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Numerical Simulation of Masonry Walls Through Inclusion of Interface Elements

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Abstract

Numerical simulation in the field of masonry structures is gaining acceptance among researchers worldwide due to many downsides in large scale testing. The application of finite element method to the analysis of continuous or discontinuous system has received a significant interest in recent years. This paper focus on the application of discontinuities in the masonry walls through the inclusion of interface elements which helps to better understand the in-depth failure mechanism under seismic loading. Concept of simplified micro modeling strategies for masonry walls is incorporated here. Two-dimensional models subjected to dynamic load are developed first and then finite element computer program (FORTRAN) is written to implement on models. Brick is considered as linear solid element whereas joint as nonlinear element, incorporating the effects of discontinuities acting as planes of weakness, and nonlinear inelastic behaviour of the constituent materials. Main purpose is to permit the relative movements to occur between adjacent blocks and the transfer of shear stress across the interfaces. Opening or sliding of masonry under seismic loads are governed by the constitutive relation of stress and strain developed in the interfaces. This paper presents an efficient and realistic finite element analysis technique which shows great usefulness in analyzing discontinuous systems subjected to seismic loading. From this research, it is concluded that the technique applied for the analysis of masonry offers a more realistic alternative to an analysis that assumes masonry as continuum.

Keywords

Masonry – Discontinuities – Interface Elements – Simplified Micro Modeling – Nonlinear

1. Introduction

Masonry is one of the oldest building materials which is heterogeneous in nature and still widely used in construction. It is stronger in resisting and transforming compressive loads but weak in tensile demands. Building construction in Nepal dates back to several thousand years. Many houses, monumental structures and cultural heritage are masonry built. In addition to this, many masonry houses in Nepal are used by many generations without any strengthening measures which makes them highly vulnerable to even small earthquakes.

In 2015 Gorkha earthquake, major loss of lives happened due to the catastrophic destruction of masonry buildings. We found the news of casualty due to the falling of brick though the building was not collapsed. So, the major questions arise in this topic; Is it good to assume rigid connection between masonry blocks? Throughout the loading history, will connection between these blocks remains intact? If it would have been remained intact throughout the earthquake, no need of discontinuity analysis but at some point of loading history, there will be relative displacement of two mating surface. So, the importance of detail study and analysis of masonry is hereby reasoned.

Despite the recent advances in earthquake engineering and in the different fields of structural mechanics, there is still lack of reliable predictable techniques to access the vulnerability of masonry structures and to evaluate the adequacy of techniques for their retrofit. This paper focus on the development of analysis method and modeling technique for the masonry structure through the inclusion of joint or interface element. The influence of mortar joints has been considered by using interface elements to simulate the time-dependent sliding and separation along interfaces. The constitutive model of the interface elements reflects the limiting value of the shear stress/normal stress ratio at the interfaces. In general, this occurs at shear levels that are significantly lower than the limiting shear of the block material; consequently, an analysis that assumes perfect bonding at the interface could not accurately predict the shear transfer and leads to improper response of the structures. Thus, the actual dynamic behaviour of the system can be determined only by a non-linear analysis technique that accounts for the effect of these discontinuities on the response of the system.

2. Theoretical Background

Masonry is a complex material. It is a composition of bricks and mortar. The possibility of combining these elements with different qualities and geometry give masonry a wide range of alternatives of mechanical behaviour and structural performance.

2.1 Failure mechanism of Masonry

Unreinforced Masonry buildings are generally not good when it comes to lateral force resistance. Under the different loading conditions, many experimental studies have shown that joints or interfaces are the weakest zones of masonry structures. Some major masonry wall failure modes are shown in Figure 1.

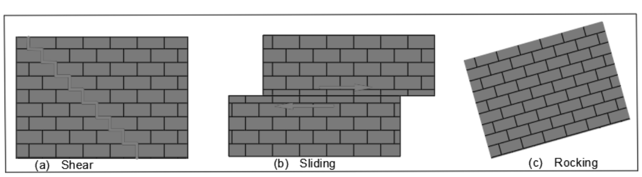

Figure 1: Major failure modes of masonry wall

- **Shear failure:** It occurs when the principal tensile stress, which developed in wall as a resultant of horizontal and vertical stress, exceeds the tensile resistance of masonry (Figure 1a).
- **Sliding failure:** In case of low vertical load and low friction at horizontal layers, crack forms at interface and slides between two layers occurs (Figure 1b).
- **Rocking failure:** In case of high moment/shear ratio or highly improved shear condition, failure may occur due to overturning and crushing of compressed toe (Figure 1c).

2.2 Modeling strategies of masonry

To minimize the enormous expenses of experiments and estimate the result of test, numerical analysis becomes necessary. Accurate modeling and analysis of masonry structures are still one of the challenging field to engineers because of its heterogeneous nature and nonlinear behaviour. Masonry structures have specific aspects and different numerical approaches are available for studying their behaviour. The three principle modeling strategies are corresponded to three different scales of complexity which have been identified by Lourenco and Rots [1]:

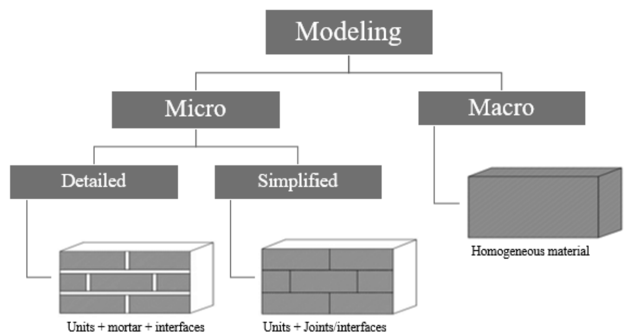

Figure 2: Principal modeling strategies of masonry

In detailed micro modeling the units (brick or blocks) and mortar are represented by continuum element whereas unit-mortar interface is represented by discontinuum element. Young's modulus of elasticity, Poisson's ratio and inelastic properties of both unit and mortar are taken into account and mortar interface is assumed as the plane of failure, thus enables the combined effects of units and mortar. In simplified micro modeling expanded units are represented by continuum element whereas behaviour of mortar joint and unit-mortar interface are lumped as discontinuum element. The mortar and unit mortar interface is lumped into average properties and units are extended to keep the geometry unchanged, thus masonry is considered as elastic block connected by potential crack joint. It has lesser accuracy as compared to first method since

Poisson's effect is not included. In macro modeling the units, mortars and unit-mortar interface are considered as homogeneous continuum element. It does not consider the individual behaviour of unit and interface and treats the masonry as homogeneous anisotropic continuum element. It is simple and easy, be applicable where compromises between accuracy and efficiency is needed.

3. Methodology

The main objective of this research is to implement the finite element program for the non-linear analysis of the discontinuous or jointed system. The program is written in FORTRAN 77 language and its extended standard. It uses the routine LAPACK[1] for the calculation of Eigen values and vectors. Basically, in the field of numerical modeling and simulation, we deal with following three main stages.

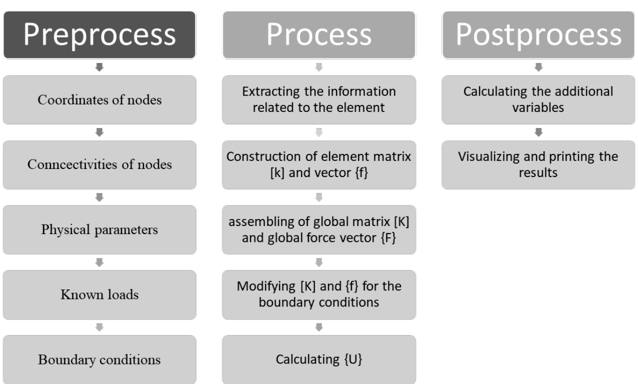

Figure 3: Stages of finite element program

In the preprocess part, the input data of the problem are read in and/or generated. This includes nodal coordinate, connectivity, boundary conditions, loading and material information.
The proprocess stage involves stiffness generation, stiffness modification and solution of equations resulting in the evaluation of nodal variables. Other derived quantities such as gradients or stresses may be evaluated at this stage.

The postprocess stage deals with the analyzing and visualizing results. Typically, the deformed configuration, mode shapes and stress distribution are computed and played at this stage.
A complete finite element analysis is a logical interaction of the three stages. For post processing, GiD[2] is used.

4. Modeling Discontinuities

For the efficient non-linear analysis of discontinuous systems, it is necessary to consider relative slip, debonding and cycles of closing and opening of the interfaces, since these can significantly affect the overall behaviour of the structure. To account for this behaviour, a special interface element has been proposed, together with some simplified constitutive laws to define its behaviour. Insertion of the interface element between the contact surfaces of a discontinuous system leads to the satisfaction of the compatibility condition at the interface and to the representation of the energy contributed by the discontinuous interface. Considering brick as solid elements and their interfaces as joint elements has been employed here. Geometric model is shown in Figure 4.

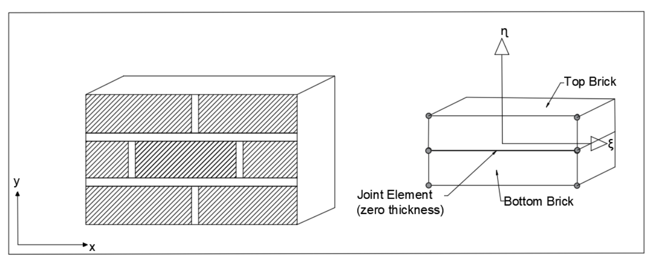

Figure 4: Geometric modeling of brick and joint element (4 node element)

The main purpose of this research is to permit the large relative movements to occur between adjacent blocks and the transfer of shear stress across the interface. The ultimate objective of dynamic analysis is to solve the widely known equation of motion:

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=-[M]\{\ddot{u}_{\mathrm{g}}\} \qquad (1)$$

[1]LAPACK is a freely-available software package. It is available from netlib via anonymous ftp and the World Wide Web at `http://www.netlib.org/lapack`.

[2]GiD is a universal, adaptive and user-friendly pre and post processor for numerical simulations in science and engineering. Copyright © 2017 · GiD · CIMNE

The numerical model is developed by making equivalent 4 node elastic solid elements for brick and 4 node joint elements for interfaces between bricks. The stiffness matrix (Equation 3) for solid element shown in Figure 4 can be formulated as derived by Chandraputla and Belegundu [2]. The displacement field can be represented as

$$\{u\} = [N]\{q\} \tag{2}$$

Where,
$\{u\} = \{u, v\}^T$ are global coordinates.
$\{q\} = \{u_1\ v_1\ \cdots\cdots\cdots\ u_4\ v_4\}^T$ are local coordinates.
$[N] = [N_1, N_2, N_3, N_4]$ are shape or interpolation functions.

After some mathematical manipulation, Element stiffness matrix for brick element is given by

$$[K]_s^e = \left[\int_{-1}^{1}\int_{-1}^{1} [B]^T [D] [B] \, det\,[J] \; d\xi d\eta\right] \tag{3}$$

The displacement of joint element depends upon relative movement of top and bottom solid elements (Figure 5), and corresponding stiffness matrix for zero thickness joint elements can be formulated from [3], [4] and [5] through Equations 5 to 8.

$$\{u\} = \{u_{Top}\} - \{u_{Bottom}\} \tag{4}$$

Then element stiffness matrix for joint element is given by

$$[K]_j^e = \left[\int_{-1}^{1}\int_{-1}^{1} [N]^T [K] [N] \, det\,[J] \; d\xi d\eta\right] \tag{5}$$

where,

$$[k] = \begin{bmatrix} k_s & 0 \\ 0 & k_n \end{bmatrix} \tag{6}$$

Here, k_s and k_n are components (shear stiffness and normal stiffness) of material property matrix $[k]$ of joint, $[N]$ and $[J]$ are shape function and Jacobian matrix, ξ and η are local coordinates. Normal and shear stiffness are calculated by regarding the wall as a series of two vertical springs, one representing the solid unit and the other representing the joint which leads to the following Lourenco [6].

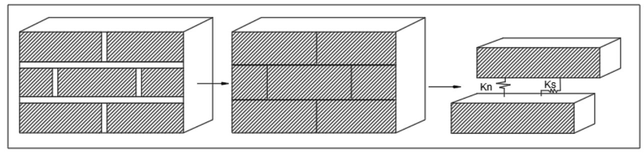

Figure 5: Interface model of brick masonry

$$k_n = \frac{1}{h\left(\frac{1}{E_{wall}} - \frac{1}{E_{unit}}\right)} \tag{7}$$

$$k_s = \frac{K_n}{2(1+v)} \tag{8}$$

Where, h is height of unit (average height of brick unit), E_{wall} is Young's modulus of elasticity of wall, E_{unit} is Young's modulus of elasticity of brick. Material properties needed for brick and cement mortar are taken from Pradhan P.L.[7]. E_{unit} is taken equal to 3.022E9 N/m^2 and υ is Poisson's ratio which value is taken equal to 0.15 and values of k_s and k_n for all three models calculated as per Equation 7 and 8 is shown in Table 1.

Table 1: Normal (k_n) and Shear (k_s) stiffness

Stiffness	Wall 1	Wall 2	Wall 3	Units
k_n	297302.82	110002.04	36667.35	tf/m^3
k_s	129262.1	47826.98	15942.33	tf/m^3

To get the damping matrix, the mass and stiffness proportional to Rayleigh damping has been used.

$$[C] = \alpha[M] + \beta[K] \tag{9}$$

in which α and β are proportionality constants selected to control the damping ratios of the lowest and highest modes expected to contribute significantly to the response. For the problem under consideration, the value of α was taken as 0.0 and following the value of β was taken as 0.24 according to Tzamtzis and Asteris [5].

4.1 Constitutive relationship

Joint is characterized as fully elastic, perfectly plastic and incapable of taking any tensile forces. The idealized constitutive relationship shown in Figure 6 has been used to denote the sliding and opening of joints.

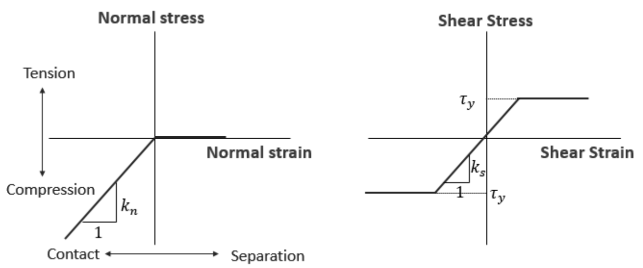

Figure 6: Constitutive relations for joints in normal(left) and shear (right)

- **Separation:** When normal strain; $\varepsilon_n > 0$, the joint cannot take any tensile stress and both act in the normal direction, the shear stiffness has also set to zero, consequently, shear or direct stress cannot be transmitted across the joint.
- **Contact:** When normal strain; $\varepsilon_n < 0$, normal forces are assumed to be restored corresponding to the normal stiffness of the joint.
- **Sliding:** Sliding occurs when $\tau > \tau_y$, i.e. the shear at joints exceeds the value given by the Mohr-Coulomb yield criterion:

$$\tau_y \ = c + \sigma_n \tan\phi \tag{10}$$

where, τ_y is yield shear stress, c is cohesion, σ_n is normal stress and ϕ denotes friction angle.

5. Preprocess

Three 2D wall models of different shape and sizes are taken for the analysis as shown in Figure 7. Brick is modeled as 4 node linear isotropic element and joint as 4 node elastic, perfectly plastic nonlinear element. All walls are masonry wall with cement mortar joint interfaces (zero thickness) and are fixed at the bottom and remaining nodes are free to move in any two directions. In wall 2, 1tonf is applied at the top level of the wall which is divided at all the nodes at top surface of wall. Description of various model taken is shown in Table 2.

Table 2: Wall descriptions

SN	Model	Description	Elements
1	Wall1	Brick size: 0.1mx0.037m Wall size: 0.3mx0.185m	Brick: 30 Joint: 36
2	Wall2	Brick size: 0.2mx0.1m Wall size: 1mx1m	Brick: 100 Joint: 135
3	Wall3	Brick size: 0.45mx0.3m Wall size: 4.95mx3m Window size: 0.9mx1.2m Door size: 0.9mx2.4m	Brick: 86 Joint: 137

The primary purpose of the analysis is to judge the appropriateness of method to evaluate the behaviour masonry wall under earthquake loadings regardless the properties of ground motion and local site effects. Here, 1940 El Centro earthquake data[3] is taken. For the analysis, only horizontal component of seismic record is considered. The PGA for 1940 El Centro earthquake in horizontal direction is 3.42 m/s^2 and its time history plot is shown in Figure 8.

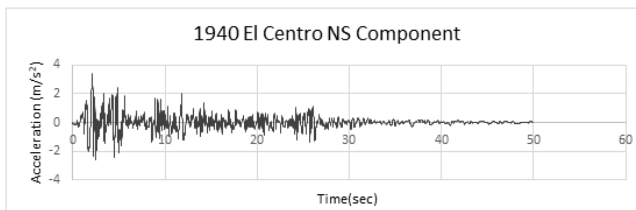

Figure 8: Seismic record of 1940 El Centro earthquake

6. Results, Discussion and Verification

Three wall models as described above were analyzed. Brick elements have been assumed linear and the focus is in the nonlinear deformation at joints.

6.1 Results

Various results are obtained from the analysis. All the models followed the constitutive mechanism showing sliding and opening behaviour in the joints. Table 3 shows the maximum displacement and acceleration recorded at the top ends of the node during the seismic loading for all the wall models. Maximum displacement recorded is for the wall 2.

[3]Source: https://ngawest2.berkeley.edu/

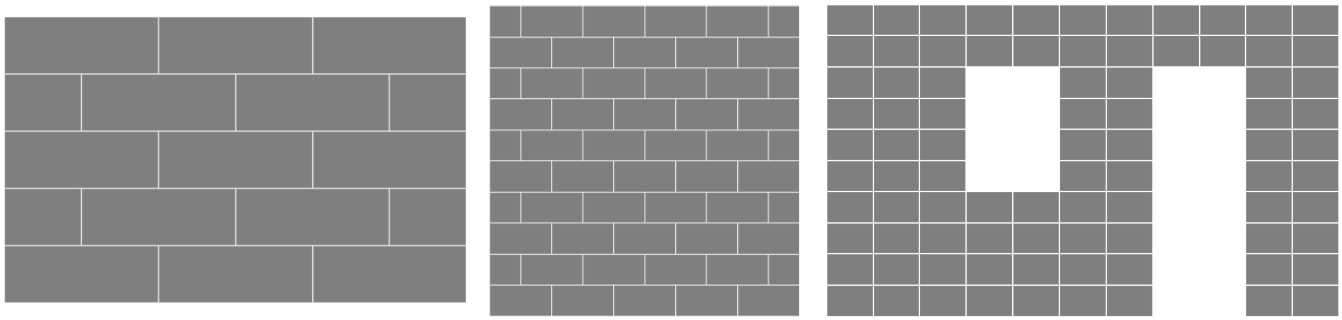

Figure 7: 2D Wall models (From left: wall 1, wall 2 and wall 3 respectively)

Table 3: Top node displacement

SN	Model	Node	Displacement (m)	Acceleration (m/s^2)
1	Wall 1	80	0.204	1.94
		94	0.203	1.42
2	Wall 2	281	0.498	1.53
		310	0.498	1.7
2	Wall 3	281	0.167	3.38
		310	0.201	2.34

Graphs of relative and absolute displacement of the top end nodes and plots of acceleration experienced by the top node and base node of all wall models for the total time steps (1600) of delta 0.01 sec are plotted in Figure 9 and Figure 10 respectively. Here we can see the nature of displacement (failure behaviour) for a single node recorded during 16 sec time period of 1940 El Centro earthquake.

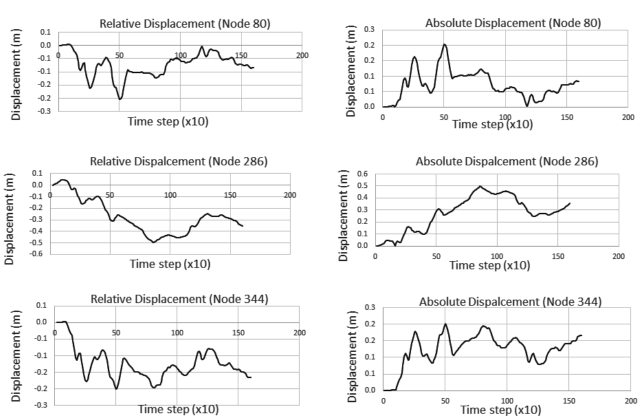

Figure 9: Relative and absolute displacement of walls (From top: wall 1, wall 2 and wall 3 respectively)

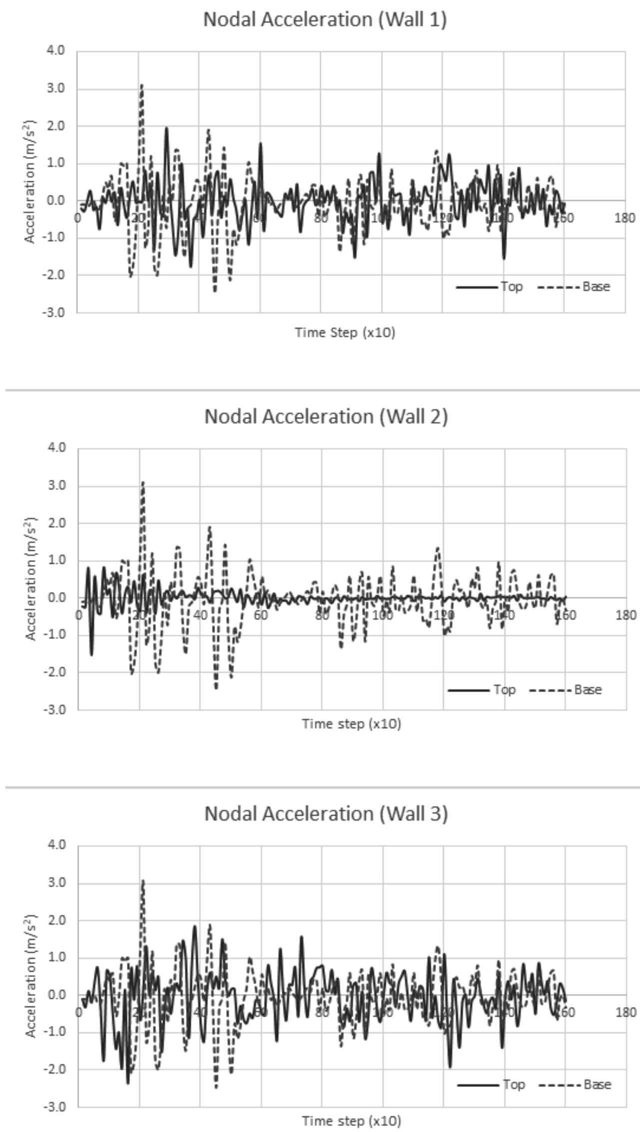

Figure 10: Acceleration measured at base and top nodes of walls

Simulated maximum horizontal displacement during seismic loading are shown in Figure 11. Colour bar at the left side of the figure shows the respective displacement.

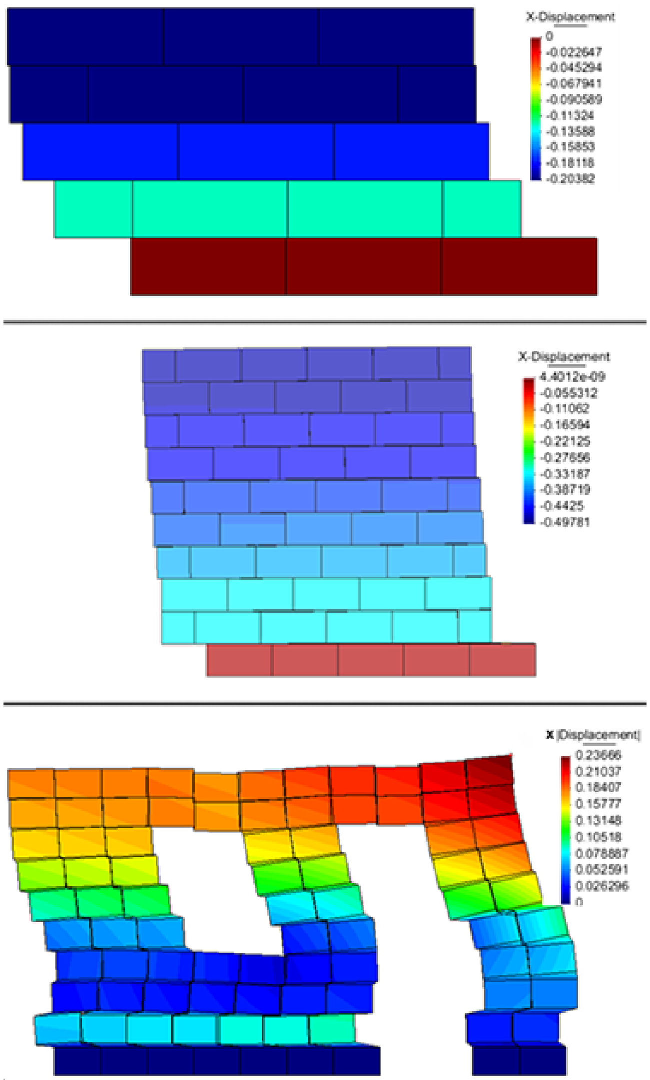

Figure 11: Maximum simulated displacement

6.2 Discussion

In this analysis, the program code is written for maximum number of 5000 iterations and if the number of iterations exceed the given limit, the code stops. Our aim is not to look whole collapse process. If we are looking for effectiveness, the deformation of such models under seismic loadings should be less than the few centimeters. Major discussions obtained are as follows:

1. **Displacement of top end node (Figure 9)**

   - Node experienced the first peak displacement of 0.163 m at 2.6 sec which is just after the PGA and as the yield strength or interlocking forces between the bricks decreases, even small seismic acceleration of 2.4 m/s^2 at 5 sec displaced the node to 0.2 m.
   - Node experienced the maximum peak at 8.6 sec of 0.498 m. Up to 8.6 sec, the relative displacement curve gradually increases. As the height increases, stiffness decreases, so it experienced higher displacement.
   - Here, we can see the gradual increment of displacement as the time increases. Maximum displacement experienced by this node is 0.201 m at 5 sec

2. **Acceleration measured at top and base nodes (Figure 10)**

   - Peak acceleration at the base and top node of the wall 1 is 3.04 m/s^2 and 1.94 m/s^2 respectively.
   - Peak acceleration at the base and top node of the wall 2 is 3.04 m/s^2 and 1.53 m/s^2 respectively. Here, top node experienced the acceleration early and after few secs, the graph is mild. Point to be noted, that in wall 2 we have assigned load of 1 tonf at the top of the wall i.e. in every delta time step vertical external load is applied at top.
   - Peak acceleration at the base and top node of the wall 3 is 3.02 m/s^2 and 2.34 m/s^2 respectively.

3. **Maximum simulated displacement (Figure 11)**

   - The maximum displacement measured is 0.203 m and minimum displacement measured is 0.022 m.
   - The maximum displacement measured is 0.498 m and minimum displacement measured is 0.055 m.
   - The maximum displacement measured is 0.236 m and minimum displacement measured is 0.026 m.

6.3 Verification

Using similar method, few studies have been done. This is a self-verified research through consecutive (hysteresis) relation.
Lets plot stress-strain obtained from wall 1 (element 60), as normal strain goes beyond zero, the joint cannot take any tensile forces and normal stiffness becomes zero. Here, no transfer of shear occurs, and separation starts up to the value of normal strain 0.01 and then the rebonding occurs when the normal strain becomes less then zero i.e. the forces are assumed to be restored.

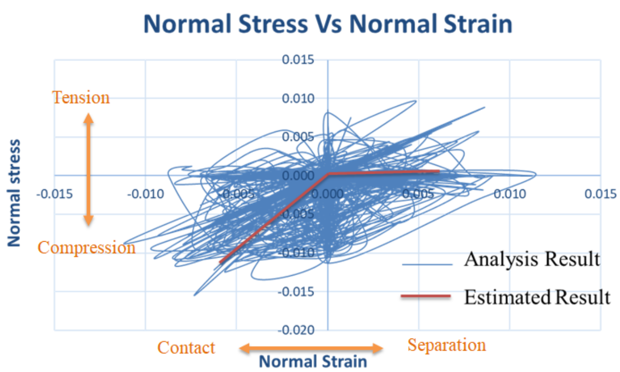

Figure 12: Normal stress vs normal strain of wall 1 (Joint element 60)

Similarly, lets plot shear stress-strain obtained from wall 1 (element 39). Up to 0.015 tonf/m^2 i.e. the yield stress, deformation occurs within elastic region. Beyond that yield stress, deformation goes into plastic region where the sliding occurs. Here, comparatively the yield stress value is low, but it is considerable as our test model is unreinforced and unconfined at the sides or top.

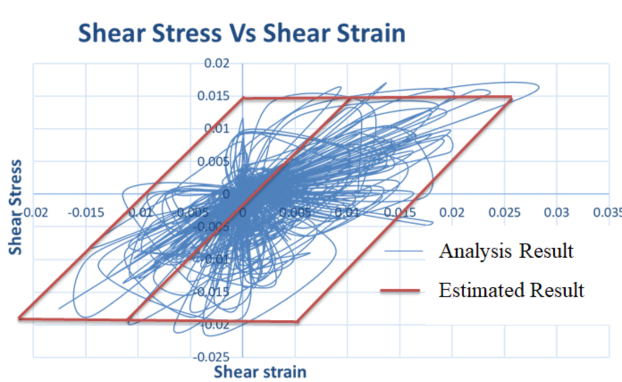

Figure 13: Shear stress vs shear strain of wall 1 (Joint element 39)

7. Conclusion

Two-dimensional masonry wall models are developed considering masonry as two phase (heterogeneous) material, thus allowing nonlinear deformation characteristics at the joints. For the influence of mortar joints, interface elements has been considered to simulate the time-dependent sliding and separation along the interfaces. The overall response of a discontinuous system to external loading is significantly affected by behaviour at the interfaces between the contacting materials, these discontinuities established the planes of weakness and exhibit non-linear inelastic behaviour, such as interface sliding, separation and contact. The accuracy and potential of the results obtained are verified with the constitutive relations. From Figure 12 and 13, it can be concluded that the program code developed for the analysis of these discontinuous structures through inclusion of interface elements is capable for such analysis. This simplified micro analysis shows the in-depth behaviour of masonry with detailed local failure mechanism where the continuum analysis fails to show the actual behaviour.

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