Non Linear Seismic Analysis of Masonry Structures

Divya C. Bose
Post Graduate Student, Department of Civil Engineering.
M. A College of Engineering
Kothamangalam
Kollam, India.

Dr. Mathews M. Paul
Associate Professor, Department of Civil Engineering
M. A College of Engineering,
Kothamangalam
Kothamangalam. India.

***Abstract*— Nowadays, even though many new construction techniques have been introduced, masonry has got its own importance in building industry. Masonry buildings of brick and stone are superior with respect to durability, fire resistance, heat resistance and formative effects. Because of the easy availability of masonry materials, economic reasons and merits, this type of construction is employed in rural, urban and hilly regions up to its optimum, since it is flexible enough to accommodate itself according to the prevailing environmental conditions. Masonry structures fail miserably under lateral loading conditions like earthquakes and impact loads. The occurrence of recent earthquake in India and in different parts of the world have highlighted that most of the loss of human lives and damage to property have been due to the collapse of masonry structures. Though an earthquake could not be prevented, the loss of life and property could be minimized, if necessary steps could be taken to reduce the damages on the existing masonry structures. This paper investigates the application of Nonlinear Seismic Analysis of masonry building using ANSYS software.**

Keywords—Earthquake, Masonry structures, Micromodelling, Finite element, Non Linear Static Analysis, Transient analysis

I. INTRODUCTION

Masonry buildings may be defined as the construction of building units bonded together with mortar. The units may be stones, bricks or precast blocks. Masonry buildings are constructed as massive structures and hence attract large horizontal forces during earthquakes. A number of the world's greatest earthquakes occurred in India in the last century. The occurrences of recent earthquakes in India and in different parts of the world result in losses, especially human lives that have highlighted the structural inadequacy of buildings to carry seismic loads. Severity of ground shaking, at a given location during an earthquake may be minor, moderate or strong. Relatively speaking minor shaking occurs frequently; moderate shaking occasionally and strong shaking rarely causes significant damages to masonry structures. Intensity of shaking at a location depends not only on the magnitude of the earthquake, but also on the distance of the site from the earthquake source and the geology/ geography of the area. Isoseismals are the contours of equal earthquake intensity. The area that suffers strong shaking and significant damage during an earthquake is termed as meizoseismal region.

Earthquakes occurred for millions of years and will continue to occur in the future as they have in the past. Some will occur in remote, undeveloped areas where damage will be negligible. Others will occur near densely populated urban areas and result in significant damages to inhabitants and the infrastructure. It is impossible to prevent earthquakes from occurring, but it is possible to mitigate the damages of a strong earthquake to reduce loss of life, injuries and damage.

An urgent need has been identified for assessment of the building in its present condition accounting for strength of component materials. IS13828-1993 recommends state that inclusion of special earthquake design and construction features may improve the earthquake resistance of the masonry structures and reduce the loss of life. To study the load deformation response it is important to analyze masonry structures in Non linear regime. This report mainly concentrates on the Nonlinear Seismic Analysis of masonry buildings.

Masonry buildings are the most common type of construction used for housing in Kerala. Post-earthquake surveys prove that the masonry buildings are most vulnerable and damaged significantly in the past earthquakes. Recently, it is observed that the frequency of occurrences of earthquakes in Kerala has increased. Though an earthquake could not be prevented, the loss of life and damage to property could be minimized by adopting a proper design. Steps can be suggested to reduce the damages to existing masonry structures. The present work illustrates the procedure for Non linear seismic analysis of masonry building using ANSYS software. The effects of openings in masonry structures have also been studied. The proposed method can be used to check whether retrofitting of the existing building is required or not.

II. EARTHQUAKES

A. General

Rocks are made of elastic material, and so elastic strain energy is stored in them during the deformations that occur due to the gigantic tectonic plate actions that occur in the Earth. But, the material contained in rocks is also very brittle. Thus, when the rocks along a weak region in the Earth's Crust reach their strength, a sudden movement takes place there opposite sides of the fault (a crack in the rocks where movement has taken place) suddenly slip and release the large elastic strain energy stored in the interface rocks.

The sudden slip at the fault causes *the earthquake*, a violent shaking of the earth when large elastic strain energy released spreads out through seismic waves that travel through the body and along the surface of the earth. And, after the earthquake is over, the process of strain build-up at this modified interface between the rocks starts all over again "Fig. 1,". Earth scientists know this as the *Elastic Rebound Theory*. The material points at the fault over which slip occurs usually constitute an oblong three-dimensional volume, with its long dimension often running into tens of kilometers.

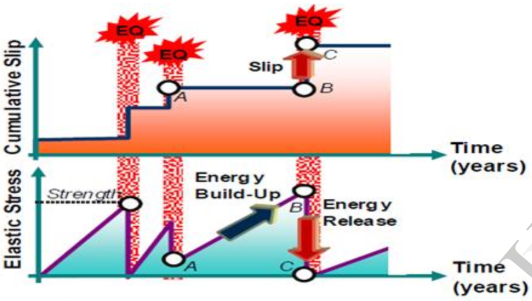

Fig. 1. Elastic Rebound Theory

Earthquakes subject the structure to a series of vibrations which cause additional bending and shear stresses in structural walls.

B. Sliding shear failure

Sliding shear failure, results in a building sliding off its foundation or on one of the horizontal mortar joints. It is caused by low vertical load and poor mortar. If the building is adequately anchored to the foundation, the next concern is for adequate resistance of the foundation itself, in the form of some combination of horizontal sliding friction and lateral earth Pressure. Sliding shear failure can also occur within the building structure, a classic case being the dislocation of a lightly attached roof.

C. Diagonal cracks

Due to diagonal cracks in masonry walls, when the tensile stresses developed in the wall under a combination of vertical and horizontal loads, exceed the tensile strength of the masonry material.

D. Failure due to Overturning

Failure due to overturning is caused by the effect of overturning moments. This may result in the building tipping over. The critical nature of the overturning effect has much to do with the form of the building's vertical profile. Buildings that are relatively squat in form are unlikely to fail in this manner, while those with tall, slender forms are vulnerable.

The tendency of a wall to topple when pushed in the weak direction can be reduced by limiting its length-to-thickness and height-to-thickness ratios. A wall that is too tall or too long in comparison to its thickness is particularly vulnerable to shaking in its weak direction.

E. Nonstructural failure

While structural elements of a building should be the prime concern for earthquake resistance, everything in the building construction should resist forces generated by earthquakes. Nonstructural walls, suspended ceilings, window frames and fixtures should be secure against movement during the shaking actions. Failure here may not lead to building collapse, but it still constitutes danger for occupants and requires costly replacements or repair.

Interior partitions, curtain walls, wall finishes, windows and similar building elements are often subjected during earthquakes to shear stresses, for which they do not have sufficient resistive strength. The most common damage resulting from this is breakage of window panes and cracks in internal plaster and external rendering. A possible remedy for the former is to isolate the window frames from the surrounding walls by the introduction of flexible joints; the latter can be avoided by reinforcing the plaster by introducing control joints (groves).

F. Site Failure

Site failures can also cause earthquake. Five common site failures that may occur during an earthquake. If significant in dimension site failures can cause damage to fences, retaining wall etc.

G. Foundation Failure

Site failures described above can cause damage to the building foundations. If the supporting ground moves, the foundations will move.

It is essential that the foundation system move in unison during an earthquake. When supports consist largely of isolated column footings in order to achieve this and to enable the lateral loads to be shared among all the independent footings

III. INDIAN SEISMIC CODES

Ground vibrations during earthquakes cause forces and deformations in structures. Structures need to be designed to withstand such forces and deformations. Seismic codes help to improve the behavior of structures so that they may withstand the earthquake effects without significant loss of life and property. Countries around the world have procedures outlined in seismic codes to help design engineers in the planning, designing, detailing and constructing of structures. An earthquake-resistant building has four *virtues* in it, namely:

(a) *Good Structural Configuration*: Its size, shape and structural system carrying loads are such that they ensure a direct and smooth flow of inertia forces to the ground.

(b) *Lateral Strength*: The maximum lateral (horizontal) force that it can resist is such that the damage induced in it does not result in collapse. (c) *Adequate Stiffness*: Its lateral load resisting system is such that the earthquake-induced deformations in it do not damage its contents under low-to moderate shaking. (d) *Good Ductility*: Its capacity to undergo large deformations under severe earthquake shaking even after yielding is improved by favorable design and detailing strategies. Seismic codes cover all these aspects.

The first forma seismic code in India, namely IS 1893, was published in 1962. The Bureau of Indian Standards (BIS) has the following seismic codes for masonry buildings: IS 1893 (Part I) : 2002, IS 4326 -1993, IS 13828-1993, IS 13920-1993, IS 13935-1993, and IS 1905 – 1987. These standards do not ensure that structures suffer no damage during earthquake of all magnitudes. But, to the extent possible, they ensure that structures are able to respond to earthquake shakings of moderate intensities without structural damage and of heavy intensities without total collapse.

IV. NONLINEAR SEISMIC ANALYSIS

The finite element model has become a paramount tool in the solution of a large number of problems in the physical and engineering sciences for the last 50 years. In the present analysis, the finite element approach is adopted for investigating the seismic behavior of brick masonry walls. Masonry walls can introduce changes in the dynamic characteristics of frames due to their features and their connection to the frames. The non linear seismic analysis of masonry structures is carried out using ANSYS 11.

A. Description of The Structure

(a) Building is located at zone V. According to MSK 64 (Medvedev – Sponhener - Karnik) Intensity scale (Annex I)

Zone	Area liable to shaking intensity
II	VI (and lower)
III	VII
IV	VIII
V	IX (and higher)

Four models were prepared for the non linear seismic analysis. Each model is subjected to a vertical load of **26.23kN/m** on the top of the wall.

1. **Model – 1:** Brick masonry wall of dimension **3.21 × 3 m**
2. **Model – 2:** Brick masonry wall of dimension **3.21 × 3m** and with an opening of size **1.5 ×1m** at the centre and a concrete belt around the opening.
3. **Model – 3:** Brick masonry wall of dimension **3.21 x 3m** with a door opening of size 2.1 x 1m at the centre and a concrete belt at the lintel
4. **Model – 3:** Brick masonry wall of dimension **3.21 x 3m** with a door opening of size 2.1 x 1m at the centre

B. Steps involved in the Analysis

Collecting material parameters like Young's Modulus, Poisson's ratio, density etc.

- Modeling the masonry structure
- Meshing the model.
- Applying the boundary conditions.
- Performing seismic coefficient method of analysis (Static method of analysis).
- Performing Transient Analysis.
- Comparing the Analytical results.

C. Size of structural elements

- Size of Slab = **100 mm**
- Thickness of masonry wall = **190mm**
- Size of one brick with nominal size 20×10×10cm = **19 × 9 × 9cm**
- Thickness of horizontal and vertical layers of mortar (1:6) = **10mm**
- Grade of concrete = **M30**
- Crushing strength of brick = **3.5N/mm^2**
- Crushing strength of mortar = **2.7N/mm^2**
- Thickness of shell = **500mm**
- Width of concrete belt at lintel level = **150mm**
- Width of concrete belt at sill level = **150mm**

D. Material Properties

Sl.No	Description	Young's Modulus (kN/mm^2)	Poisson's Ratio	Density (Kg/m^3)
1.	Brick	5	0.2	2100
2.	Mortar	2	0.15	2162
3.	Concrete	30	0.2	2400
4.	Steel	200	0.3	7850

E. Stress –Strain data of brick

Stress(MPa)	0	5	8	11	12.5	15
Strain	0.000	0.001	0.002	0.003	0.004	0.006

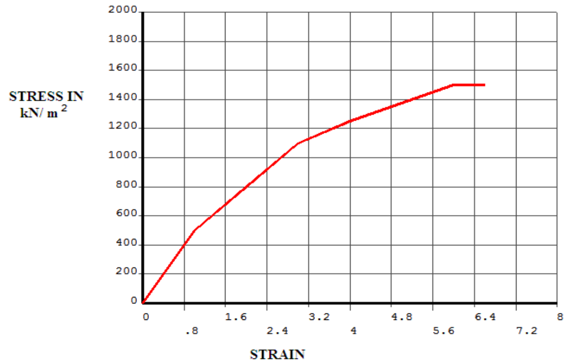

Fig. 2. *Stress- Strain curve of brick*

F. Modelling of masonry structures

The numerical modeling of masonry structures using FEM is computationally very demanding task because: (1)The typological characteristics of masonry buildings do not allow the use of simplified static schemes (2) The mechanical properties of the material lead to a widely non linear behavior whose prediction is very tricky. The finite element modeling of masonry is of two types (Lorenco et al, 2004):

In **heterogeneous modeling** the units and mortar are considered separately. This approach suits small size models. Because of the complexity of modeling the analysis cannot be performed in economical time ranges.

Homogeneous modeling can be applied for the large scale models. The masonry units, mortar elements are assumed to be smeared and they are assigned as an isotropic or anisotropic material. In this modeling it is necessary to have test results of large masonry part containing adequate number of units and mortar combinations.

The following modeling strategies can be adopted depending on the level of accuracy, simplicity desired and application field (1) Detailed **micro modeling**: Units and mortar joints are represented by continuum elements where as the unit brick interface is represented by discontinues elements. "Fig. 3," shows the detailed Micro modeling.

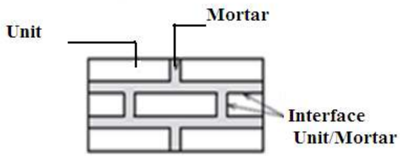

Fig. 3. Detailed Micro-modeling

(2)Simplified Micro modeling: Expanded units are represented by continuum elements whereas the behavior of the mortar joints and unit-mortar interface is lumped in discontinuous elements. These interface elements represent the preferential crack locations where tensile and shear cracking occur. "Fig. 4," shows the simplified micro modeling.

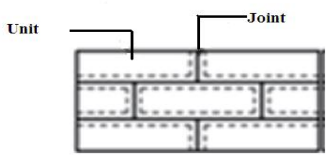

Fig. 4. Simplified Micro-modeling

(3) Macro-modeling units, mortar and unit-mortar interface are smeared out in the continuum. "Fig. 5," shows the Macro-modeling. Macro-modeling is more practice oriented due to the reduced time and memory requirements as well as user friendly mesh generation. This type of modeling is most valuable when a compromise between accuracy and efficiency is needed.

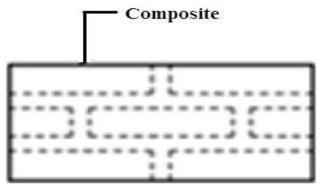

Fig. 5. Macro-modeling

The present work uses detailed micro modeling. The main advantage of detailed micro modeling is that almost all the failure modes can be considered. But it is not convenient for the modeling of whole structure, because the number of elements required can be huge, and consequently the cost of calculation time increase tremendously. Memory requirements are also very high (Lorenco1996: Loren co et al, 2004).

G. Solid 65

The element used for modeling the brick units, mortar and concrete is Solid 65. Solid 65 is used for the 3-D modeling of solids with or without reinforcing bars "Fig. 6". The solid is capable of cracking in compression. In concrete applications, for example the solid capability of the element may be used to model the concrete while the rebar capability is available for modeling reinforcement behavior. The element is defined by eight nodes with degrees of freedom at each node: translations in the nodal x, y and z directions. Up to three different rebar specifications may be defined. The most important aspect of this element is treatment of non linear material properties. The concrete is capable of cracking (in three orthogonal directions) , crushing, plastic deformation, and creep. The rebar can sustain tension and compression, but not shear. They are also capable of plastic deformation and creep.

The next step is to model the masonry wall and assign the properties and element type (ANSYS-11). The next step is meshing of the model.

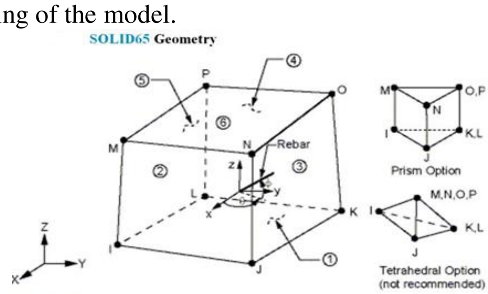

Fig. 6. Solid 65

V. SEISMIC COEFFICIENT METHOD OF ANALYSIS

Dynamic forces on multi-storied are best computed through a detailed vibration analysis. Detailed dynamic analysis or modal analysis or pseudo-static analysis should be carried out depending on the importance of the problem. BIS Code 1893 (Part I): 2002 recommend [Ref: Cl. 7:8:1]

Equivalent Lateral Force Method (Seismic Coefficient Method)

Seismic analysis of most structures is still carried out on the assumption that the lateral (horizontal) force is equivalent to the actual (dynamic) loading. This method requires less effort because, except for the fundamental period, the periods and shapes of higher natural modes of vibration are not required. The base shear which is the total horizontal force on the structure is calculated on the basis of the structure's mass, its fundamental period of vibration, and corresponding shape. The base end shear is distributed along the height of the structure, in terms of lateral forces, according to the code formula. Planar models appropriate for each of the two orthogonal lateral directions are analysed separately; the results of the two analyses and the various effects, including those due to torsional motions of the structure, are combined. This method is usually conservative for low- to medium-height buildings with a regular conformation.

Static method of analysis was performed to find the seismic load and its distribution.

A. *Building location and type of foundation*

Building is located at Zone- 5 and soil medium stiff and raft foundation is used.

B. *Load Calculations*

a) Live load on floor = $4\text{kN}/m^2$

Load area = $\frac{1}{2}(3.2+0.3)\times\frac{3}{2}\times 2$

= $5.25\,m^2$

Total live load on beam = 4×5.25

= $21\text{kN} = 6.56\text{kN}/m$

b) Dead load on floor = $12\text{kN}/m^2$

Load area =

$$\frac{1}{2}(3.21+0.3)\times\frac{3}{2}\times 2$$

= $5.265\,m^2$

Total dead load on beam = 12×5.265

= 63.18kN

= $19.68\text{kN}/m$

c) Total load (W) = $26.24\text{kN}/m$

C. *Design of seismic force*

Code permits seismic coefficient method for lateral load analysis for buildings less than 40m in height.

The base shear or total design lateral force along any principal direction shall be determined by the following expression:

$$V_B = A_h \times W$$

Where

V_B = Design base shear

A_h = Design horizontal acceleration spectrum value using the fundamental natural time period, T.

W = Seismic weight of the building.

The design horizontal seismic coefficient, $A_h = \frac{ZIS\,a}{2R\,g}$

Where, Z = Zone factor given in table 2, for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the MCE zone factor to the factor for Design Basis Earthquake (DBE)

I = Importance factor, depending upon the functional use of structures characterized by hazardous consequences of failure, post-earthquake functional needs, historical value or economic importance (Table 6 of IS 1893 (Part 1): 2002)

R= Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0. The values for buildings are given in Table 7 of IS 1893 (Part 1): 2002.

$\frac{Sa}{g}$ = Average response acceleration coefficient

Zone Factor

Seismic zoning assesses the maximum severity of shaking that is anticipated in a particular region. The zone factor (Z), thus, is defined as a factor to obtain the design spectrum depending on the perceived seismic hazard in the zone in which the structure is located. The basic zone factors included in the code are reasonable estimate of effective peak ground acceleration. Zone factors as per IS 1893 (Part 1): 2002 are given.

TABLE 1. ZONE FACTOR (Z)

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very Severe
Z	0.1	0.16	0.24	0.36

Importance Factor

The importance factor is a factor used to obtain the design seismic force depending upon the functional use of the structure.

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It is customary to recognize that certain categories of building use should be designed for greater levels of safety than the others, and this is achieved by specifying higher lateral design forces. Such categories are:

(a) Buildings which are essential after an earthquake-hospitals, fire stations, etc.

(b) Places of assembly-schools, theatres, etc.

(c) Structures the collapse of which may endanger lives-nuclear plants, dams, etc.

TABLE 2. THE IMPORTANCE FACTOR

Structure	Importance factor (1)
Important service and community buildings,, such as hospitals; schools; monumental structures; emergency buildings like telephone exchanges, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls; and subway stations, power stations	1.5
All other buildings	1

Response Reduction Factor

The basic principle of designing a structure for strong ground motion is that the structure should not collapse but damage to the structural elements is permitted. Since a structure is allowed to be damaged in case of severe shaking, the structure should be designed for seismic forces much less than what is expected under strong shaking, if the structures were to remain linearly elastic. Response reduction factor (R) is the factor by which the actual base shear force should be reduced, to obtain the design lateral force. Base shear force is the force that would be generated

Fundamental Natural Period

The fundamental natural period is the first (longest) modal time period of vibration of the structure. Because the design loading depends on the building period, and the period cannot be calculated until a design has been prepared, IS 1893 (Part l): 2002 provides formulae from which Ta may be calculated.

For a moment-resisting frame building without brick infill panels, Ta may be estimated by the empirical expressions

$T_a = 0.075h^{0.75}$ for RC frame building

$T_a = 0.085h^{0.75}$ for steel frame building

For all other buildings, including moment-resisting frame buildings with brick infill panels, T_a may be estimated by the empirical expression

$$T_a = \frac{0.09h}{\sqrt{d}}$$

Where h is height of building in meters (this excludes the basement storey's, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storey's, when they are not so connected), and d is the base dimension of the building at the plinth level, in meters, along the considered direction of the lateral force.

Seismic Base Shear

The total design lateral force or design seismic base shear (V_B) along any principal direction is determined by

$$V_B = A_h W = 0.09 \times 165 = 15 \text{ kN}$$

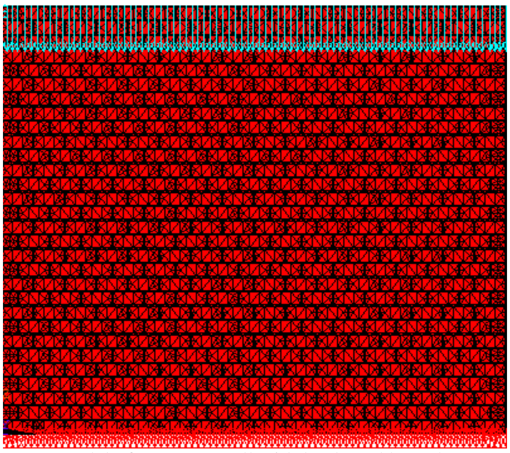

Fig. 7. FEM model of masonry wall with loads and boundary conditions (Model 1)

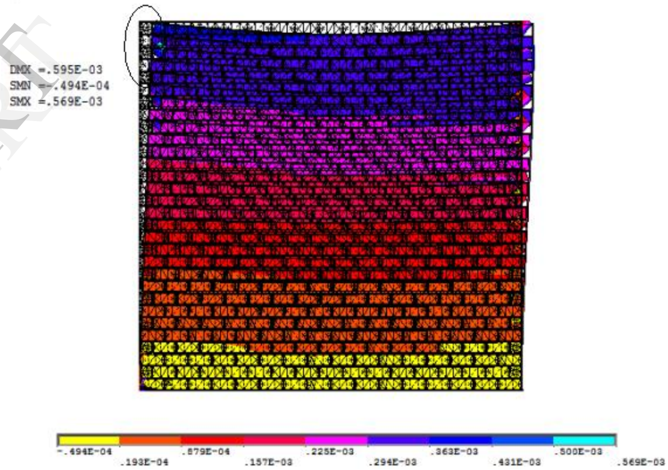

Fig. 8. Contour plot showing displacement (m) in X- direction

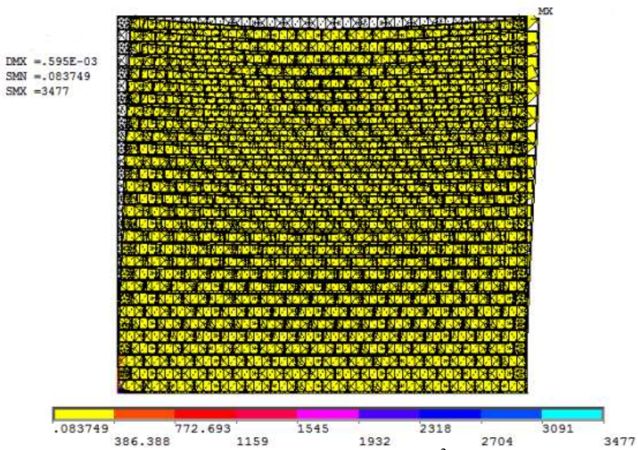

Fig. 9. Stress distribution diagram (kN/m^2) (Model -1)

TABLE 3. DISPLACEMENT – LOAD VALUES OF MODEL-1

Time in Second	**Displacement in x – direction (mm)**	**Load in (kN)**
0	0	0
0.01	0.144	1.5
0.02	0.201	3
0.03	0.256	4.5
0.04	0.301	6
0.05	0.398	7.5
0.06	0.411	9
0.08	0.484	10.5
0.09	0.522	12
1	0.595	12.5

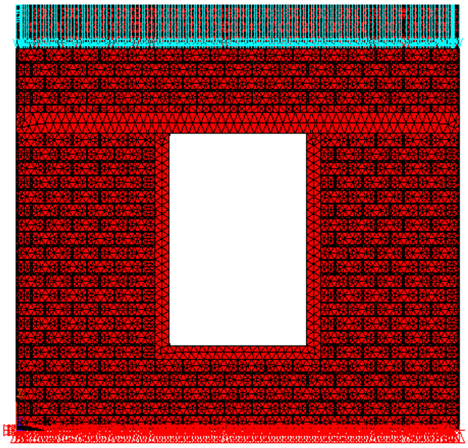

Fig. 10. *FEM model of masonry wall with loads and boundary conditions (Model 2)*

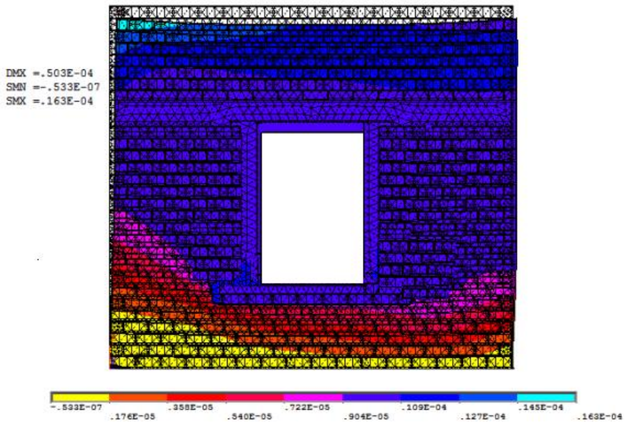

Fig. 11. *Contour plot showing displacement in X- direction*

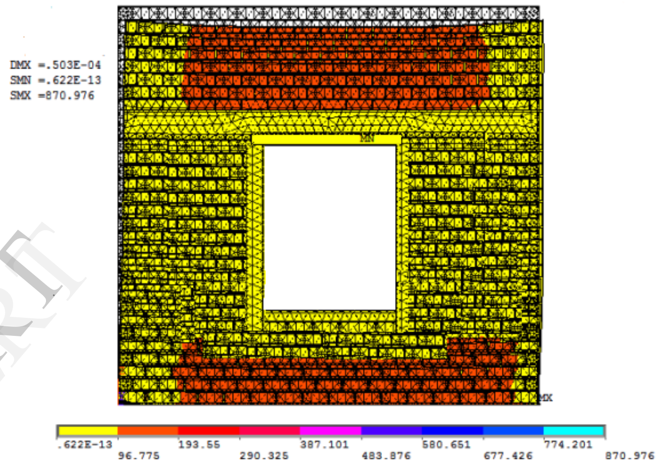

Fig . 12. *Stress distribution diagram (Model-2)*

TABLE 4. DISPLACEMENT – LOAD VALUES OF MODEL -2

Time in Second	**Displacement in X-direction (mm)**	**Load in (kN)**
0	0	0
0.01	0.0144	1.5
0.02	0.0234	3
0.03	0.0256	4.5
0.04	0.0311	6
0.05	0.0398	7.5
0.06	0.0423	9
0.08	0.0484	10.5
0.09	0.0501	12
1	0.0503	13

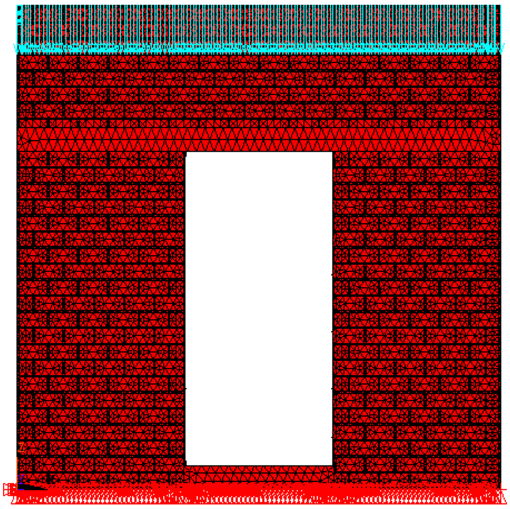

Fig. 13. FEM model of masonry wall with loads and boundary conditions (Model 3)

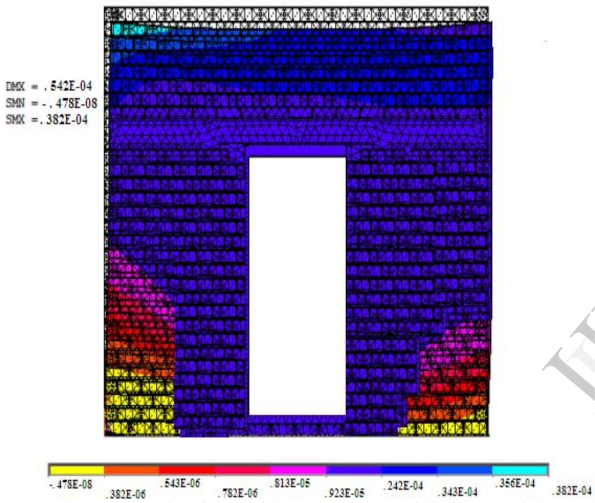

Fig. 14. Contour plot showing displacement in X- direction(Model-3)

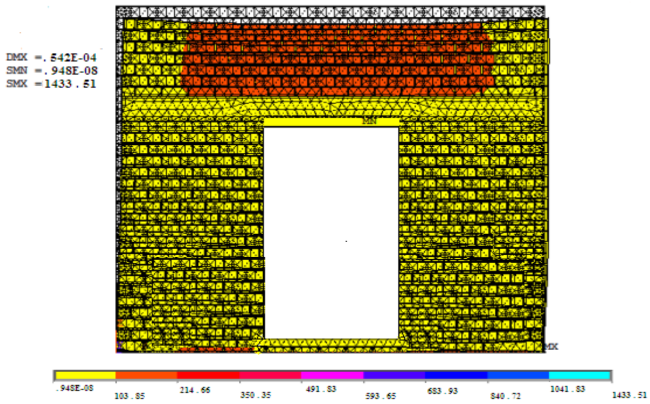

Fig. 15. Stress distribution diagram(Model -3)

TABLE 5. DISPLACEMENT – LOAD VALUES OF MODEL -3

Time in Second	Displacement in X-direction (mm)	Load in (kN)
0	0	0
0.01	0.0132	1.5
0.02	0.0225	3
0.03	0.0256	4.5
0.04	0.0322	6
0.05	0.0412	7.5
0.06	0.0432	9
0.08	0.0502	10.5
0.09	0.0514	12
1	0.0542	13

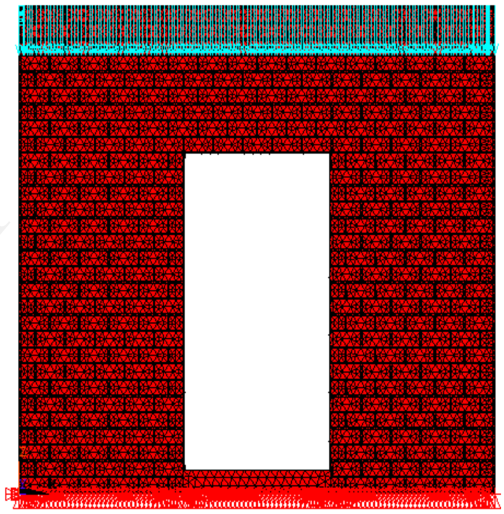

Fig. 16. FEM model of masonry wall with loads and boundary conditions (Model 4)

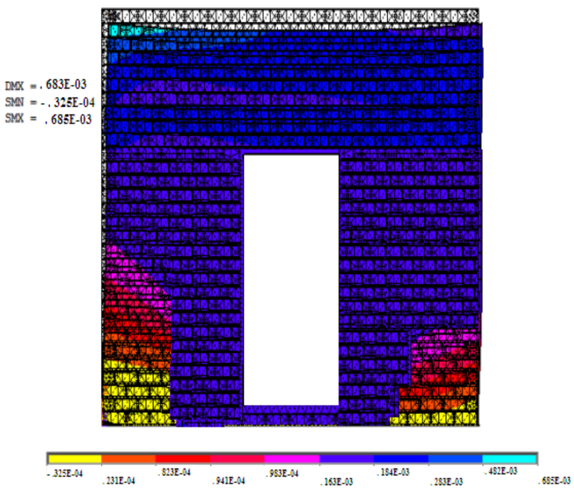

Fig. 17. Contour plot showing displacement in X- direction(Model-3)

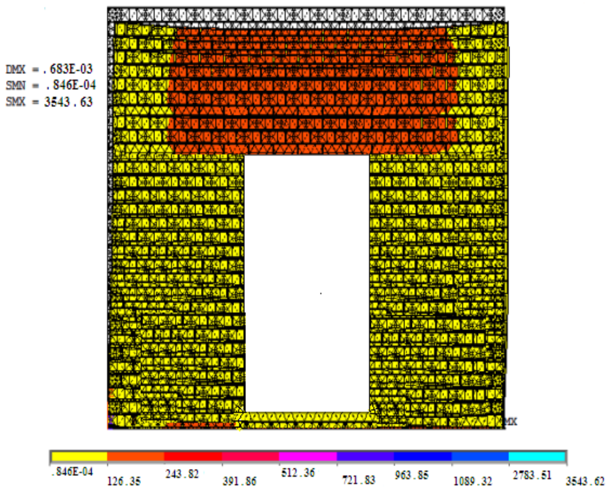

Fig .18. Stress distribution diagram (Model -4)

TABLE 6. DISPLACEMENT – LOAD VALUES OF MODEL -4

Time in Second	Displacement in X-direction (mm)	Load in (kN)
0	0	0
0.01	0.0149	1.5
0.02	0.0225	3
0.03	0.0334	4.5
0.04	0.0439	6
0.05	0.0525	7.5
0.06	0.0536	9
0.08	0.0612	10.5
0.09	0.0652	12
1	0.0683	13

VI. TIME HISTORY METHOD OF ANALYSIS

Time history and response spectrum are the two basic methods commonly used for the seismic dynamic analysis. The time history method is relatively more time consuming, lengthy and costly. The response spectrum method is relatively more rapid, concise and economical. However, time history method must be employed when geometrical and/or material nonlinearities are present in the system. Nowadays it is more convenient to use time-history method due to advances in computer hardware and software.

Transient dynamic analysis (or Time-History Analysis) is used to determine the dynamic response of a structure under the action of any general time dependent loads. This is used to determine the time varying displacements, stresses, strains and forces as it responds to any combination of static, transient and harmonic loads. The time scale of loading is such that inertial or damping effects are considered to be important.

Transient dynamic analysis in ANSYS is not too difficult. The geometry and finite element model is created in the usual manner in **PREP7** with loads and boundary conditions being applied in the **SOLUTION** phase. There are various types of analysis options such as **FULL, REDUCED, MODAL SUPERPOSITION.** Then the required datas are entered and finally the solution is activated method in ANSYS 11.

Once the static analysis is completed the next step was to carry out the transient analysis using as input, the acceleration- time data of earthquake. In this paper the May 18, 1940 EL Centro earthquake(or 1940 imperial valley earthquake) occurred at 21:35 pacific standard time on May 18 (05:35 UTC on May 19) in the imperial valley in south eastern southern California near the inter-national border of the united states and Mexico whose acceleration- time data was used. It had a magnitude of 6.9 and a maximum perceived intensity of X (Intense) on the Mercalli intensity scale. It was the first major earthquake to be recorded by a strong-motion seismograph located next to a fault rupture. The earthquake was characterized as a typical moderate-sized destructive event with a complex energy release signature. It was the strongest recorded earthquake to hit the Imperial Valley, and caused widespread damage to irrigation systems and led to the deaths of nine people.

A. Procedure

There are five main steps for performing transient dynamic analysis

(a) Build the model
(b) Choose analysis type and options
(c) Specify BC's and initial conditions
(d) Apply time- history loads and solve
(e) Review results.

Transient Analysis was done for two separate cases **(1)** In plane, where acceleration was applied to the base nodes in a direction parallel to the longer side of the wall**. (2)**Out of plane where acceleration was applied perpendicular to longer side of the wall. From the transient analysis it was observed that the maximum stress was obtained during the **24.34**th second of the earthquake in both the cases mentioned above and the corresponding acceleration was **0.92g**. "Fig. 19," gives the stress distribution diagram obtained from transient analysis of the two models. If the vertical load on the top of the wall is increased, the maximum equivalent stress developed on the wall increases. The circle indicates the position of the maximum equivalent stress developed on the masonry wall. This is the point where the first crack appears.

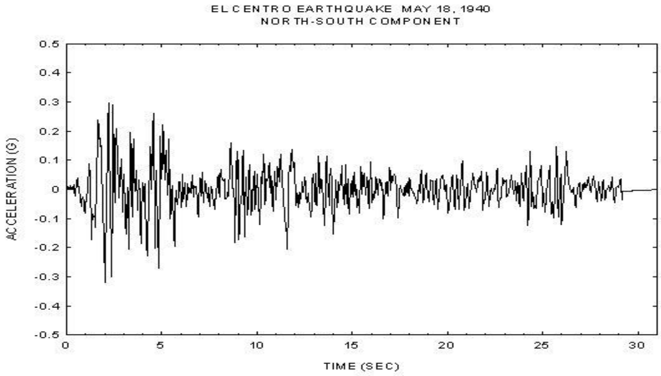

Fig. 19. Time- Acceleration datas of EL CENTRO earthquake

A. Acceleration in X- direction – In plane

In this case, the acceleration data of the EL CENTRO earthquake was applied to the base nodes of the masonry wall in a direction parallel to the longer side of the wall. Each model was subjected to magnitude of vertical loading on the top of the wall. The maximum equivalent stress was found to be developed during the **24.34** th second of the earthquake with a corresponding acceleration of **0.92g.** The stress details which includes the **X, Y**, and **Z** stress components, shear stress in **XY, YZ** and **XZ** planes and Von mises stress of two models respectively during the **EL CENTRO** earthquake

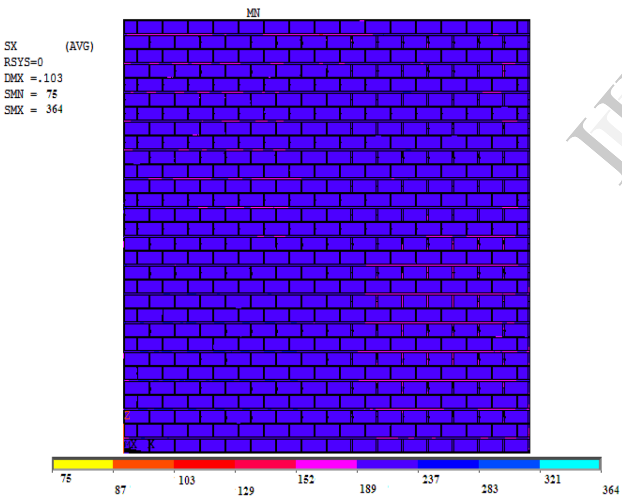

Fig. 20. Stress Distribution of Model -1

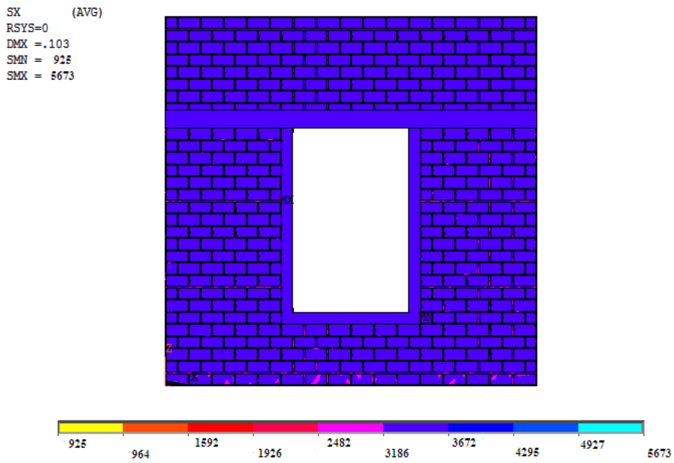

Fig. 21. Stress Distribution of Model-2

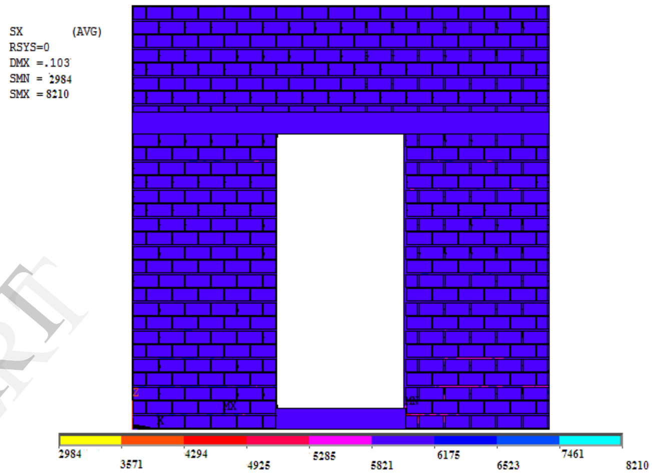

Fig. 22. Stress Distribution of Model-3

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TABLE 7. MAXIMUM STRESS DETAILS FOR IN PLANE MOTION

SL NO	TIME (S)	ACCELERATION (g)	STRESS (N/mm^2)			SHEAR STRESS (N/mm^2)			VON MISES STRESS (N/mm^2)
			X	Y	Z	XY	YZ	XZ	
Model 1 - Brick masonry wall of dimension 3.21 × 3 m									
1	24.34	0.92	0.325	0.354	0.235	0.356	0.178	0.025	0.364
Model 2 - Brick masonry wall with opening and a concrete beam around it									
2	24.35	0.92	2.234	2.674	1.584	1.081	0.396	0.037	5.630
Model 3- Brick masonry wall with door opening and a concrete beam around it									
3	24.35	0.92	4.523	4.962	3.821	2.345	1.952	1.267	8.210
Model 4- Brick masonry wall with door opening									
4	24.35	0.92	1.982	0.743	1.625	0.925	1.472	1.211	2.936

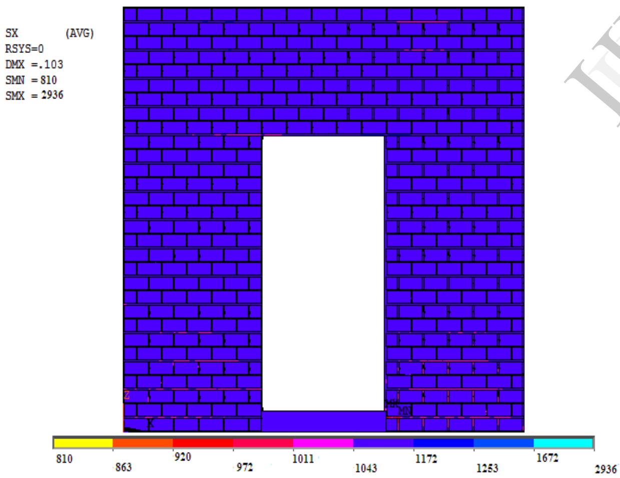

Fig. 23. Stress Distribution of Model – 4

VII. DISCUSSION OF RESULTS

The following points were observed:

Four models of masonry wall were prepared for the analysis. Each model was subjected to a vertical load of magnitude **26.24kN / m** on the top of the wall.
The duration of EL CENTRO earthquake was 31.18Seconds.The maximum stress was developed on the masonry wall during the **24.34** th second of the earthquake in all the four models and corresponding acceleration was 0.92g. Seismic Coefficient method and Transient analysis was carried out on different models. Firstly, the acc eleration of EL CENTRO earthquake was applied in a direction parallel to the longer side of the wall (In Plane).
From the results it is observed that the wall is more vulnerable to earthquake hitting perpendicular to its longer side than to the earthquake hitting parallel to its longer side.
The maximum stress was developed on left side of the wall near the base in the first model in In Plane case .In the second model and the third model, most of the stress is taken by the concrete beam around the opening. Here also, the maximum stress is developed on the right bottom corner of the concrete beam. In this model, the entire brick masonry portion around the concrete beam is protected. Only small magnitude of stress is developed on the brick masonry.
In the first model and the fourth model, the maximum stress developed in the in-plane case is 0.36N/mm^2 and 2.936N/mm^2 which is greater than 0.35N/mm^2 which is the maximum permissible crushing/compressive stress of brick masonry with mortar of 1:6 proportions. So the first model will subjected to damaged in the In Plane cases
In the second model and the third model , the maximum stress developed in the in plane is 5.63N/mm^2 and 8.210N/mm^2 which is less than the permissible Value of 15N/mm^2 .So the third model will remain undamaged if the above said EL CENTRO earthquake hits the building In Plane direction

VII. CONCLUSION

. Heterogeneous modeling gives more accurate results than homogenous modeling. But heterogeneous modeling is time consuming, lengthy and costly. The magnitude of the stress is large near the base of the wall and decreases towards the top of the wall. Earthquake wave hitting perpendicular to longer side of the wall is more vulnerable than that hitting parallel to the longer side of the wall. This is mainly due to the height to thickness ratio of the masonry wall. When the wave hit perpendicular to the longer side of the wall height to thickness ratio is much greater than when the wave hit parallel to the longer side of the wall.

In the first case, the maximum stress developed on the left bottom end of the wall and the magnitude is 0.36N/mm^2 for the In Plane case. The crack on model-1 appears to start from the left bottom end of the wall. The maximum permissible value of stress is 0.35N/mm^2 and the wall collapse in In Plane case. The strength of the wall can be increased by providing a protective concrete cover around the wall, we can prevent the damage on this wall.

In the second model the stress is concentrated near the corners of the opening in the wall. The maximum stress developed in the in plane is 5.63 N/mm^2 which is less than the permissible Value of 15N/mm^2 for concrete**.** The wall remains safe in In Plane case. Only a small magnitude of stress is developed on the brick masonry In the case of In Plane, Value of stress developed on the brick is only 33.673 $\times 10^{-6}\, N/mm^2$ **.** From the second model, it can be seen that provision of concrete beam around openings in the wall makes the existing unreinforced brick masonry safe against collapse.

In the third model the stress is concentrated near the corners of the opening in the wall. The maximum stress developed in the in plane is 8.210N/mm^2 which is also less than the permissible Value of 15N/mm^2 for concrete. The wall remains safe in In Plane case. Only a small magnitude of stress is developed on the brick masonry. From the third model also, it is seen that provision of concrete beam around openings in the wall makes the existing unreinforced brick masonry safe against collapse.

In the fourth model the stress concentrated near the corners of the opening in the wall. The maximum stress developed in the in plane is 2.936N/mm^2 which is greater than the maximum permissible value of 0.35N/mm^2 and the wall collapse in plane.

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