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The Dynamic Behavior of Reinforced Concrete Shear Walls

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Abstract—**The reinforced concrete buildings are subjected to lateral loads due to wind and earthquake and these forces are predominant especially in tall and slender buildings. In general, the structural response of shear wall strongly depends on the type of loading, aspect ratio of shear wall, size and location of the openings in the shear wall and ductile detailing (strengthening) around the openings of shear walls. The behavior of shear wall remains linearly elastic till certain level of loading; it may not be possible for a shear wall to behave in a same fashion throughout the loading history. Hence, in order to properly proportion and design the shear wall, it is of paramount importance to understand the behavior of shear wall, in linear as well as in non-linear regimes.**

Shear walls have been conferred as a major lateral load resisting element in a building in any seismic prone zone. It is essential to determine behavior of shear wall in the pre-elastic and post-elastic stage. Shear walls may be provided with openings due to functional requirement of the building. The size and location of opening may play a significant role in the response of shear walls. Though it is a well-known fact that size of openings affects the structural response of shear walls significantly, there is no clear consensus on the behavior of shear walls under different opening locations. The present study aims to study the dynamic behavior of shear walls under various opening locations using nonlinear finite element analysis using degenerated shell element with assumed strain approach.

Index Terms—Reinforced concrete, Shear wall, strength, Shell Element, Material Modeling

I. Introduction

Shear walls have been conferred as a major lateral load resisting element in a building in any seismic prone zone.Shear wallsmay be provided with openings due to functional requirement of the building. The size and location of openingmay play a significant role in the response of shear walls. Though it isa well-known fact that size of openings affects the structural response of shear walls significantly, there is no clear consensus on thebehavior of shear walls under different opening locations. The present study aims to study the dynamic behavior of shear walls under various opening locations using nonlinear finite element analysis.

In the 21st century, there has been the tremendous growth in the infrastructure development in the developing countries, especially India, in terms of construction of buildings, bridges and industries etc. This infrastructure development is mainly due to the growing population and to fulfill their demands. Since the land is limited, there is a huge scarcity of land in urban cities. To overcome this problem tall and slender multi-storied buildings are constructed. There is a high possibility that such structures are subjected to huge lateral loads. These lateral loads are generated either due to wind blowing against the building or due to inertia forces induced by ground shaking (excitation) which tends to snap the building in shear and push it over in bending. In the framed buildings, the vertical loads are resisted by frames only, however, the lateral resistance is provided by the infill wall panels.For the framed buildings taller than 10-stories, frame action obtained by the interaction of slabs and columns is not adequate to give required lateral stiffness [1] and hence the framed structures become an uneconomical solution for tall buildings. The lateral forces due to wind and earthquake are generally resisted by the use of shear wall system, which is one of the most efficient methods of maintaining the lateral stability of tall buildings.

Tall reinforced concrete buildings are subjected to lateral loads due to wind and earthquake. In order to resist these lateral loads, shear walls are provided in the framed structure as a lateral load resisting element [1–3]. Shear walls possess sufficient strength and stiffness under any loading conditions. The importance of shear wall in mitigating the damage to reinforced concrete structures is well documented in the literature [3, 4]. Shear walls are generally classified on the basis of aspect ratio (height/width ratio). Shear walls with aspect ratio between 1 and 3 are generally considered to be of squat type and shear walls with aspect ratio greater than 3 are considered to be of slender type. In general, the structural response of shear walls depends strongly on the type of loading, aspect ratio of shear wall, and size and location of the openings in the shear walls. Squat shear walls generally fail in shear mode whereas slender shear walls fail in a flexural mode.

The presence of openings in shear walls makes the behavior of shear wall slightly vulnerable under dynamic loading conditions. The structural analysis of the shear walls with openings becomes complex due to the stress concentration near the openings [5]. Various experimental investigations have been performed on shear walls with and without openings subjected to severe dynamic earthquake loading conditions [6].

Though reasonable studies have been made on the response analysis of shear walls with openings, very few literatures exist on the influence of opening locations on the structural response of shear walls [11, 12]. Hence, it is essential to study the influence of opening locations on the structural response of RC shear walls. The present study analyses the response of RC shear walls for different opening locations for both squat and slender shear walls.

II. Literature Review

Literature survey interprets old information and generates a combination of new information with old information. So, in this section there is a brief description of various research papers and occurrence of summary and synthesis of research papers.

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For the analysis of shear wall, several analytical methods have been proposed by various researchers which range from simplified conventional approach to the sophisticated finite element approach. Due to the complexity of numerous factors which influence the overall behavior of RC shear walls, the validity of modeling and analysis techniques could only be established by comparing the same with experimental results. In this chapter several experimental and analytical investigations are presented pertaining to the assessment of the shear walls of different aspect ratios with and without openings and subjected to different loading conditions.

During the past few decades, efforts have been directed towards the development of effective analytical techniques that are able to model the behavior of shear walls adequately. Simplified methods have been proposed by various researchers in the past: the simplified methods such as equivalent column model, lumped plasticity models, equivalent frame model, Rosman- approach, method of relaxation etc are quite popular among the engineering fraternity. However, these simplified models are applicable only to shear wall with regular geometry and with linear elastic behavior. On the other hand, finite element method is capable of analyzing shear wall of irregular geometry subjected to loads varying with time in the linear as well as non-linear regimes.

Lindeburg and Baradar (2001) developed the simplified hand calculation method to analyze the shear walls with openings with varying assumptions (Lindeburg and Baradar, 2001). The accuracy of this simplified method was checked by Neuenhofer (2006) using the linear elastic finite element model of shear wall with conventional four-noded plane stress elements and observed that simplified method consistently underestimates the impact of the openings on the stiffness reduction, thus producing a lateral stiffness larger than that obtained using finite element analysis. Moreover, simplified methods have been found to produce remarkably poor results for shear walls with small aspect ratios where shear deformation controls the structural behavior (Neuenhofer, 2006). Hence, a more versatile method of analysis like finite element method is sought for the analysis of shear wall with varying geometry and subjected to different loading conditions.

Quadrilateral elements with quadratic functions using iso-parametric approach are considered superior to model the 2-D structures because of its high accuracy in analysis. Iso-parametric elements are the class of elements, which are more arbitrary in shape and effectively used to model the curved geometry of the structure (Ahmad et al. 1970). Moreover, the iso-parametric elements are widely used in many structural applications related to plates and shells. Franklin (1970) used special frame-type elements, quadrilateral plane stress elements, axial bar elements, two-dimensional bond links with advanced non-linear capabilities to analyze the behavior of RC frames coupled with shear walls. Due to its simplicity, the use of plane stress elements are popular and used by numerous investigators to study the behavior of RC frame and wall systems. Cervenka (1970) analyzed the shear walls using plane stress elements and proposed a constitutive relationship for the composite concrete-steel material for uncracked, cracked and plastic stages. Nayak and Zienkiewicz (1972) also conducted the investigation on the elasto-plastic behavior of concrete in compression using plane stress finite elements. Dotroppe et al. (1973) used a layered finite element procedure in which slab elements were divided into layers to account for the progressive cracking through the slab thickness. Scanlon and Murray (1974) used layered rectangular slab elements to analyze the slab by incorporating both cracking and time-dependent effects of creep and shrinkage in slabs. Lin and Scordelis (1975) used layered triangular finite elements in RC shell analysis by incorporating the membrane and bending effects, as well as the tension stiffening effect of concrete between cracks in the model.

The use of shell elements to model moderately thick structures like shear wall is well documented in the literature (Dvorkin and Bathe, 1984). Nevertheless, the general shell theory based on the classical approach has been found to be complex in the finite element formulation. In order to reduce the number of nodes, it was proposed to use degenerated shell element with nodes situated only at the mid plane of the element (Ahmad et al. 1970). The degenerated shell element derived from the three-dimensional element, has been quite successful in modeling moderately thick structures because of their simplicity and circumvents the use of classical shell theory. However, in the case of thin shells, the shear and membrane locking appeared to be disturbing the solutions.

Massone et al. (2004) conducted experimental investigations on four RC shear wall models, scaled approximately to 1/4th of the size of the actual shear wall. Out of four shear wall models, two were of rectangular cross-sections and remaining two was of T-shaped. The shear walls were 3.66 m (12 ft) tall and 101.6 mm (4 in) thick with web and flange widths of 1.22 m (4 ft). The floor slabs were provided at 0.91 m (3 ft) intervals over the height of the shear walls. The special boundary elements were provided over the bottom 1.22 m (4 ft) of each shear wall. Based on their investigations, it was revealed that the coupling between the inelastic flexural and shear deformations exist despite the shear wall having the shear strength of approximately two times the demand.

Lee (2015) tested seven shear wall panels of size (width × height × thickness) 1200 mm × 1200 mm × 40 mm to assess the strength and deformability of the shear wall in the presence of window and door openings. The reinforcement ratio in the vertical as well as horizontal direction was kept at 0.31% as per the Australian Standards for Concrete Structures (AS3600-2009). In order to prevent shrinkage cracking, reinforcing bar strip of small length were diagonally placed at the corners of the opening. He observed that the size of the openings significantly affects the structural response of the shear wall.

III. Material Modelingof RC Structures

The material modeling of RC structures has been the subject of interest for many decades, because incorrect modeling results in poor characterization of its behavior. To cater to the increased demand for seismic design of RC structures, many experimental and analytical studies to capture the nonlinear response of RC structures under extreme loading conditions have been performed. Though the experimental investigation gives the required information close to reality, it is not always a viable alternative as experimental parametric studies incur huge cost and time. On the other hand, the design method envisaged in various codes based on many concepts often underestimates or over-estimates the structural response. Thus, there is a need for a reliable analytical model which predicts the behavior close to real behavior. Non-linear finite element analysis is an established analytical tool to evaluate response of RC structures.

The non-linearity in the reinforced concrete may be due to change in structural and material characteristics and may result in structural cracking of concrete, yielding of concrete and steel and crushing of concrete. Nevertheless, these non-linearities are considered to be instantaneous and assumed to be time independent. Such time independent non-linearities are usually incorporated in the analytical modeling of reinforced concrete. On the other hand, the non-linearity may also be caused due to creep and shrinkage effects which are time dependent. Such time dependent non-linearities are difficult to be incorporated analytically and hence not considered in the present study.

The time independent behavior of materials can be further idealized into elastic behavior and plastic behavior. For an elastic material, there exists a one-to-one coordination between stress and strain. An elastic material is the material which returns to the original shape when the loads are removed. This is the minimal requirement for the material to qualify as elastic. The material which satisfies this minimal requirement is also known as hypo-elastic material. In a more restricted sense, an elastic material must also satisfy the energy equation of thermodynamics. The elastic material characterized by this additional requirement is known as hyper-elastic.

On the other hand, the plastic material is the one in which the reversibility is not satisfied, i.e. the material undergoes some permanent deformation which cannot be retraced even after the removal of loads and stresses. Hence, the strain in the plastic material may be considered as the sum of the reversible elastic strain and the permanent irreversible plastic strain. The stress-strain law for a plastic material reduces, essentially, to a relation involving the current state of stress and strain and the incremental changes of stresses and plastic strains. This relation is generally assumed to be homogeneous and linear in the incremental changes of the components of stress and plastic strain. This assumption precludes viscosity effects, and thus contributes to the time-independent idealization. For the complete modeling of RC, constitutive laws representing elastic and plastic states are to be defined clearly. Elasticity based constitutive laws are required to define the behavior of the material in the elastic state while plasticity based constitutive laws are required to define the behavior of the material in the plastic state.

Furthermore, the development of analytical models to determine the response of RC structures is complicated due to the following three factors: a) Reinforced concrete is a composite material made up of concrete and steel, which have very different physical and mechanical behavior, b) concrete exhibits non-linear behavior even under low level of loading due to cracking of concrete and c) reinforcing steel and concrete interact in a complex way through bond-slip and aggregate interlock. Thus, for the finite element analysis of RC structures such as panels and shear walls, the analytical model must include (i) a strength criterion for concrete subjected to various stress combinations, (ii) concrete cracking and crack propagation, (iii) steel yielding, (iv) concrete crushing, and (v) the tension-stiffening behavior of reinforced concrete through bond-slip.

The modeling of material may play a crucial role in achieving the correct response. The presence of nonlinearity may add another dimension of complexity to it. The nonlinearities in the structure may accurately be estimated and incorporated in the solution algorithm. The accuracy of the solution algorithm depends strongly on the prediction of second-order effects that cause nonlinearities, such as tension stiffening, compression softening, and stress transfer nonlinearities around cracks.

These nonlinearities are usually incorporated in the constitutive modeling of the reinforced concrete. In order to incorporate geometric nonlinearity, the second-order terms of strains are to be included. In this study, only material nonlinearity has been considered. The subsequent sections describe the modeling of concrete in compression and tension, modeling of steel.

(i) Concrete Modeling in Tension

The presence of crack in concrete has much influence on the response of nonlinear behavior of reinforced concrete structures. The crack in the concrete is assumed to occur when the tensile stress exceeds the tensile strength. The cracking of concrete results in the loss of continuity in the load transfer and hence the stresses in both concrete and steel reinforcement differ significantly. Hence the analysis of concrete fracture has been very important in order to predict the response of structure precisely. Crack modeling has gone through several stages due to the advancement in technology and computing facilities. Earlier Research work indicates that the formation of crack results in the complete reduction in stresses in the perpendicular direction, thus neglecting the phenomenon called tension stiffening. With the rapid increase in extensive experimental investigations as well as in computing facilities, many finite element codes have been developed for the nonlinear finite element analysis, which incorporates the tension stiffening effect.

The cracks are always assumed to be formed in the direction perpendicular to the direction of the maximum principal stress. These directions may not necessarily remain the same throughout the analysis and loading and hence the modeling of orientation of crack plays a significant role in the response of structure. Still, due to simplicity, many investigations have been performed using fixed crack approach, where in the direction of principal strain axes may remain fixed throughout the analysis. In this study also, the direction of crack has been considered to be fixed throughout the duration of the analysis. However, the modeling of aggregate interlock has not been taken very seriously.

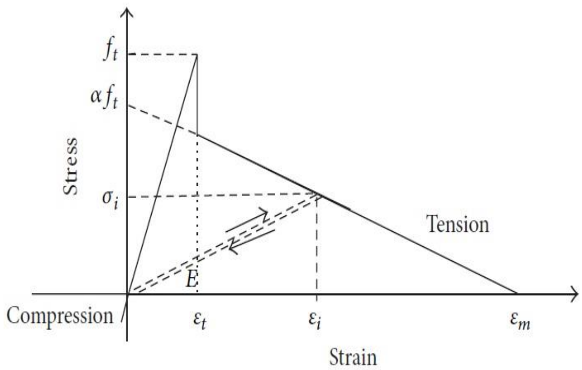

Figure 1: Tension stiffening effect of cracked concrete

The constant shear retention factor or the simple function has been employed to model the shear transfer across the cracks. Apart from the initiation of crack, the propagation of crack also plays a crucial role in the response of structure. The prediction of crack propagation is a very difficult phenomenon due to scarcity and confliction of test results. Nevertheless, the propagation of cracks plays a crucial role in the response of nonlinear analysis of RC structures. The plain concrete exhibits softening behavior and reinforced concrete exhibits stiffening behavior due to the presence of active reinforcing steel. A gradual release of the concrete stress is adopted in this present study as shown in Figure 1 [12]. The reduction in the stress is given by the following expression:

$$E_i = \alpha f_t'(1 - \frac{\varepsilon_i}{\varepsilon_m})\frac{1}{\varepsilon_i};\varepsilon_t \le \varepsilon_i \le \varepsilon_m \qquad (1)$$

$\alpha$and$\varepsilon_m$are the tension stiffening parameters. $\varepsilon m$is the maximum value reached by the tensile strain at the point considered. $\varepsilon i$is the current tensile strain in material direction i.

(ii) Concrete Modeling in Compression

The theory of plasticity has been used in the compression modeling of the concrete. The failure surface or bounding surface has been defined to demarcate plastic behavior from the elastic behavior. Failure surface is the important component in the concrete plasticity. Sometimes, the failure surface can be referred to as yield surface or loading surface. The material behaves in the elastic fashion as long as the stress lies below the failure surface. Several failure models have been developed and reported in the literature [14]. Nevertheless, the five parameter failure model proposed by Willam and Warnke [15] seems to possess all inherent properties of the failure surface.

The failure surface is constructed using two meridians, namely, compression meridian and tension meridian. The two meridians are pictorially depicted in a meridian plane and cross section of the failure surface is represented in the deviatoric plane.

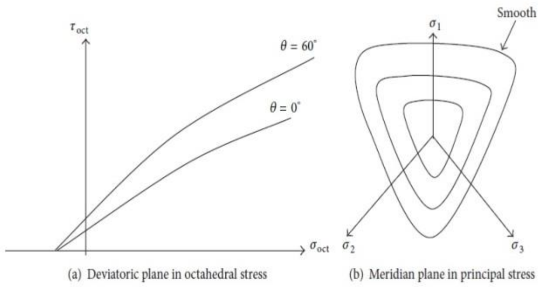

Figure 2: Willam-Warnke failure model

The variations of the average shear stresses $\tau_{mt}$and τ_{mc}along tensile (θ = 0°) and compressive (θ = 60°) meridians, as shown in Figure 2, are approximated by second-order parabolic expressions in terms of the average normal stresses σ_m, as follows:

$$\frac{\tau_{mt}}{f_c'} = \frac{\rho_t}{\sqrt{5}f_c'} = a_0 + a_1\left(\frac{\sigma_m}{f_c'}\right) + a_2\left(\frac{\sigma_m}{f_c'}\right)^2 \qquad \theta = 0^\circ,$$

$$\frac{\tau_{mc}}{f_c'} = \frac{\rho_c}{\sqrt{5}f_c'} = b_0 + b_1\left(\frac{\sigma_m}{f_c'}\right) + b_2\left(\frac{\sigma_m}{f_c'}\right)^2 \qquad \theta = 60^\circ. \qquad (2)$$

These two meridians must intersect the hydrostatic axis at the same point $\sigma_m/f_t' = \xi_0$(corresponding to hydrostatic tension); the number of parameters that need to be determined is reduced to five. The five parameters ($a_0$or b_0, a_1, a_2, b_1, b_2) are to be determined from a set of experimental data, with which the failure surface can be constructed using second-order parabolic expressions. The failure surface is expressed as

$$f(\sigma_m, \tau_m, \theta) = \sqrt{5}\frac{\tau_m}{\rho(\sigma_m, \theta)} - 1 = 0, \qquad (3)$$

The formulation of Willam-Warnke five-parameter material model is described in Chen [13].

Once the yield surface is reached, any further increase in the loading results in the plastic flow. The magnitude and direction of the plastic strain increment are defined using flow rule.

(a) Flow Rule

In this method, associated flow rule is employed because of the lack of experimental evidence in non-associated flow rule. The plastic strain increment expressed in terms of current stress increment is given as

$$d\varepsilon_{ij}^p = d\lambda\frac{\partial f(\sigma)}{\partial\sigma_{ij}}. \qquad (4)$$

$d\lambda$ determines the magnitude of the plastic strain increment. The gradient $(\sigma)/\partial\sigma_{ij}$defines the direction of plastic strain increment to be perpendicular to the yield surface; (σ) is the loading condition or the loading surfaces.

(b) Hardening Rule

The relationship between loading surfaces (or effective stress) and the plastic work (accumulated plastic strain) is represented by a hardening rule. The "Madrid parabola" is used to define the hardening rule. The isotropic hardening is adopted in the present study, as shown in Figure 3.

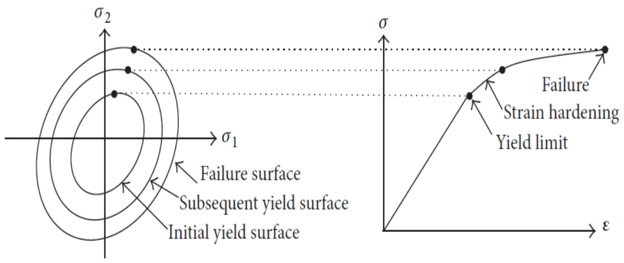

Figure 3: Isotropic hardening with expanding yield surfaces and the corresponding uniaxial stress-strain curve

$$\sigma = E_0\varepsilon - \frac{1}{2}\frac{E_0}{\varepsilon_0}\varepsilon^2 \qquad (3)$$

$E_0$is the initial elasticity modulus, ε is the total strain, and $\varepsilon_0$is the total strain at peak stressf_c'.

(iii) Modeling of Reinforcement in Tension and Compression

Reinforcing bars in structural concrete are generally assumed one-dimensional elements without transverse shear stiffness or flexural rigidity. The reinforcing bar can generally be treated as either discrete or smeared. The major advantage of discrete representation of reinforcing bar is existence of one to one correspondence between the real structure and model. In the smeared reinforcement, the average stress-strain relationship is calculated for an element area and incorporated directly as part of the overall concrete element stiffness matrix. In the present investigation, the smeared layered approach is adopted. The bilinear stress-strain curve with linear elastic and strain hardening region is adopted in this study as shown in Figure 4. Sometimes, the tri-linear idealization has also been adopted as the stress-strain curve in tension. Typically, the hardening strain modulus is assumed to be 1%of initial elasticity modulus.

The position and thickness of steel layers are to be defined as input parameters along with the elasticity modulus and hardening modulus. The direction of steel (horizontal or vertical) can be set up by defining the angle with respect to local x-axis. There can only be two states of stress for the reinforcing bar, namely, elastic and linear strain hardening.

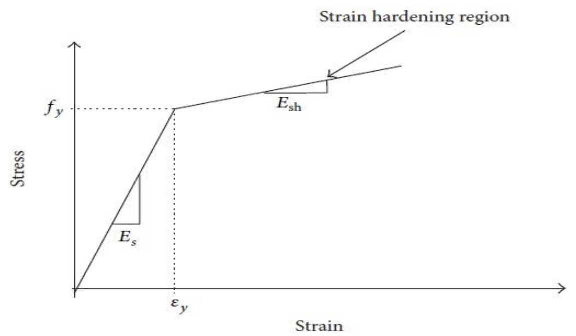

Figure 4: Stress-strain curve of steel

IV. DYNAMIC ANALYSISOF RC SHEAR WALL

The dynamic analysis of structure can be performed by threeways, namely, (i) equivalent lateral force method, (ii) responsespectrum method, and (iii) time history method. The equivalentlateral force method determines the equivalent dynamiceffect in the static manner. The response spectrum methodaims at determining the maximum response quantity of thestructure. For tall and irregular buildings, dynamic analysisby response spectrum method seems to be a popular choiceamong designers. The time history analysis of the structurehas been successfully used to analyze the structure especiallyof huge importance. Even though time history analysis consumestime, it is the only method capable of giving resultscloser to the actual one especially in the nonlinear regime. Inthe dynamic analysis, the loads are applied over a period oftime and the response is obtained at different time intervals.

The equation of dynamic equilibrium at any time "t" is given by (eq 4):

$$[M]\,[\ddot{U}^t] + [C]\,[\dot{U}^t] + [K]\,[U^t] = [R^t] \quad (4)$$

M, C, and K are the mass, damping, and stiffness matrices, respectively. The mass matrix can be formulated either by using consistent mass approach or by using lumped mass approach. Since damping cannot be precisely determined analytically, the damping can be considered proportional to mass or stiffness or both depending on the type of the problem.

The direct time integration [9] of the equation of motion can be performed using explicit (central difference scheme) and implicit (Houbolt method, New mark Beta method, and Wilson Theta method) time integration. In the explicit time integration, the formation of complete stiffness matrix of the structure is not required and hence saves a lot of computer time and money in storing and saving of those data. Moreover, in the case of all explicit time integration schemes, the iterations are not required as the equilibrium at time $t + \Delta t$ depends on the equilibrium at time t.Nevertheless, the major drawback of explicit time integration is that the time step (Δt) used for calculation of response has to be smaller than the critical time step (Δt_{cr}) to ensure the stable solution.

$$\Delta t \leq \Delta t_{cr} = \frac{T_n}{\pi} = \frac{2}{\omega} \quad (5)$$

On the other hand, implicit time integration requires the iterations to be carried out within the time step as the solution at time $t + \Delta t$ involves the equilibrium equation at $t + \Delta t$. The New mark β method converges to various implicit and explicit schemes for different values of Beta, called the stability parameter. In this study, for β = 0.25, the New mark β method converges to the constant acceleration implicit method, known as trapezoidal rule. The trapezoidal rule is unconditionally stable and hence allows larger time step to be used in the calculation of response. Nevertheless, the time step can be made smaller from the accuracy point of view. The formulation of implicit new mark Beta method (trapezoidal rule) is mentioned in the literature [9].

(i) Formulation of Mass Matrix

In a dynamic analysis, a correct estimate of mass matrix is very important in predicting the dynamic response of RC structures. There are two different ways, namely, (i) consistent approach and (ii) lumped approach, by which the element mass matrix can be developed. In the case of consistent approach, the masses are assumed to be distributed over the entire finite element mesh.

In this approach, the shape functions (**N**$_i$) used for the computation of mass matrix is the same shape functions used for the development of stiffness matrix and hence the name "consistent" approach. The mass matrix developed using the consistent approach is known as consistent mass matrix. The consistent element mass matrix (**M**$_e$) is given by

$$M_e = \iiint \rho_m N_i N_i dV \quad (6)$$

Where $\rho_m$the mass density "i" is the node number and the integration is performed over the entire volume of the element. The consistent mass matrix contains off-diagonal terms and hence is computationally expensive. On the other hand, the lumped mass matrix is purely diagonal and hence computationally cheaper than the consistent mass matrix. Nevertheless, the diagonalization of the mass matrix from the full mass matrix results in the loss of information and accuracy [8]. Nodal quadrature, row sum, and special lumping are the three lumping procedures available to generate the lumped mass matrices. All the three methods of lumping lead to the same mass matrix for nine node rectangular elements. Nevertheless, one of the most efficient means of lumping is to distribute the element mass in proportion to the diagonal terms of consistent mass matrix [6] and also discarding the off-diagonal elements. This way of lumping has been successfully used in many finite element codes in practice.

The advantage of this special lumping scheme is the assurance of positive definiteness of mass matrix. The use of lumped mass matrix is mostly employed in lower order elements. For higher order elements, the use of lumped mass matrix may not be an appropriate option and hence the present study uses only consistent mass matrix. Moreover, the lumped mass matrix may be an ideal option in the case of RC framed structures in which the masses can be lumped at floor level.

As RC shear wall is the concrete structure, it may be appropriate to use consistent mass matrix. The element mass matrices for consistent and lumped mass matrices are as mentioned below:

$$[M_e^c] = \begin{bmatrix} I_1 & 0 & 0 & 0 & I_2 \\ 0 & I_1 & 0 & -I_2 & 0 \\ 0 & 0 & I_1 & 0 & 0 \\ 0 & -I_2 & 0 & I_3 & 0 \\ I_2 & 0 & 0 & 0 & I_3 \end{bmatrix} \quad (7)$$

$$[M_e^L] = \begin{bmatrix} I_{L1} & 0 & 0 & 0 & 0 \\ 0 & I_{L1} & 0 & 0 & 0 \\ 0 & 0 & I_{L1} & 0 & 0 \\ 0 & 0 & 0 & I_{L2} & 0 \\ 0 & 0 & 0 & 0 & I_{L2} \end{bmatrix} \quad (8)$$

Equation (7) is very general and can be used to develop the consistent mass matrix for any displacement based finite element. In (7), "I_1" represents the contribution of mass to resisting the linear or translational motion and is defined as $I_1 = \int \rho_m dz$. Hence, masses corresponding to translational degrees of freedom are represented by "I_1."

On the other hand, "I_3" represents the contribution of mass to resisting the change in the rotatory motion and is expressed mathematically as $I_3 = \int \rho_m z.zdz$, $I_2 = \int \rho_m zdz$. The total mass matrix M is the sum of the element mass matrices $\mathbf{M}_e$. z is the position of layer middle surface from shell middle surface. Equation (8) represents the lumped mass matrix which is not used in the present analytical study.

(ii) Nonlinear Solution

The numerical procedure for nonlinear analysis employs the iterative procedure to satisfy the equilibrium at the end of the load step. Once the convergence of the solution is achieved, the algorithm proceeds to the next step. It is always desirable to keep the load step very small especially after the onset of nonlinear behavior. The stiffness matrix is updated at the beginning of each load step. The convergence is said to be achieved if the out of balance forces, calculated as follows, are less than the specified tolerance:

$$\psi_i^n = f^n - p_i^n = f^n - \int_V B^T \sigma_i^n dV < \text{Tolerance } (0.0025) \quad (9)$$

V. Analysis of Slender Shear Wall- Non-Linear Dynamic Analysis

In order to perform the non-linear analysis of RC shear wall under dynamic loading conditions, the rectangular shear wall, shown in Fig. 7, has been discretized into 120 finite elements using 9-noded 5-dof assumed strain based degenerated shell element with layered approach. The geometry of the shear wall, its elevation [Fig. 5(a)] and longitudinal section [Fig. 5(b)] are the same as analyzed by Agarwal et al. (1981).

The material properties of RC shear wall are as mentioned in Table 1. The reinforcement is provided in two layers in horizontal direction and a single layer in vertical direction. The diameter of the reinforcing bar used as vertical and horizontal reinforcement is 4 mm diameter. In order to incorporate the effect of steel reinforcement, the layered approach is adopted in this study.

The steel is modeled as a smeared layer of equivalent thickness. The properties of the steel are assumed constant in that layer. The bi-linear stress strain curve with linear elastic and strain hardening region is adopted in this study. The simulated EL Centro earthquake, shown in Fig. 6, is applied at the base of the shear wall with maximum amplitude or 1.05g.

The response of the structure is traced for 1.5 seconds of duration. Several investigators have also adopted this way of response calculation by predicting the response only for the most intense earthquake period in order to simplify the computation.

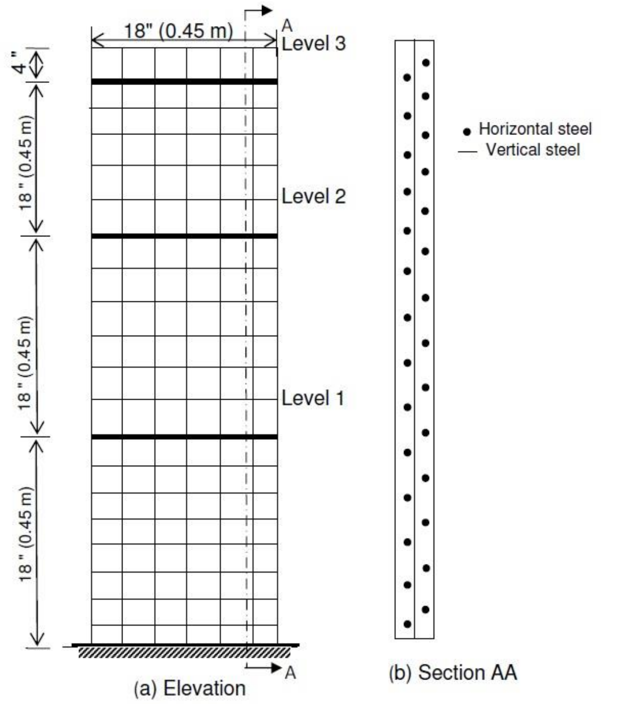

Figure 5: Finite element idealization of rectangular shear wall

Table 1: Material properties of concrete and steel

Units	Concrete			Reinforcing Steel			
	Elasticity Modulus	Yield strength in compression	Tensile strength	Elasticity Modulus	Strain hardening modulus	Yield stress Vertical	Yield stress Horizontal
psi	3.8×10^6	4720	409	2.9×10^7	7.03×10^4	53,500	53,500
MPa	26200	32.54	2.82	199947.9	48.5	368.8	368.8

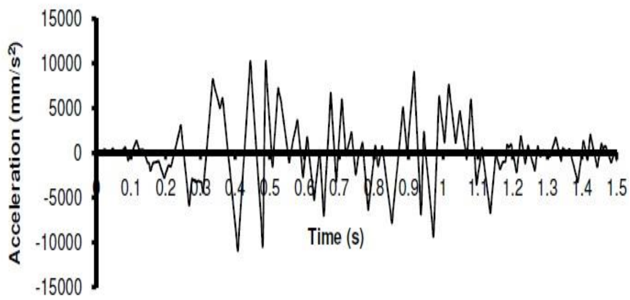

Figure 6: Input ground acceleration applied at the base of shear wall

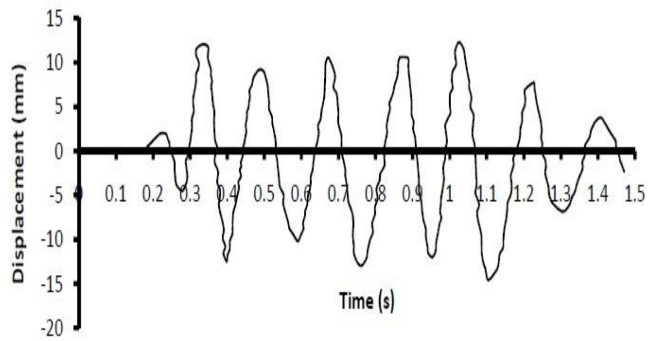

Figure 7 (a): Experimental displacement time history of the shear wall (Hsu et al. 1981)

Fig. 7(a) and Fig. 7(b) show the experimental and analytical displacement time history responses, respectively, of RC shear wall subjected to scaled EI Centro ground motion as reported by Lefas et al. (1990). There is a considerable difference between the observed experimental response and analytical response reported by Agarwal et al.

Though the maximum displacement response from both analytical and experimental approach is not much different, the shape of the displacement time history is different, especially for the duration from 0.4 seconds to 1.0 seconds. Fig. 8 shows the displacement time history response of RC shear wall proposed in the present study.

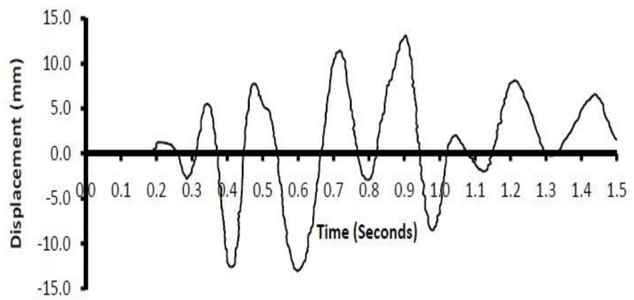

Figure 7 (b): Analytical displacement time history of the shear wall

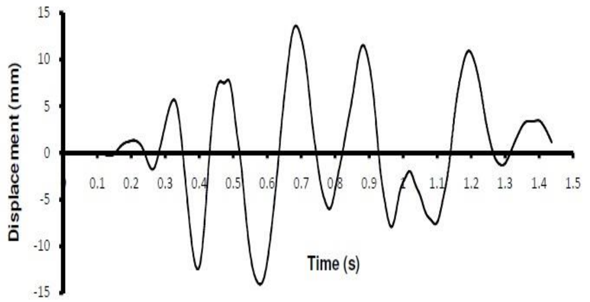

Figure 8: Displacement response history response of shear wall subjected to scaled EL Centro earthquake (Present study)

VI. Conclusion

In this paper we analyzed the dynamic behavior of reinforced concrete shear walls. Shear walls may be provided with openings due to functional requirement of the building. The size and location of opening may play a significant role in the response of shear walls.It studied the dynamic behavior of shear walls under various opening locations using nonlinear finite element analysis using degenerated shell element with assumed strain approach.

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