

A NONLINEAR QUADRILATERAL LAYERED MEMBRANE AND SHELL **ELEMENT WITH DRILLING DOF FOR THE MODELING OF RC WALLS**

F. Rojas⁽¹⁾, J.C. Anderson⁽²⁾, L. Massone⁽³⁾

⁽¹⁾ Assistant Professor, Department of Civil Engineering, University of Chile, Santiago, Chile, e-mail: frojas@ing.uchile.cl ⁽²⁾ Professor, Department of Civil and Environmental Engineering, University of Southern California, LA, USA, jamesa@usc.edu

⁽³⁾ Associate Professor, Department of Civil Engineering, University of Chile, Santiago, Chile, e-mail: Imassone@ing.uchile.cl

Abstract

Reinforced Concrete (RC) walls are a fundamental part of lateral resisting force system against earthquake and wind loads for tall and mid-rise multi-story buildings. This type of systems provides the necessary stiffness and strength to satisfy the demands produced by strong ground motions. The design and modeling of reinforced concrete wall elements has been an extensive area of research, this is not only due to the complex behavior of the material, and the coupling between axial, moment and shear behavior, but also for the behavior of the wall as a structural element, as was demonstrated during the Chilean Earthquake of February 27, 2010, Mw 8.8. In addition, wall response change depending on the different configuration (sizes, shape), and disposition of the components of the walls (reinforcement, aggregated size, confining of the concrete), and also their location inside of the structural system.

In this work, a quadrilateral layered membrane and thin flat shell element with drilling degrees of freedom (DOF) are presented and tested for the nonlinear analysis of reinforced concrete walls under static and cycling loads. The membrane and shell element allows to model complex configuration of wall system in 2D and 3D, respectively. The drilling degrees of freedom refer to the incorporation of the in-plane rotation as a degree of freedom at each node of the element. The membrane element consists of a quadrilateral element with a total of 12 DOF, 3 per node, 2 displacements and 1 in-plane rotation, and uses a blended field interpolation for the displacements over the element. In addition, the membrane model is combined with the Discrete Kirchhoff Quadrilateral Element (DKQ, 12 DOF), formulated by Batoz and Tahar in 1982, to generate the shell element, with a total of 24 DOF, 6 per node. The modeling of the section of the membrane and shell element consists of a layered system of fully bonded, smeared steel reinforcement and smeared orthotropic concrete material with the rotating angle formulation. The proposed elements are validated using experimental results of different wall configurations, such as slender walls, T-Shaped walls and walls with irregular disposition of openings, that are available in the literature. It is shown that the proposed elements produce excellent agreement with the experimental results for cyclic loading. The elements are able to predict the maximum capacities (shear, flexural, compression) and global and local behavior for the different wall configurations analyzed in this work.

Keywords: Reinforced concrete wall; Membrane element, shell element; concrete rotating angle model;



1. Introduction

RC Walls are a fundamental part of lateral resisting force system against the lateral forces of earthquake for tall and mid-rise multi-story buildings. Wall and wall-frame systems provide the necessary stiffness and strength to limit the story drift and satisfy the demands produced by strong ground motions. In addition, these systems have exhibit a good performance in the recent earthquakes with rare occurrences of complete collapse.

Although the design of RC walls is a relatively simple procedure when using current codes, as the ACI318-08 [1] or later, the behavior of RC walls under seismic loads are actually very complex, because the walls behave differently depending on their configurations (wall size, height/length ratio, steel reinforcement, etc.) and loading conditions. This scenario implies that the behavior of RC walls depends on the interrelation and coupling of a combination of flexural, shear, and axial deformation over their cross sections at different levels, along with other complex mechanisms such as the rigid body rotation for the bond slippage of longitudinal reinforcement at the base, effects of confinement, dowel action in reinforcement, cracking, aggregate interlock, creep, and tension stiffening, which have been demonstrated by various researchers (Paulay and Priestley 1992 [2], Bertero 1980 [3], Orakcal et al. 2006 [4], Eimani et al. 1997[5]). In addition, the main behaviors, such pure flexion failure or pure shear failure, typically occur in isolated walls, once the walls are combined with other elements or other walls in the building (T, L wall systems), the behavior can change, producing combined failures such as flexural and compression failure, or combinations of failure ranging between flexural, axial and shear in which the failure is produced before reaching one of the walls maximum capacity in pure flexure, axial or shear, as has been observed after recent Chilean's earthquakes (Naeim et al. [6], Rojas et al. [7], Moehle et al. [8]).

Due to this complex behavior in RC walls, a large amount of research has been done in recent decades, and still does, with the goal of develop analytical models that can accurately predict the behavior of RC walls and important material characteristics, such as concrete stiffening, cracking, and neutral axial migration. The analytical models developed for the modeling of RC walls can be separated into two main groups, macroscopic models and microscopic models. The macroscopic models are based on predicting the overall behavior of a wall element with the use of simplified assumptions and idealizations (Vulcano and Bertero 1987 [9]). These models are typically done creating a system of springs, where each spring has an independent hysteric curve that represents a portion of the wall behavior. Meanwhile, microscopic models are typically based on the finite element method (FEM or FE) and theory of continuum mechanics. In this methodology, a RC wall is divided into a series of elements, over which the respective constitutive law, representing the behavior of the reinforced concrete material, is imposed in a stress-strain space or other possible mixed representation, and the equilibrium equation is satisfied in an average sense with integration over each finite element. The existing microscopic models for reinforced concrete walls can be grouped into three main categories: membrane elements, shell elements and 3D solid brick elements, with the membrane element the more used for the modeling of 2D walls.

In this article, a quadrilateral layered membrane element with drilling degrees of freedom (DOF) and a quadrilateral thin flat layered shell element for the nonlinear analysis of reinforced concrete walls, are presented and tested. The drilling degrees of freedom refers to the incorporation of the in-plane rotation as a degree of freedom at each node of the element. The membrane element consists of a quadrilateral element with a total of 12 DOF, 3 per node (2 displacements and 1 in-plane rotation), and uses a blended field interpolation for the displacements over the element proposed by Rojas in [10] 2012. This formulation is an extension of the one developed by Xia et al. in 2009 [11]. The shell element is created by the combination of the membrane element mentioned before, and the Discrete Kirchhoff Quadrilateral Element (DKQ, 12 DOF), formulated by Batoz and Tahar [12], to model the out of plane bending behavior of the element. The modeling of the section of the membrane and the shell element consists of a layered system of fully bonded, smeared steel reinforcement and smeared orthotropic concrete material with the rotating angle formulation. The layered section for the shell includes the coupling membrane and bending effects. The two elements are validated against experimental RC wall test with different configurations available in the literature. The element mebrane presented in this work is different to other membrane elements developed before, due to the incorporation of the drilling DOF, and the combination of the characteristic introduced in the concrete and steel materials.



2. Formulation of the quadrilateral layered elements

In this section, the formulation of a quadrilateral layered membrane and shell element are presented (Fig. 1).



Fig. 1 - The membrane and shell element definitions

2.1 Formulation of the quadrilateral layered membrane element with drilling DOF

Membranes are in a state of plane stress, in which only in-plane behavior is considered ($\sigma_z = \tau_{zx} = \tau_{zy} = 0$). The finite element formulation for this type of element, using a displacement-based approach, is typically developed from the concept of virtual work, and it is well known. The element stiffness matrix and resisting force assuming a fully bonded layered section, which are neccesary to implement the element in any framework of nonlinear finite element, and also used in this work, are presented next.

The tangent stiffness of the membrane using a displacement-based approach is defined as

$$K = \int_{A} \left[B\right]^{T} \left[D\right] \left[B\right] dA \tag{1}$$

where [B] is the kinematic matrix, which relate the deformations and displacements, and it can be defined as:

$$[B(x,y)] = \underbrace{\begin{bmatrix} \frac{\partial}{\partial x} & 0\\ 0 & \frac{\partial}{\partial y}\\ \frac{\partial}{\partial y} & \frac{\partial}{\partial x} \end{bmatrix}}_{[\partial]} [\Psi(x,y)] = [\partial] [\Psi(x,y)]$$
(2)

and [D] is the material tangent matrix, which include the concrete and steel layers, and it can be expressed in a discrete form using the procedure presented by Zhang et al. [13,14] (Fig. 2a), see Eq. (3)

$$[D] = \int_{-\frac{t}{2}}^{\frac{t}{2}} [D(z)] dz = \sum_{i=1}^{Nc} [D_{c_i}] (z_{i+1} - z_i) + \sum_{j=1}^{Ns} [D_{s_j}] t_{s_j}$$
(3)

where $[D_{ci}]$ and $[D_{sj}]$ are the plane stress material stiffness tangent of the ith concrete layer and jth Steel layer, respectively. N_c and N_s are the number of concrete and steel layers, respectively. z_{i+1} and z_i is the position of the top and bottom part of each concrete layer and t_{sj} is the thickness of the jth steel layer.

Also, the internal resisting force (R) for the membrane, can be obtained using a similar approach, and assuming zero initial stress, as:

$$R = \int_{A} [B]^{T} \underbrace{\left(\int_{-\frac{t}{2}}^{\frac{t}{2}} \{\sigma\} dz\right)}_{\{\hat{\sigma}\}} dA = \int_{A} [B]^{T} \begin{cases} n_{x} \\ n_{y} \\ n_{xy} \end{cases} dA$$
(4)

where, $\{\sigma\}$ is the stress and can be calculated in a discrete manner, as follow:



$$n_{x} = \int_{-\frac{t}{2}}^{\frac{t}{2}} \sigma_{x} dz = \sum_{i=1}^{Nc} \sigma_{x_{i}}^{c} (z_{i+1} - z_{i}) + \sum_{j=1}^{Ns} \sigma_{x_{j}}^{s} t_{s_{j}}$$

$$n_{y} = \int_{-\frac{t}{2}}^{\frac{t}{2}} \sigma_{y} dz = \sum_{i=1}^{Nc} \sigma_{y_{i}}^{c} (z_{i+1} - z_{i}) + \sum_{j=1}^{Ns} \sigma_{y_{j}}^{s} t_{s_{j}}$$

$$n_{xy} = \int_{-\frac{t}{2}}^{\frac{t}{2}} \tau_{xy} dz = \sum_{i=1}^{Nc} \tau_{xy_{i}}^{c} (z_{i+1} - z_{i}) + \sum_{j=1}^{Ns} \tau_{xy_{j}}^{s} t_{s_{j}}$$
(5)

where, $\{\sigma_i^c\}$ is the in-plane stresses at the ith concrete layer, and $\{\sigma_i^s\}$ is the in-plane stresses at the jth steel layer. Although the finite element formulation is straightforward, the optimal representation of the displacement field interpolation for the membrane element that includes rotational degrees of freedom remains an area of research. Next, it is presented a blended displacement interpolation that includes rotational DOF, which satisfied the restriction that the rotation at the nodes must be the true rotation derived from the mechanics (Eq. (6))



Fig. 2 – The membrane and shell transversal layered section

The blended displacement interpolation presented, it is an extension of the displacement proposed by Xia et al. in 2009 [10]. The blended interpolation is based in the combination of a cubic interpolation in "y" and a linear interpolation in "x" to represent the displacement in the direction "x", and the combination of a cubic interpolation for "x" and a linear interpolation for "y" to represent the displacement in "y". This can be represented in a matrix form as:

$$\{u\} = \begin{cases} u \\ v \end{cases} = [\Psi(x, y)] \{U\}$$
(7)

where $[\Psi(x,y)]$ is the interpolation matrix, and $\{U\}$ is the vector of nodal displacement (Fig. 1a).

$$\{U\} = \{ u_1 \quad v_1 \quad \theta_1 \quad u_2 \quad v_2 \quad \theta_2 \quad u_3 \quad v_3 \quad \theta_3 \quad u_4 \quad v_4 \quad \theta_4 \}^T$$
(8)

The interpolation matrix can be defined as:

$$\left[\Psi\left(x,y\right)\right] = \left[MN\left(x,y\right)\right] \left[Tr\right] \tag{9}$$

where [Tr] is the transformation matrix, that relate the local and global displacement in the element, and can be written as:



and [MN(x,y)] is the matrix that contains the blended interpolation functions (N₁ and N₂: Linear interpolation; M₁,M₂, M₃ and M₄: the Hermitian interpolations).

$$\begin{bmatrix} MN(x,y) \end{bmatrix} = \begin{bmatrix} M_1(x) N_1(y) & 0 & -M_1(x) N_2(y) & 0 & M_2(x) N_1(y) & 0 \\ 0 & M_1(y) N_1(x) & 0 & M_1(y) N_2(x) & 0 & M_1(y) N_3(x) \\ -M_2(x) N_2(y) & 0 & M_2(x) N_3(y) & 0 & -M_2(x) N_4(y) & 0 \\ 0 & M_1(y) N_3(x) & 0 & M_2(y) N_3(x) & 0 & M_2(y) N_4(x) \end{bmatrix}$$
(11)
$$\begin{bmatrix} M_1(x) N_3(y) & 0 & -M_1(x) N_4(y) & 0 \\ 0 & M_2(y) N_1(x) & 0 & M_2(y) N_2(x) \end{bmatrix}$$

Using as base, the membrane element presented above, a thin flat layered shell element for small deformation is presented next.

2.2 Formulation of the quadrilateral layered shell element.

The formulation for the thin flat layered shell element presented here (Fig. 1b), using a displacement-based approach, is similar to the proposed by Oñate [15] in 1992, but with some modifications to include a fully bonded layered transversal section and the use of the blended displacement interpolation, presented before, and the displacement interpolation of the Discrete Kirchhoff Quadrilateral Element (DKQ, 12 DOF), formulated by Batoz and Tahar [12] in 1982 to represent the out of plane bending. The element stiffness matrix and resisting force, which are neccesary to implement the element in any framework of nonlinear finite element, and also used in this work, are as follows:

$$K = \int_{A} [B']^{T} [D'] [B'] dA$$
(12)

where [B'] is the kinematic matrix (see Eq. (13)), which relate the deformations and displacements inside of the shell, and is composed by the kinematic matrix for the membrane element ($[B_m]$, see Eq. (2)) and the kinematic matrix for the out of plane bending ($[B_b]$, proposed by Batoz and Tahar [12]).

$$[B'(x',y')] = \begin{bmatrix} B_m(x',y') \\ \cdots \\ B_b(x',y') \end{bmatrix}$$
(13)



and [D'] is the material tangent stiffness matrix (see Eq. (14)), which is composed by the tangent stiffness matrix of the membrane portion $([D'_m])$, the tangent stiffness matrix that relates the coupling between the membrane and bending $([D'_{mb}])$, and the tangent stiffness matrix of the out of plane bending portion $([D'_b])$.

$$[D'] = \int_{-\frac{t}{2}}^{\frac{t}{2}} [A_t]^T \frac{\partial \{\sigma'\}}{\partial \{\varepsilon'\}} [A_t] dz' = \int_{-\frac{t}{2}}^{\frac{t}{2}} \begin{bmatrix} [D(z')] & z' [D(z')] \\ z' [D(z')] & z'^2 [D(z')] \end{bmatrix} dz' = \begin{bmatrix} [D'_m] & [D'_{mb}] \\ [D'_{mb}] & [D'_b] \end{bmatrix}$$
(14)

Each of the portions of the material tangent stiffness matrix ([D']) can be expressed in a discrete form using the procedure presented by Zhang et al. [13,14] (Fig. 2b), see Eq. (15),

$$\begin{bmatrix} D'_{m} \end{bmatrix} = \int_{-\frac{t}{2}}^{\frac{t}{2}} \left[D(z') \right] dz' = \sum_{i=1}^{N_{c}} \left[D'_{c_{i}} \right] \left(z'_{i+1} - z'_{i} \right) + \sum_{j=1}^{N_{s}} \left[D'_{s_{j}} \right] t_{s_{j}}$$

$$\begin{bmatrix} D'_{b} \end{bmatrix} = \int_{-\frac{t}{2}}^{\frac{t}{2}} z'^{2} \left[D(z') \right] dz' = \frac{1}{3} \sum_{i=1}^{N_{c}} \left[D'_{c_{i}} \right] \left(z'_{i+1}^{3} - z'^{3}_{i} \right) + \sum_{j=1}^{N_{s}} \left[D'_{s_{j}} \right] t_{s_{j}} z'^{2}$$

$$\begin{bmatrix} D'_{mb} \end{bmatrix} = \int_{-\frac{t}{2}}^{\frac{t}{2}} z' \left[D(z') \right] dz' = \frac{1}{2} \sum_{i=1}^{N_{c}} \left[D'_{c_{i}} \right] \left(z'_{i+1}^{2} - z'^{2}_{i} \right) + \sum_{j=1}^{N_{s}} \left[D'_{s_{j}} \right] t_{s_{j}} z'_{j}$$

$$(15)$$

where $[D'_{ci}]$ and $[D'_{sj}]$ are the plane stress material stiffness tangent of the ith concrete layer and jth Steel layer, respectively. N_c and N_s are the number of concrete and steel layers, respectively. z'_{i+1} and z'_i is the position of the top and bottom part of each concrete layer and t_{sj} is the thickness of the jth steel layer.

In addition, the internal resisting force (R) for the thin flat shell element, can be obtained using a similar approach, and assuming zero initial stress, as:

$$R = \int_{A} [B]^{T} \{\hat{\sigma}'\} dA = \int_{A} [B]^{T} \begin{cases} n_{x'} \\ n_{y'} \\ n_{x'y'} \\ \cdots \\ m_{x'} \\ m_{x'y'} \\ m_{x'y'} \\ m_{x'y'} \end{cases} dA$$
(16)

where, $\{\sigma'\}$ is the stress and can be calculated in a discrete manner, as follow:

$$n_{x'} = \int_{-\frac{t}{2}}^{\frac{t}{2}} \sigma'_{x} dz' = \sum_{i=1}^{N_{c}} \sigma_{x'_{i}}^{c} \left(z'_{i+1} - z'_{i} \right) + \sum_{j=1}^{N_{s}} \sigma_{x'_{j}}^{s} t_{s_{j}}$$

$$n_{y'} = \int_{-\frac{t}{2}}^{\frac{t}{2}} \sigma'_{y} dz' = \sum_{i=1}^{N_{c}} \sigma_{y'_{i}}^{c} \left(z'_{i+1} - z'_{i} \right) + \sum_{j=1}^{N_{s}} \sigma_{y'_{j}}^{s} t_{s_{j}}$$

$$n_{x'y'} = \int_{-\frac{t}{2}}^{\frac{t}{2}} \tau_{x'y'} dz' = \sum_{i=1}^{N_{c}} \tau_{x'y'_{i}}^{c} \left(z'_{i+1} - z'_{i} \right) + \sum_{j=1}^{N_{s}} \tau_{x'y'_{j}}^{s} t_{s_{j}}$$

$$m_{x'} = \int_{-\frac{t}{2}}^{\frac{t}{2}} z' \sigma'_{x} dz' = \frac{1}{2} \sum_{i=1}^{N_{c}} \sigma_{x'_{i}}^{c} \left(z'_{i+1}^{2} - z'_{i}^{2} \right) + \sum_{j=1}^{N_{s}} z'_{j} \sigma_{x'_{j}}^{s} t_{s_{j}}$$

$$m_{y'} = \int_{-\frac{t}{2}}^{\frac{t}{2}} z' \sigma'_{y} dz' = \frac{1}{2} \sum_{i=1}^{N_{c}} \sigma_{y'_{i}}^{c} \left(z'_{i+1}^{2} - z'_{i}^{2} \right) + \sum_{j=1}^{N_{s}} z'_{j} \sigma_{y'_{j}}^{s} t_{s_{j}}$$

$$m_{x'y'} = \int_{-\frac{t}{2}}^{\frac{t}{2}} z' \tau_{x'y'} dz' = \frac{1}{2} \sum_{i=1}^{N_{c}} \tau_{x'y'_{i}}^{c} \left(z'_{i+1}^{2} - z'_{i}^{2} \right) + \sum_{j=1}^{N_{s}} z'_{j} \tau_{x'y'_{j}}^{s} t_{s_{j}}$$



3. Material constitutive models

This section describes the constitutive models used to define the concrete and steel layers in this work.

3.1 Formulation of the concrete constitutive model

For the concrete layers, an orthotropic model formulation rotating angle, was used. In this formulation, it is assumed that the axes orthotropy coincide with the principal axes of deformation. In addition, the model of concrete incorporating features of the models proposed by the group at the University of Houston (CSMM developed by Mansour and Hsu [16] and [17] in 2005) and the group at the University of Toronto (cyclic model developed by Palermo and Vecchio [18]), and incorporate Poisson ratio under biaxial loads in compression developed by Vecchio [19] in 1992, and coefficients biaxial strength and reducing the compressive strength of concrete due to the tensile strain (softening compression). The assumptions used in this study to formulate the concrete material are the following:

- The main directions of stress and strain coincide.
- The stress-strain relationship can be represented by the average stress-strain relationship.
- The constitutive model of concrete in each of the principal directions of stress, can be represented by a uniaxial concrete model.
- The Poisson's ratio is neglected after cracking.

For the uniaxial constitutive model of the concrete, the model proposed by Massone [20] in 2006 with some modification to incorporate the cyclic behavior was used. In the model, the compression envelope was defined with the curve defined by Thorenfeldt et al. [21] in 1987, and later calibrated by Collins and Porasz [22] in 1989 (Fig. 3a). The envelope of tension, implemented by Massone [20], is the proposal by Belarbi and Hsu [23] in 1994, which is divided into two sections, pre and post cracking. Before cracking, linear interpolation is proposed, and after cracking, a descending branch to include concrete stress (tension stiffening), see Fig. 3b.





Fig. 3 – Constitutive model for concrete in compression and tension

Fig. 4 – Cyclic Constitutive model for concrete



The proposed cyclical behavior, use a linear representation of loading and unloading compression, connected to each other, also with a linear equation with slope equal to the initial stiffness of concrete (Ec), see Fig. 4. This constitutive model is used for confined and unconfined concrete, simply by varying the parameters that defining the model to be able to incorporate the effects of confinement.

3.2 Formulation of the steel constitutive model

The steel material is based on the assumption that the steel bars are considered as a homogeneous material within the wall, and the variation of stresses due to cracking over an area can be modeled using the average stresses and strains in the steel [10]. The assumptions used in this study to formulate the steel material are the following:

- Stress-strain relationship can be represented by the average stress-strain relationship of steel bars embedded in the concrete.
- Steel is considered homogeneous (smeared) and acts only along the direction of its orientation
- Concrete and smeared steel are considered fully bonded.

For the uniaxial constitutive model of steel were used two models. The first, it was the model proposed by Menegotto and Pinto [24] in 1973 and later modified by Filippou et al. [25] in 1983, see Fig. 5. This model is mainly used to represent the behavior of the longitudinal and transverse bars located in the center of walls or the rebars that not suffer buckling during analysis.



Fig. 5 - Constitutive Menegotto-Pinto model for steel

The second model used to represent the behavior of the longitudinal bars at the edges of walls that are susceptible to buckling, is the model proposed by Massone and Moroder [26] in 2009, which incorporates the buckling of the bars using an initial imperfection, see Fig. 6.



Fig. 6 – Bar buckling model (After Massone and Moroder [26])



4. Evaluation and verification

In this section, the evaluation of the accuracy and applicability of the layered membrane and shell element in modelling RC wall is presented. To verify the formulation, the results of a set of available experimental data reported in the literature for RC wall elements, with different configurations (slender walls, and wall with irregular disposition of openings) and levels of confinement, under reversed loads are compared with the results obtained from the corresponding analytical model.

In all the models presented next, the tensile strength of the concrete (f_{cr}) was considered equal to $0.31\sqrt{f_c'}$ [MPa], and the tension strain ε_{cr} at the maximum tensile strength was equal to 0.00008 [mm/mm]; moreover, the maximum compressive strength of concrete (f_c') reported for each experiment, and a compression strain (ε_{co}) between -0.0025 and -0.0055 [mm/mm] at the maximum compressive strength of concrete depending on the level of confinement, was used. In addition, for steel the f_y reported for each experiment, and a base value of 1% for the hardening ratio, was used. Also, a 3-by-3 Gauss integration, and a mesh of around 150 [mm] by 150 [mm] in each element for the analysis of the analytical model were used. The analytical models were analyzed using the displacement control solution algorithm with pseudo-constant incremental steps, which indicate that if the analysis has not reached convergence for an increment of displacement, the increment is reduced until the initial increment has passed; this is a variation of the algorithm developed by Batoz and Dahtt [27] in 1979.

4.1 Evaluation for the membrane element

For the membrane element, this study used the experimental results of the rectangular wall RW2 reported by Thomsen and Wallace [28] in 1995 and the experiment result of a RC wall with irregular disposition of openings reported by Yañez [29] in 1993, both test under reversal loading. Fig. 7 and Fig. 8 show the geometry and results of the comparison between the experiment and the analytical model for RW2







Fig. 8 – Comparison of load vs top displacement curve and vertical strain at the base of specimen wall RW2



As mentioned before, the other test used to validate the model with membrane element was the wall S4 tested by Yañez [29] in 1993, only in this model the steel constitutive law, which incorporated bar buckling, was used. Fig. 9 and Fig. 10 show the geometry and results of the comparison between the experiment and the analytical model for S4.



Fig. 9 - Geometry and comparison of load vs top displacement curve of specimen wall S4



Fig. 10 - Comparison of the vertical strain at the base of specimen wall S4 including bar buckling

4.2 Evaluation for the thin flat shell element

For the shell element, this study used the experimental results of the TW2 (T-shaped) reported by Thomsen and Wallace [28] in 1995. Fig. 11 show the geometry and reinforcement details of specimen TW2. Fig. 12 show the comparison of the load vs top displacement between the experiment and the analytical model of TW2.



Fig. 11 – Geometry and reinforcement details of specimen wall TW2 [28]





Fig. 12 - Comparison of load vs top displacement curve of specimen wall TW2

5. Conclusion

In this paper, the development and verification of a quadrilateral layered membrane element with drillings degrees of freedom (DOF) and a thin flat layered shell element for nonlinear analysis of reinforced concrete walls (MHA) under cyclic loads are presented. The mixed interpolation used in this study to represent the displacement within the membrane element, it is an improvement proposed by Rojas [10] to the interpolation defined by Xia et al. [11] in 2009. In addition, to generate the shell element, the membrane element is combined with the Discrete Kirchhoff Quadrilateral Element (DKQ, 12 DOF), formulated by Batoz and Tahar [12], in 1982, to represent the out of plane bending. Steel mesh inside the wall is modeled assuming it is a homogenous layer within the element, which requires consider an average stress-strain model in the direction of the bar. To represent the uniaxial stress-strain curve in the direction of the bar, the steel model proposed by Menegotto and Pinto [24] and the steel model proposed by Massone and Moroder [26] in 2009, which includes bar buckling, are used. The smeared crack approach using an orthotropic model with equivalent uniaxial average stress-strain relations along the axes of orthotropy was selected to represent the concrete plane stress behavior. The uniaxial stress-strain relationship for concrete is the proposed by Massone [20] in 2006 with some modifications. In the last section, the accuracy, applicability and validation of the proposed elements are presented. In conclusion, the elements show an excellent agreement with experimental data, and they are able to reproduce the global and local behavior at different drift levels for walls with different configurations under cyclic loads.

6. Acknowledgements

This study was partially financially supported by Chile's National Commission on Scientific and Technological Research (CONICYT) for the Fondecyt Initiation into research 2014 project N° 11140429.

7. References

- [1] ACI COMMITTEE 318. 2008. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. Farmington Hills, MI, USA, American Concrete Institute.
- [2] Paulay, T., Priestley, M.J.N. (1992) Seismic Design of Reinforced Concrete and Masonry Buildings. John Wiley and Sons, Inc.
- [3] Bertero, V.V. (1980) Seismic behaviors of rc wall structural system. In: 7th World Conference on Earthquake Engineering; vol. 6. Instalbul, Turkey, p. 323–330.
- [4] Orakcal, K., Massone, L.M., Wallace, J.W. (2006) Analytical modeling of reinforced concrete walls for predicting flexural and coupled-shear-flexural responses. *PEER Report 2006/07*; Pacific Earthquake Engineering Research Center. University of California, Los Angeles.
- [5] Eimani, T. (1997) Analytical model for reinforced concrete shear wall struc- tures. *Phd dissertation*; University of Southern California; Los Angeles, California.
- [6] Naeim, F., Lew, M., Carpenter, L.D., Youssef, N.F., Rojas, F., Saragoni, G.R., et al. (2011) Performance of tall buildings in santiago, chile during the 27 february 2010 offshore maule, chile earthquake. *The Structural Design of Tall and Special Buildings*; 20(1):1–16.



- [7] Rojas, F., Naeim, F., Lew, M., Carpenter, L.D., Youssef, N.F., Saragoni, G.R., et al. (2011) Performance of tall buildings in concepcio'n during the 27 february 2010 moment magnitude 8.8 offshore maule, chile earthquake. *The Structural Design of Tall and Special Buildings*; 20(1):37–64.
- [8] Moehle, J., Wallace, J.W., Maffei, J., Sempere, C., Celestino, A., Besa, J.J., et al. (2010) February 27, 2010 chile earthquake reconnaissance team investigation: Reinforced concrete buildings. *Reconnaissance Report*; EERI.
- [9] Vulcano, A., Bertero, V.V. (1987) Analytical models for predicting the lateral response of rc shear walls: Evaluation of their reliability. UCB/EERC 87/19; Earthquake Engineering Research Center. University of California, Berkeley.
- [10] Rojas, F. (2012): Development of a nonlinear quadrilateral layered membrane element with drilling degrees of freedom. *Phd Dissertation*, University of Southern California, Los Angeles, California.
- [11] Xia, G., Yu, M., Li, C., Zhang, J. (2009): Rectangular membrane element with rotational degree of freedom. *Computational Structural Engineering, Proceedings of the International Symposium on Computational Structural Engineering.* Shanghai, China; p. 1051–1058.
- [12] Batoz, J. L. and Tahar, M. (1982) Evaluation of a new quadrilateral thin plate bending element. *International Journal for Numerical Methods in Engineering*, 18(11):1655–1677.
- [13] Zhang, Y.X., Bradford, M.A., Gilbert, R.I. (2007a) A layered shear-flexural plate/shell element using timoshenko beam functions for nonlinear analysis of reinforced concrete plates. *Finite Elements in Analysis and Design*; 43.
- [14] Zhang, Y.X., Bradford, M.A., Gilbert, R.I. (2007b) A layered cylindrical quadrilateral shell element for nonlinear analysis of rc plate structures. *Advances in Engineering Software*;38(7):488 – 500.
- [15] Oñate E. (1992) CISM Course and Lectures No. 328. International Centre for Mechanical Sciences: Nonlinear Analysis of Shells by Finite Elements, chapter Nonlinear Finite element Analysis of Concrete, pages 195–256. Springer-Verlag-Wien.
- [16] Mansour, M., Hsu, T.T.C. (2005) Behavior of reinforced concrete elements under cyclic shear. i: Experiments. *Journal of Structural Engineering*; 131(1):44–53.
- [17] Mansour, M., Hsu, T.T.C. (2005) Behavior of reinforced concrete elements under cyclic shear. ii: Theoretical model. *Journal of Structural Engineering*; 131(1):54–65.
- [18] Palermo, D., Vecchio, F.J. (2003): Compression field modeling of reinforced concrete subjected to reversed loading: Formulation. ACI Structural Journal;100(5):616–625.
- [19] Vecchio, F.J. (1992): Finite element modeling of concrete expansion and confinement. *Journal of Structural Engineering*;118(9):2390–2406.
- [20] Massone, L.M. (2006): Rc wall shear flexure interaction: Analytical and experimental responses. *Phd Dissertation*; University of California, Los Angeles, California.
- [21] Thorenfeldt, E., Tomaszewicz, A., Jensen, J.J. (1987): Mechanical properties of high-strength concrete and application in design. *Symposium Utilization of High-Strength Concrete*. Stavanger, Norway.
- [22] Collins, M.P., Porasz, A. (1989) Shear Strength for High Strength Concrete; 66 chap. Bulletin D'Information No. 193 -Design Aspects of High Strength Concrete. Paris, France, p. 75–83.
- [23] Belarbi, A., Hsu, T.T.C. (1994): Constitutive laws of concrete in tension and reinforcing bars stiffened by concrete. *ACI Structural Journal*;91(4):465–474.
- [24] Menegotto, M., Pinto, P.E. (1973): Method of analysis of cyclically loaded reinforced concrete plane frames including changes in geometry and non- elastic behavior of elements under combined normal force and bending. *IABSE Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well-Defined Repeated Loads*.
- [25] Filippou, F.C., Popov, E.G., Bertero, V.V. (1983): Effects of bond deterio- ration on hysteretic behavior of reinforced concrete joints. UCB/EERC83/19; Earthquake Engineering Research Center, University of California, Berkeley.
- [26] Massone, L. M., Moroder, D. (2009): Buckling modeling of reinforcing bars with imperfections. *Engineering Structures*; 31, 758 767.
- [27] Batoz, J.L., Dhatt, G. (1979) Incremental displacement algorithms for non- linear problems. *International Journal for Numerical Methods in Engineering*; 14(8):1262–1267.
- [28] Thomsen J. H., I., Wallace, J.W. (1995): Displacement-based design of rein- forced concrete structural walls: An experimental investigation of walls with rectangular and t-shaped cross sections. *Report No. CU/CE-95-06*; Clarkson University.
- [29] Yáñez, F. (1993) Seismic Behaviour of R.C. Walls with Irregular Openings. PhD thesis, University of Canterbury.