SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

STATE-OF-THE-ART ON THE RECENT EVOLUTION OF MACRO-MODEL OF RC SHEAR WALLS

Salima DJEHAICHIA[1] and Rachid LASSOUAD[2]

ABSTRACT

Reinforced concrete structural walls are very efficient elements for protecting buildings against excessive early damage and against collapse under earthquake actions. Several research as well in theoretical and numerical domain as experimental, were conducted to investigate the behavior of RC shear walls under the lateral loads, it is therefore of interest to develop a numerical model which simulates the typical behavior of these units. From a structural engineering point of view, be classified in tow major model levels; micro models and macro models, but in this paper attention is focused on a macroscopic approach. These range from models attempts to incorporate the entire behavior of a major region of a structural wall, such as a storey height or part thereof, including the wall's constituents such as the concrete, the reinforcing steel and the interaction effects between concrete and steel. The objective of this paper is to provide a state-of-the-art on the recent advancements and evolution of macro models, some of the more important numerical models are presented in this paper. The different models are described and their advantages and limitations mentioned.

INTRODUCTION

In many buildings, reinforced concrete structural walls provide an important part of the resistance against lateral actions, such as wind and earthquake. During the recent years, an enormous effort has been done to provide analytical models that are able to simulate the actual behavior of RC elements including shear walls, The numerical modeling of RC walls is not involved only in the applications for new construction, but it is also extended to the applications of retrofitting of existing structures. In that case, it is important to construct a representative model that is able to evaluate the expected response of an existing RC shear wall under lateral load, and to predict its expected mode of failure in order to be able to choose the most suitable and effective retrofitting technique for that wall that would meet a target performance. The numerical modeling of structural walls may, from a structural engineering point of view, be classified in two major model levels: macro models and micro models. Micro-modeling such as finite element analysis or fiber analysis is based on representing the behavior of different materials that compose the RC element and the interaction between them. Some researchers used the finite element method to study the effects of different design parameters on the response of reinforced concrete members. accurate simulations of behavior in reinforced concrete .This type of modeling is unsuitable for inelastic analysis of entire structural behavior because of their immense demand on computational needs Therefore, the macro modeling focusing on overall

[1]Department of Civil Engineering, Faculty of Science and Technology, Laboratory of Materials and Durability of Constructions, University Constantine 1, Algeria Email: sali05000@yahoo.fr
[2]Department of Civil Engineering, Faculty of Science and Technology, Laboratory of Materials and Durability of Constructions, University Constantine 1, Algeria Email: rachid_lassoued@yahoo.fr,

behavior of the RC element is more widely used and numerous efforts have been done in such modeling strategy. Former attempt to model the overall behavior of a structural wall cross section over a certain height, while the latter base the behavior upon the constitutive laws of the mechanics of solids..However, it appears possible to divide the more important and frequently used models into three types,

I-HYSTERESIS MODEL

Modeling the hysteretic behavior of structural elements is one of the core aspects in nonlinear structural analysis. These models can be used to represent the axial, flexure and shear behavior of the element. The hysteretic model consists of a primary curve (backbone curve) that control the monotonic loading and some hysteresis rules that control the loading and unloading element behavior under cyclic loading. Several improvements in the hysteretic models were accompanied by the advancements in the wall modeling, but in this paper the attention is focused on the axial-stiffness hysteresis model and origin-oriented hysteresis model. the characteristics of each model are described.

II-RC WALL MACROSCOPIC MODELS

II-1 BEAM ELEMENT MODELS

The simplest numerical model for a structural wall consists of beam elements, with six degrees of freedom per element. The wall in this case is considered as a deep column. This is a very commonly used concept and in some analysis situations it may provide a model which is sufficiently realistic. If the vertical deformations at the wall edges due to flexure are considered unimportant, the entire wall model for one storey assimilate as a single beam element (Linde P, 1993).

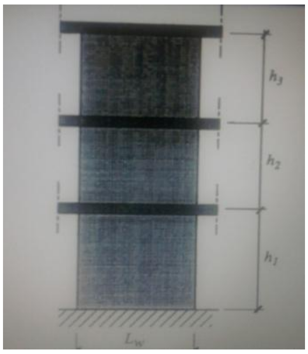


Figure 1. Structural wall prototype (linde,1993)

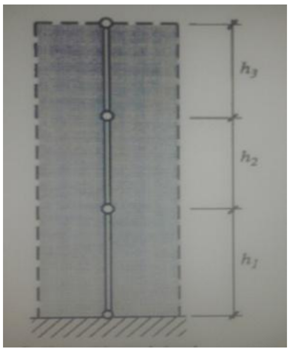


Figure2. Beam element model (linde P,1993)

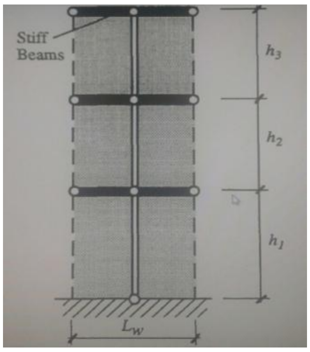


Figure3. Beam element model with horizontal stiff beams (linde P,1993)

For walls with considerable horizontal length, it may be necessary to consider the vertical edge deformations .a simple solution including this effect has been suggested by adding horizontal rigid beams on either side of the vertical beam (linde P,1993) , as shown in figure 3.that's main each shear wall is replaced by an idealized frame structure consisting of a column and rigid beams located at floor levels . The column is placed at the wall's centroidal axis and assigned to have the wall's inertia and axial area. This model present some advantages consist of: the uncomplicated modeling, sometimes possibilities to check the results by frame analogy hand calculations, few degrees of freedom especially in dynamic analysis.

The limitations are mainly due to: The inability to describe the walls behavior along its cross section, The vertical deformations at edge of the wall are not considered if there are no horizontal rigid beams, In spite of the presence of rigid beams the strain distribution will not be realistically modeled since the shift of the neutral axis, which is typical for a wall when flexural cracking and subsequent yielding occurs, cannot be reproduced. This is especially noticeable under flexure at the tensile edge where the large tensile strains are not considered by the model, the model was limited to lateral load analysis of rectangular building frames without torsion.

II-2 MULTI – COMPONENT BEAM-COLUMN ELEMENT

The multi-component beam-column element was the first nonlinear beam-column model was used for structural analysis of a reinforced concrete element , the model proposed by Clough et al, (1965) to represent a bilinear nondegrading hysteresis. the element is composed in two parallel elements, one elastic –perfectly plastic to represent yielding and the other perfectly elastic to represent strain-hardening. When the member-end moment reaches the yield level, a plastic hinge is placed at the end of the elasto-plastic element. A member-end rotation depends on both member-end moments. The stiffness matrix of the member is the sum of the stiffnesses of the components. The main problem of this model was its inability to represent the element stiffness or strength degradation with cyclic loading. This model was improved by (Takizawa, 1976) to be able to simulate different hysteretic behavior of RC elements by using appropriate hysteresis models .

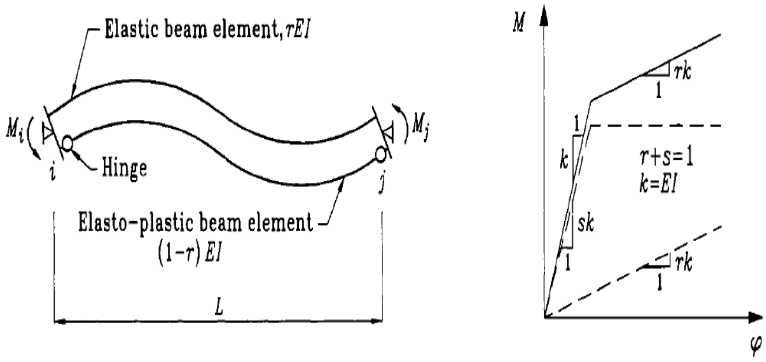


Figure 4 : Two-component element model (El-Sokkary H, 2012).

II-3 ONE – COMPONENT BEAM-COLUMN ELEMENT

Giberson (1967) proposed a concentrated spring model for column and beam elements figure 5. this model consists of a linearly elastic member with two nonlinear rotational springs. Each spring attached at each end. These springs take account of any nonlinear characteristics that occur within the members. This model is versatile since the spring at each end can have different curvilinear or bilinear hysteretic characteristics.

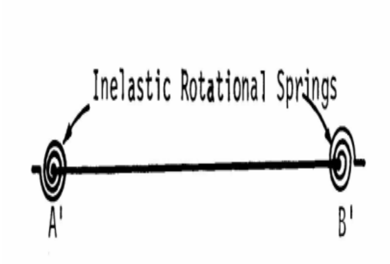


Fig 5 : One-component member model.

Among the advantages of this model is that inelastic member-end deformation depends just on the moment acting at the end so that any moment-rotation hysteretic model can be assigned to the spring. This model presents also a certain limit that the member-end rotation should be dependent on the curvature distribution along the member and, hence dependent on moments at both member ends. Consider two cases of moment distribution along a member AB with corresponding curvature distributions as shown in figure 6. The inelastic rotations at the A end are given by the shaded areas. For the same moments at the A end, case II causes larger inelastic rotation at the A end. Consequently, this simple model does not simulate actual member behavior. Furthermore, it is not rational to lump all inelastic deformations at member ends (Otani S, 1980)..

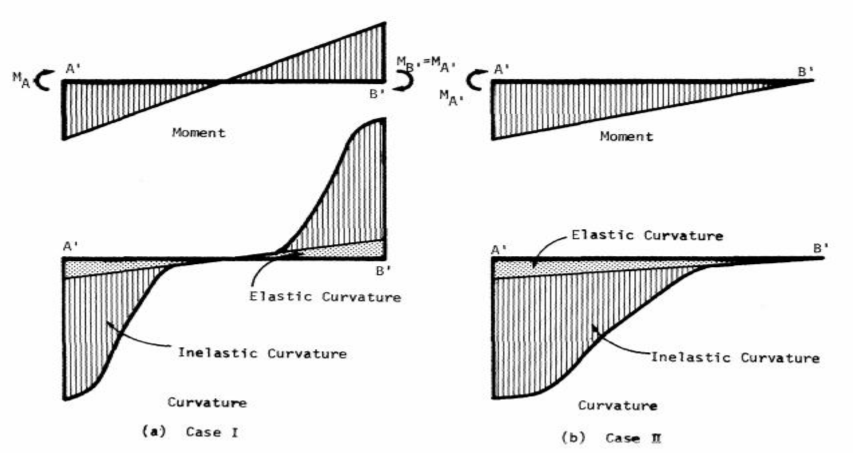


Fig 6: Inelastic rotation of beam: (a) moment; (b) curvature and inelastic rotation(Otani S ,1980).

The stiffness of an inelastic spring is defined by assuming an asymmetric moment distribution along a member with the infection point at mid span .the usage of the initial location of the infection point in Evaluating spring properties was suggested by Suko and Adams 1971. However, once yielding is developed at one member end , the moment at the other end must increase to resist a higher stress , moving the inflection point toward the member centre. At the same time, a large concentrated rotation starts to occur near the critical section .despite rational criticisms against this simple model. The performance of this model is expected to be reasonably good for a relatively low rise frame structure, in which the inflection point of a column locates reasonably close to midheight.

II-4 MULTIPLE SPRING MODELS

This model was proposed by Takayanagi and Schnobrich (1976).it is based on flexural line elements representing the walls and the connecting beams. The multiple spring model consisted of a number of inelastic springs that are connected in series using rigid members as shown in Figure 7. The inelastic properties of each spring were varied according to the segment properties and the level of axial load on that segment, however the segment properties were assumed to be constant along the segment length. The model was used to represent the behavior of coupled shear walls, while the coupling beams were modeled using one-component elements. This model was used by Emori and Schnobrich (1981) to model the shears wall of a 10-storey frame-wall building. Linear shear deformations were assumed in the analysis. The models were found to satisfactorily represent the nonlinear behavior of the studied structure.

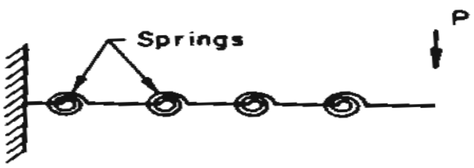


Fig 7: Multiple spring model (Takayanagi and Schnobrich ,1976).

II-5 TRUSS ELEMENT MODELS

A truss model as shown in figure 8 Was used by Vallenas et al 1979, and by Hiraishi 1984, This model consists of two vertical truss elements, and at least one diagonal truss element. These are connected by a rigid horizontal beam. The diagonal truss is supposed to model the concrete "compression strut" which forms under lateral force. This behavior may be reproduced quite well; but under force reversal it is necessary to use a diagonal truss in the opposite diagonal direction. Furthermore, the reproduction of behavior under various moment/shear applications seems problematic, as well as the realistic modeling of deformations due to gravity load and lateral force, each by itself, and combined. In the case of static monotonic force application, and for a small gravity load, the model may provide useful results, if carefully calibrated. However, its use appears to be limited to rather squat walls, where a compression strut of this nature actually develops. Further, the versatility of the model may be limited compared to other models, and dynamic analysis does not appear feasible(Linde P ,1989).

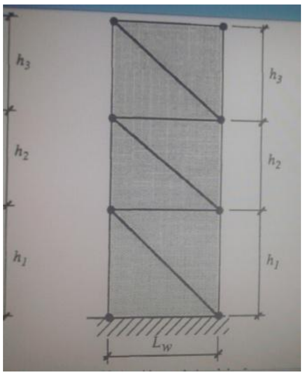


Fig 8: Truss element model (Linde P, 1989)

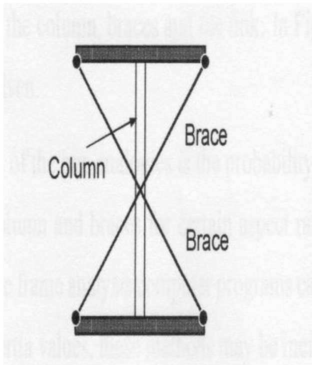


Fig 9: Braced Wide Column (Tolga A, 2004)

Smith et al (1984), was developed the braced wide column analogy. A single module consists of rigid horizontal beams, equal in length to the width of the wall, connected by a single central column. Hinged-end diagonal braces connect the ends of the beams as shown in Figure 9. A planar shear wall modelled by braced wide column. The stiffness properties of the column (Ic : moment of inertia of the column and Ac: area of the column) and braces (Ad: axial area of the diagonal brace) are determined by the following three equations which are based on the simulation of the bending, shear and axial stiffnesses of corresponding wall segments :

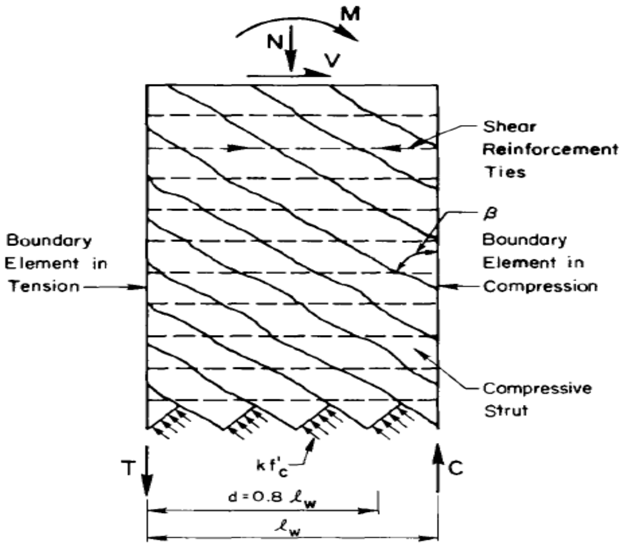
$$I_c = \frac{tb^3}{12} \quad (1)$$

$$\frac{12EI_C}{h^3} + \frac{2EA_d\cos^2\theta}{l} = \frac{btG}{h} \quad (2)$$

$$\frac{EA_C}{h} + \frac{2EA_d\sin^2\theta}{l} = \frac{EA_W}{h} \quad (3)$$

t: the thickness of the shear wall , b: the width of the shear wall
E: the modulus of elasticity, h: the height of the shear,
θ: the slope of the diagonal, l : the length of the diagonal brace,
G: the shear modulus, Aw: the sectional area of the shear wall

One of the deficiencies of this model is the probability of obtaining negative stiffness values for the column and braces for certain aspect ratios of the framework modules. Since most of the frame analysis computer programs cannot perform analysis with negative area and inertia values, these methods may be ineffective (Tolga A ,2004). (Oesterle et al ,1984) used a truss analogy to determine the shear stress associated with web crushing of barbell and I-shaped wall cross-sections as: such as the Softened-Strut-and-Tie model shown in Figure 10(a).The model was used by Yu and Hwang (2005) to predict the shear capacity of RC squat walls as shown in Figure10(b). It is worth noting that, although such models are able to predict the capacity of RC elements, they can not capture the cyclic or the hysteretic behavior of these elements.



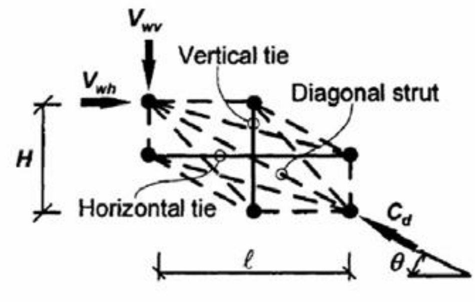


Figure 10: (a) Truss model used by (Oesterle et al , 1984) (b) Softened-Strut-and-Tie model (Yu and Hwang , 2005).

II-6 MULTIPLE SPRING ELEMENT MODELS

II-6-1Three Vertical Line Element(TVLM)

proposed by Kabeyasawa et al (1982) shown in figure 11, was essentially used in numerical analyses for the prediction of the static cyclic and pseudo dynamic tests of the full scale wall in Tsukuba, and the scale models in the US-Japan cooperative research program. The model consisted of five nonlinear springs, connected by rigid beams.

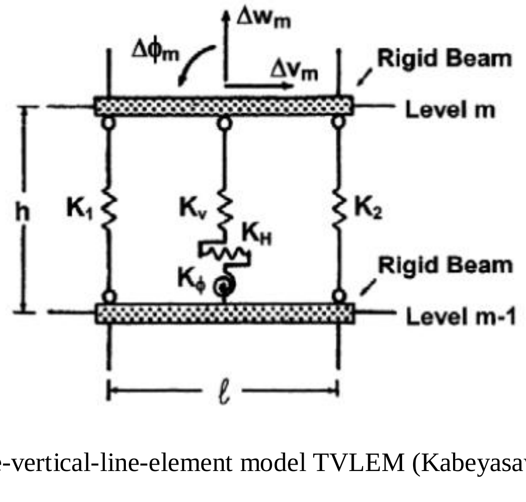


Figure 11:Three-vertical-line-element model TVLEM (Kabeyasawa et al. 1983)

The springs were made up as follows:

- Two vertical outer Springs, representing the axial behavior of the boundary columns, with stiffness K_1 and K_2
- One central vertical spring, representing the axial behavior of the web,
- One central horizontal spring representing the shear behavior of the wall section,
- One central rotational spring, intended to represent the flexural behavior of the web.

The three central springs were located at the base of the element, was considered as one component model with stiffness Kv, Kh and KΦ respectively. The axial –stiffness hysteresis model (ASHM), shown in figure 12 was proposed to simulate the response of line element and define the axial force – deformation relation of the three vertical line element of the wall model .in this hysteresis model ,Kt and Kc represent axial stiffness under tension and compression respectively (Kabeyasawa et al, 1983).

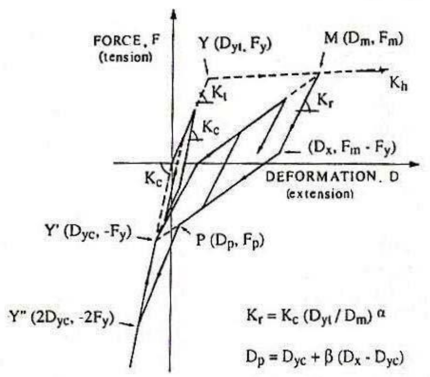


Figure 12: Axial Stiffness Hysteresis Model (Kabeyasawa et al, 1983)

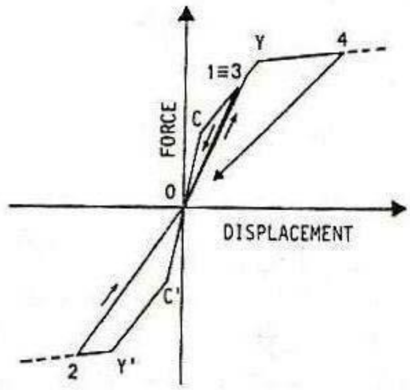


Figure 13: Origin-oriented hysteresis model (OOHM) (Kabeyasawa et al, 1983)

The origin –oriented hysteresis model in figure 13, was used for both the rotational and horizontal springs at the base of the central vertical element of a wall model. The stiffness Properties of the rotational spring were defined by referring to the wall area bounded by the inner therefore; displacement compatibility with the boundary columns was not enforced. Shear stiffness degradation was incorporated, but was assumed to be independent of the axial force and bending moment. The attempt to separately model flexural and axial behavior in this manner led to compatibility problems, mostly when flexural and axial properties are assigned to the rotational and vertical Springs, respectively, as suggested for the original model, since these assigned properties base on the independent behavior of the web and the boundary columns (Linde P,1993).

II-6-2 MODIFIED THREE VERTICAL LINE ELEMENTS (MTVLM):

To limit the empirical assumptions Volcano and Bertero (1987) proposed the MTVLEM shown in Figure 14, This model modifies TVLEM by replacing the axial ASHM with the Two-axial element-in-series model" (AESM) in this case the Hysteresis model is composed of two elements: The first (Element 1 on top -one component model -) is intended for uncracked concrete and represent the overall axial –stiffness of the boundary element in which the bond between steel and concrete is still active. while Element 2 is two component model to represent the axial stiffness of steel (S) and cracked concrete (C) where the bond is lost. Ec Ac and Es As are the axial stiffnesses of concrete and steel, respectively. Λ accounts for the tension stiffening effect. The AESM idealizes element 1 by linearly elastic curve. Element 2 is idealized by bilinear curve and linearly elastic curve in compression neglecting tensile strength, respectively for steel and cracked components of the element. This model was able to predict the flexural behavior of the tested wall that was dominated by flexural failure; it was not able to simulate the actual shear deformations of the wall, which indicates that this model is not suitable for walls dominated by shear behavior. In this model the deformation compatibility between the wall and the boundary element was still not enforced.

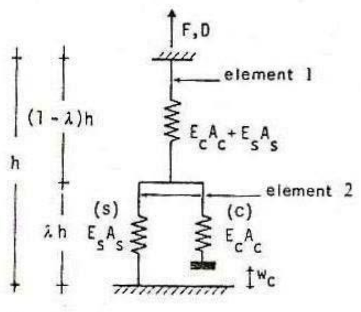
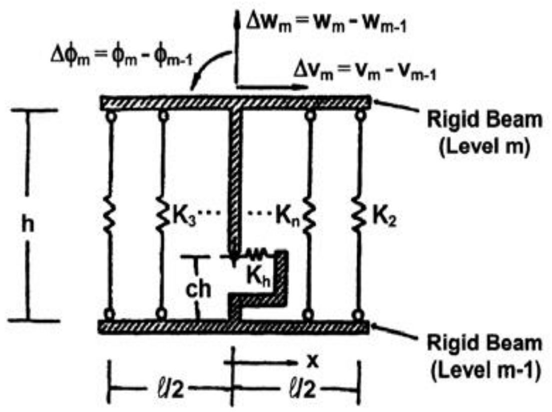


Fig 14: Axial-element-in-series model (AESM)
(Vulcano and Bertero, 1986)

II-6-3 MULTI-COMPONENT–IN-PARALLEL MODEL (Volcano et al, 1988)

This model eliminates the difficulty in assigning reasonable values to the rotational spring in TVLEM. the rotational spring was replaced by several additional vertical springs connected in parallel using infinitely rigid beam located at the top and bottom wall ends figure 15. In this approach, Two external elements represents the axial stiffnesses (K1 and K2) for the boundary elements, while the other elements with axial stiffnesses (K3,……..,Kn) simulate the combined axial-flexure behavior of the central panel. A horizontal spring was used to represent the inelastic shear behavior of the wall with stiffness Kh and hysteretic behavior described by the OOHM (figure 13).The axial –element-in series model (AESM) was modified by having two-component model for element 1, representing the uncracked concrete and steel in which the bond remained active figure 16. For element 2, the bond stresses were negligible. A parameter λ was introduced to define the relative length of the two elements to account for tension stiffening. This method was able to simulate the gradual yielding of the vertical reinforcement more smoothly, but it consists of more components and thus leads to a more complicated model. Generally, refinements lead in the direction of micro models (Orakcal K. and Wallace J W., 2006).



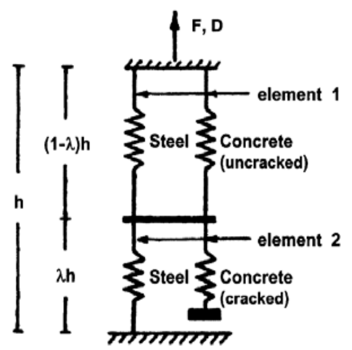


Figure 15: Multiple vertical line element model
(Vulcano et al., 1988).

Figure 16: Modified axial-element-in-series
model (Vulcano et al., 1988).

It was concluded that the model predicted the flexural behavior of the wall efficiently even when relatively few uniaxial elements were used (4 elements). It is worth noting that the proposed model considered both flexural and shear behavior, but their responses were not coupled. Orakcal and Wallace (2006), was improved the efficiency of the model in predicting the response of RC shear walls. Using an approach to incorporate coupling between flexure and shear components of wall response.

CONCLUSIONS

Various member models are reviewed, and their advantages and disadvantages are discussed. These models have been developed specifically for earthquake response. Different hysteretic models used to simulate the behaviour of RC shear walls were also discussed. The paper provided a state-of-the-art on the recent advancements and challenges in the area of modeling of RC shear walls.

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