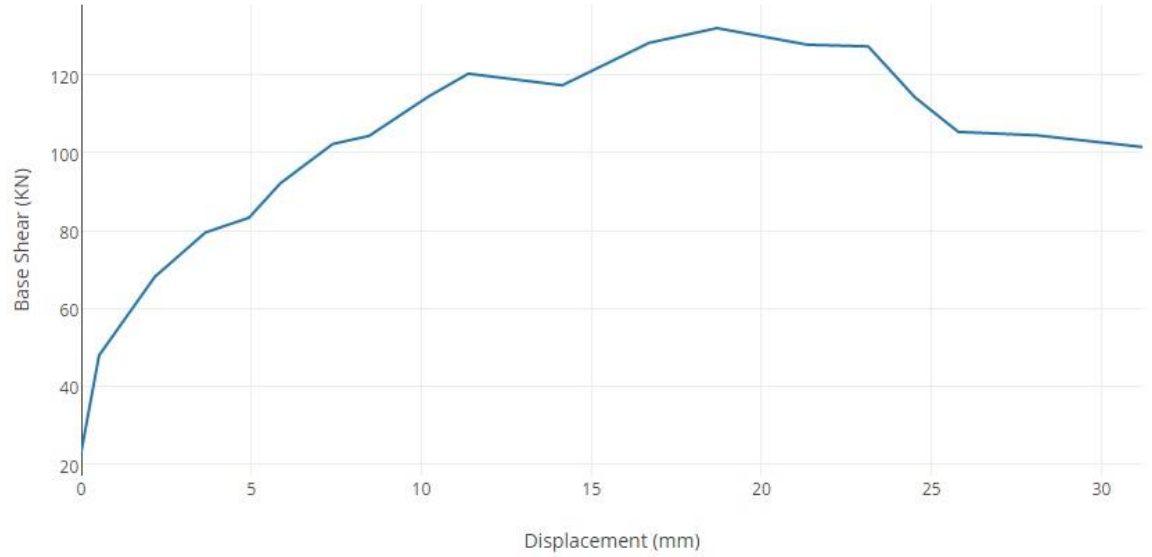
Graph 4: - For showing the in plane and out of plane failure of the masonry infill wall at ultimate failure point.



Resulted Graphs: -Base shear vs. Displacement



Graph 5: Load displacement curve for Case-1

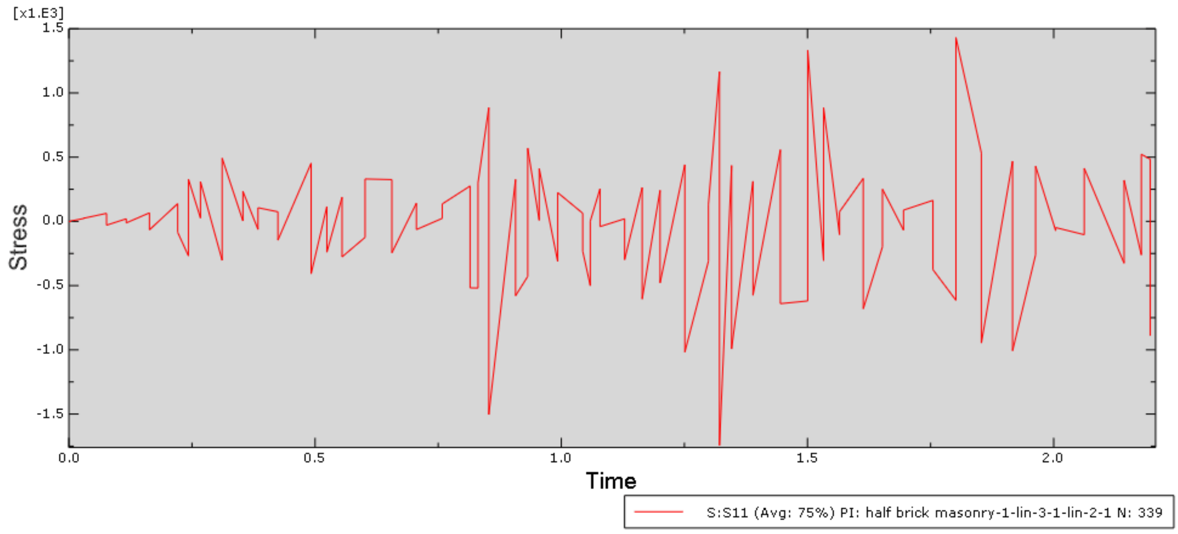


Graph 6: Load displacement curve for Case-2

As from the load displacement curve shown above it clearly shows that the:

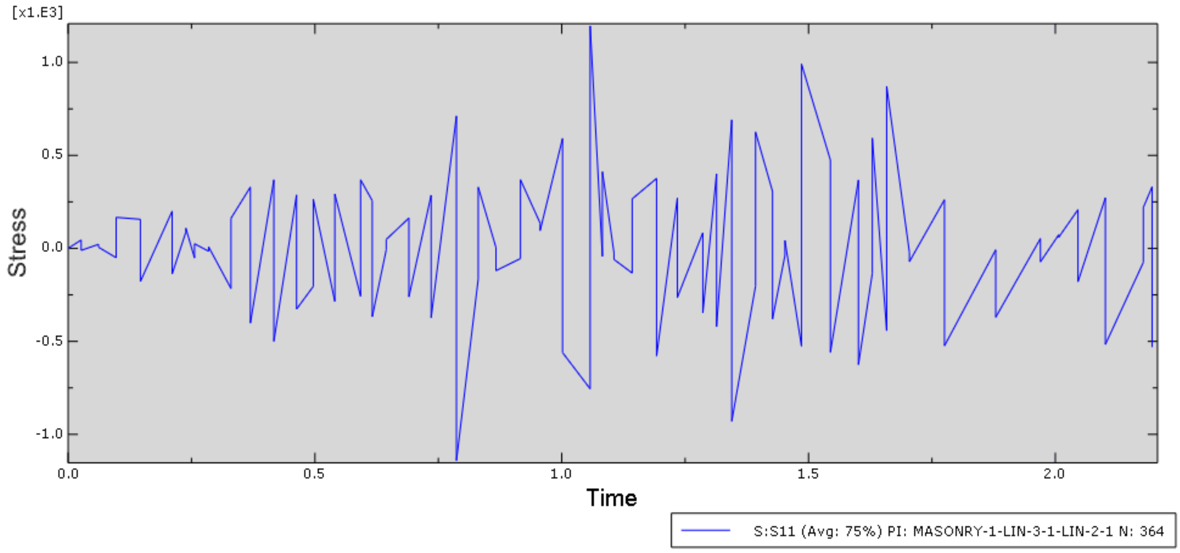
1. Curve is growing smoothly in case-2 while in the case-1 it is increasing with sharp increase and decrease.
2. In the case-2 lateral load carrying capacity improves and it is approximately 130KN and in the case-1 it is approximately 102KN.
3. Area under case-2 is more than case-1, simply implies that energy dissipation of case-2 is better.

Resulted Graphs: -Shear Force



Graph 7: Shear Force at Node No. 339

Analysis results have carried for both the cases. Both the graphs are carried out for a same node position in both cases. The first graph i.e. the case of masonry wall without reinforcement in the corner region has maximum Shear Force of 1750 N at a node 339. The maximum Shear Force is achieving at approximately 1.27 seconds.

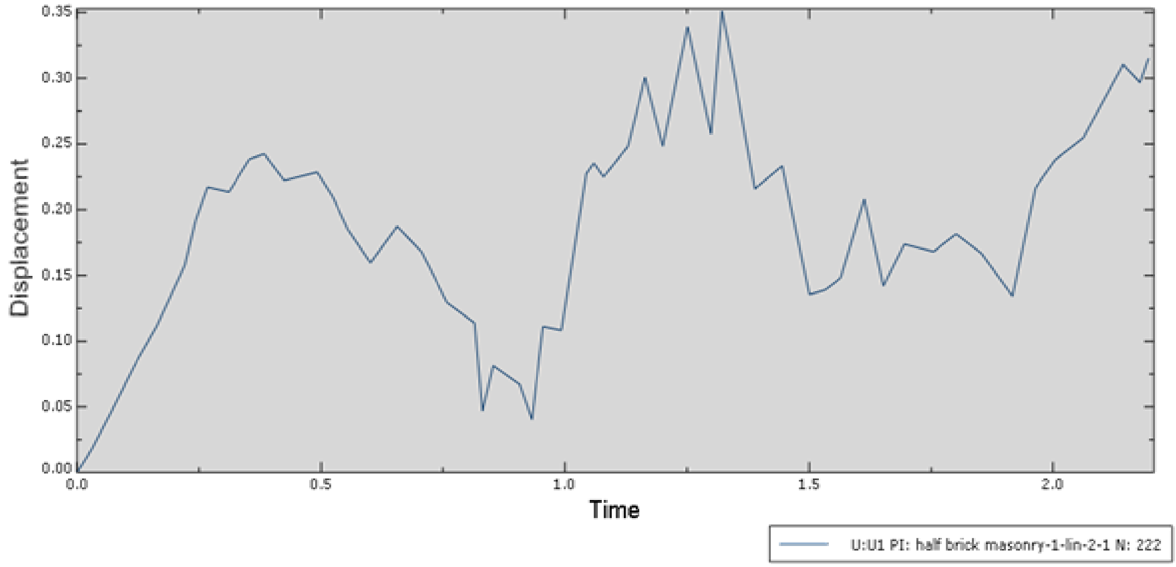


Graph 8: Shear Force at Node No. 364

The second graph i.e. the case of masonry wall with reinforcement in the corner region has maximum shear force of 1250 N at a node 364. The position of node 364 is same as the node

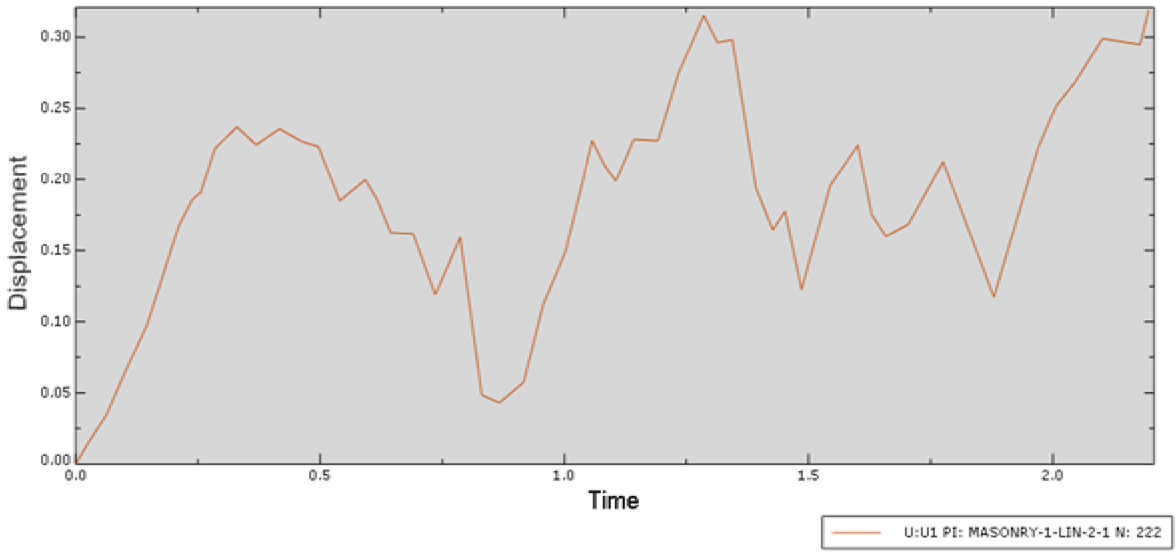
339 in the first case. In this case maximum shear force is achieving at approximately 1.07 seconds.

Resulted Graphs: -Displacement



Graph 9: Displacement at Node No. 222

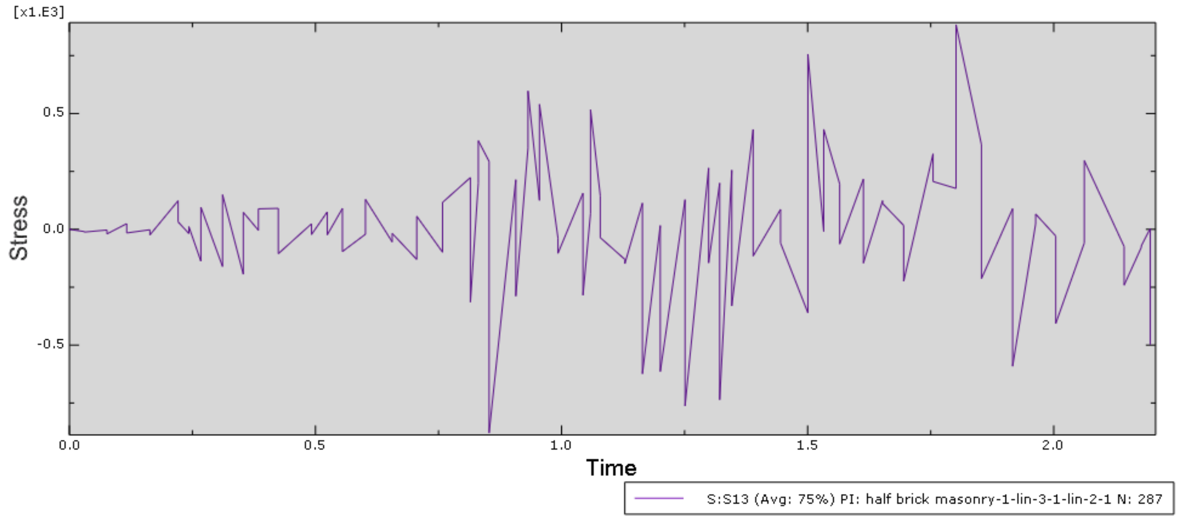
This graph of displacement is for the case of masonry infill without reinforcement at corner. The nature of the displacement is not much varied in both cases. Displacement resulting according to the graph shown above is .35m at approximate time of 1.3 seconds.



Graph 10: Displacement at Node No. 222

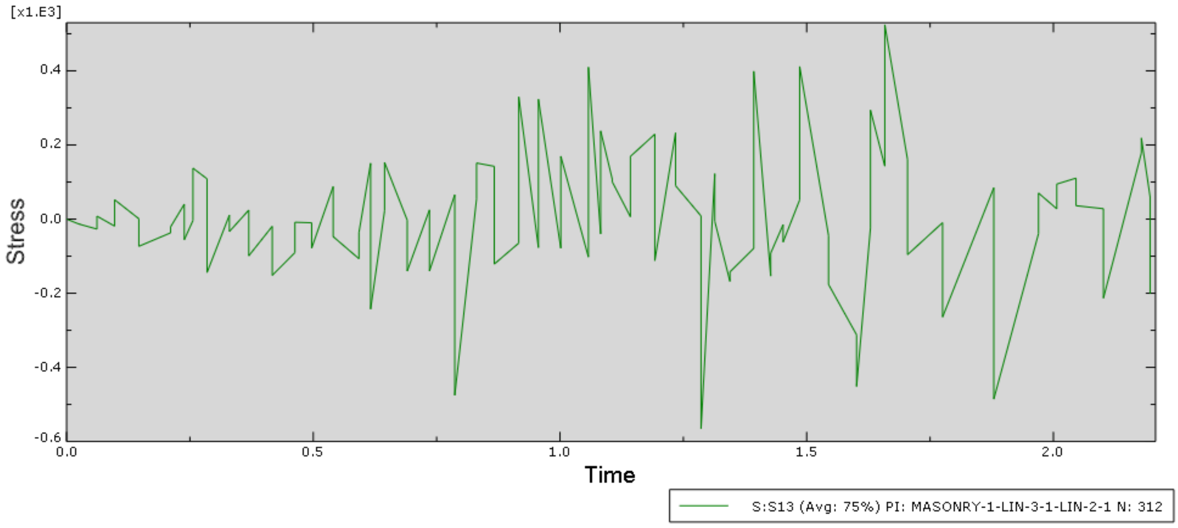
This graph of displacement is for the case of masonry infill with reinforcement at corner. The nature of the displacement is not much varied in both cases. Displacement resulting according to the graph shown above is .32m at approximate time of 1.28 seconds.

Resulted Graphs: -Shear Stresses



Graph 11: Shear Stress at Node No. 287

This graph of shear stress is for the case of masonry infill without reinforcement at corner. The nature of the shear stress varying is more at the mid height than at top and bottom of wall. According to this graph, maximum shear stress i.e. 800N/m² is occurring at approximate time of 1.76 seconds.



Graph 12: Shear Stress at Node No. 312

This graph of shear stress is for the case of masonry infill with reinforcement at corner. The nature of the shear stress is reducing in this case. The maximum shear stress is $600N/m^2$ at approximate time of 1.26 seconds for the same node position as in first case. Hence reduction in shear stress is observing while in second case.

Tabular Data (Case-1): - Stresses and Displacement at different Nodes.

Table 5: Stresses and displacement at different nodes.

EDGE-1	(CASE -1)							
Z-AXIS					**X-AXIS**			
BOTTOM	STATION	1	3		BOTTOM	STATION	1	3
	NODE	27	79			NODE	53	55
	S11	1745	579			S11	2729	1179
	S13	1084	880			S13	3909	3229
	S33	9203	10103			S33	544	416
	U1	0.00035	0.00035			U1	0.00035	0.00035
MID	NODE	33	85		MID	NODE	213	215
	S11	2384	1493			S11	6094	6392
	S13	1603	966			S13	1440	503
	S33	5882	7134			S33	1469	924
	U1	0.000332	0.000382			U1	0.000353	0.0003526
TOP	NODE	27	79		TOP	NODE	40	38
	S11	1745	579			S11	2580	1709
	S13	1083	880			S13	3929	3228
	S33	9203	10103			S33	545	414
	U1	0.00035	0.00035			U1	0.00035	0.00035
EDGE-2	(CASE -1)							
Z-AXIS					**X-AXIS**			
BOTTOM	STATION	1	3		BOTTOM	STATION	1	3
	NODE	339	287			NODE	108	106
	S11	1745	580			S11	2729	1179
	S13	1083	880			S13	3776	3230
	S33	9203	10103			S33	544	416
	U1	0.00035	0.00035			U1	0.00035	0.00035
MID	NODE	345	293		MID	NODE	224	222
	S11	2384	1493			S11	6094	6392
	S13	1603	966			S13	1444	503
	S33	5882	7134			S33	1469	924
	U1	0.000332	0.00034			U1	0.00035	0.00035

TOP	NODE	351	299		TOP	NODE	73	75
	S11	1815	645			S11	2538	1709
	S13	1083	880			S13	3905	3228
	S33	9131	10057			S33	545	414
	U1	0.00035	0.00035			U1	0.00035	0.00035

Tabular Data (Case-2): - Stresses and Displacement at different Nodes.

Table 6: Stresses and displacement at different nodes.

EDGE-1	(CASE -2)							
Z-AXIS					**X-AXIS**			
BOTTOM	STATION	1	3		BOTTOM	STATION	1	3
	NODE	52	104			NODE	41	43
	S11	1316	465			S11	1835	1022
	S13	834	635			S13	3265	3090
	S33	8591	7844			S33	425	351
	U1	0.00032	0.00032			U1	0.00032	0.00032
MID	NODE	46	98		MID	NODE	356	354
	S11	2494	1558			S11	6468	5307
	S13	1780	761			S13	1398	530
	S33	4643	6062			S33	2206	1466
	U1	0.000336	0.00035			U1	0.000318	0.000318
TOP	NODE	40	92		TOP	NODE	28	26
	S11	1195	497			S11	2049	944
	S13	833	634			S13	3242	3016
	S33	8706	8715			S33	425	351
	U1	0.00032	0.00032			U1	0.00032	0.00032

EDGE-2	(CASE -2)							
Z-AXIS					**X-AXIS**			
BOTTOM	STATION	1	3		BOTTOM	STATION	1	3
	NODE	364	312			NODE	64	62
	S11	1197	447			S11	2305	933
	S13	712	567			S13	3095	3374
	S33	8392	8436			S33	328	305
	U1	0.00032	0.00032			U1	0.00032	0.00032
MID	NODE	358	306		MID	NODE	224	222
	S11	2634	1501			S11	7015	1941
	S13	1876	758			S13	1379	475
	S33	5181	6403			S33	2397	736

	U1	0.000332	0.000352			U1	0.000322	0.000329
TOP	NODE	352	300		TOP	NODE	29	31
	S11	1212	499			S11	2668	1050
	S13	712	566			S13	3071	3396
	S33	8506	8534			S33	369	304
	U1	0.00032	0.00032			U1	0.00032	0.00032

Tabular Data (Result): - Percentage increase and decrease of stresses and displacement at nodes defined in last two tables for Case-1 and Case-2.

Table 7:%age decrease in stresses and displacement different nodes.

EDGE-1								
Z-AXIS					**X-AXIS**			
BOTTOM	STATION	1	3		BOTTOM	STATION	1	3
	NODE	27/52	79/104			NODE	53/41	55/43
	S11	24.6	19.7			S11	32.8	13.3
	S13	23.1	27.8			S13	16.5	4.3
	S33	6.7	22.4			S33	21.9	15.6
	U1	8.6	8.6			U1	8.6	8.6
MID	NODE	33/46	85/98		MID	NODE	213/356	215/354
	S11	-4.6	-4.4			S11	-6.1	17.0
	S13	-11.0	21.2			S13	2.9	-5.4
	S33	21.1	15.0			S33	-50.2	-58.7
	U1	-1.3	8.4			U1	9.9	9.9
TOP	NODE	27/40	79/92		TOP	NODE	40/28	38/26
	S11	31.5	14.2			S11	20.6	44.8
	S13	23.1	28.0			S13	17.5	6.6
	S33	5.4	13.7			S33	22.0	15.2
	U1	8.6	8.6			U1	8.6	8.6

EDGE-2								
Z-AXIS					**X-AXIS**			
BOTTOM	STATION	1	3		BOTTOM	STATION	1	3
	NODE	339/364	287/312			NODE	108/64	106/62
	S11	31.4	22.9			S11	15.5	20.9
	S13	34.3	35.6			S13	18.0	-4.5
	S33	8.8	16.5			S33	39.7	26.7
	U1	8.6	8.6			U1	8.6	8.6
MID	NODE	345/358	293/306		MID	NODE	224	222
	S11	-10.5	-0.5			S11	-15.1	69.6
	S13	-17.0	21.5			S13	4.5	5.6
	S33	11.9	10.2			S33	-63.2	20.3
	U1	0.0	-3.6			U1	8.1	5.9

TOP	NODE	351/352	299/300
	S11	33.2	22.6
	S13	34.3	35.7
	S33	6.8	15.1
	U1	8.6	8.6

TOP	NODE	73/29	75/31
	S11	-5.1	38.6
	S13	21.4	-5.2
	S33	32.3	26.6
	U1	8.6	8.6

To find the changes in both cases, values of displacement, shear force and shear stress is computed in the tabular form. And the results are satisfactorily at all nodes. Generally the behavior that comes to see accordingly the value given in table are:

- To increases in the strength.
- Decrease in the final deflections.
- Increase in the energy dissipation.
- Variation in the results of case-1 and case-2 are varying within range of 4%-28%.
- At some nodes result are not coming in positive way. This may be due to stress concentration at respective position.
- Maximum shear stress and shear force are coming in nodes at mid height.

Chapter-5

CONCLUSION

Based on the old researches, different modes of failure of masonry infill walls also considered and have been checked for the failure mechanism occurring here. From the analysis results, firstly it is concluded that as like possible mode of failures of masonry infill wall there is stress concentration taking place. This stress concentration is varying with time and taking place at different positions. Cracks initiation due to localized shear failure occurs in the different sections of model in the analyzed results. Also crushing of the masonry infill walls in the corner region occurring. The resulted failure i.e. crushing in corner region have not any particular well defined failure point just because after cracking the wall can take reversal loads and gravity loads. The in-plane and out of plane failure also capturing in the mode shapes of the model analyzed.

Instead of using equivalent strut system for the Macro modelling, a simple wall system is adopted. The behavior of elastic and deformation limits of masonry has assigned to this. And the expected general failures have captured in the final analysed results. So this is believable to consider the outputs of analyzed model.

The resulted graphs, contours and tables have clearly mentioned in the last chapter that placing of reinforcement in the corner region of masonry infill wall improves the connection behavior. As result of this improve connection system there is increase in the shear force, shear stress and energy dissipation of the system. Deflection limits at the corner regions also improving due to presence of reinforcements.

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