# NONLINEAR FINITE ELEMENT ANALYSIS OF CONFINED MASONRY WALLS

by

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## ABSTRACT

Confined masonry walls were analyzed, and the analytical results gave reasonable agreement as compared with the experimental results of full scale walls, tested in 1991 at CISMID-PERU, taking account of the scattering behavior of masonry structures.

The analysis was carried out by using Finite Element Method with incremental hypo elastic models; the constitutive law of concrete was based on the orthotropic model with the concept of equivalent uniaxial strain proposed by Darwin and Pecknold. The masonry was modelled as an uniform material in the same way as concrete.

Longitudinal reinforcement of a surrounding frame was modelled as bars and stirrups as smeared layer elements. Bond between concrete and steel was represented by bond link elements. The connection between masonry and the surrounding frame was idealized using two orthogonal link elements, where the shear connector depends always on the compressive normal stresses. The same model was considered for the joint between masonry brick elements. Cracks in the whole wall were represented by a smeared crack model, and cracks in the bottom part of the columns were represented by a discrete crack model.

Finally macroscopic models were applied for determining the ultimate shear strength of confined masonry walls. Despite of its simplicity they gave good results.

## 1 INTRODUCTION

Confined masonry walls are the most popular structural element for housing units in urban areas in Latin America. Large housing projects of masonry buildings have been made in the past decades and masonry structures are expected to be used continuously in the future. Many research programs were already started such as experimental tests, in order to understand the confined masonry behavior mainly against the seismic actions. The term confined masonry denote a wall system where masonry using clay bricks and mortar is surrounded by a reinforced concrete frame.

Analytical studies parallel to experimental tests are considered to be important in order to clarify the behavior of masonry. Nonlinear FEM studies were already begun by several researchers. Most of these studies have been made considering brick units separately to the mortar using an interface model, and the recently study with this consideration was given by Lotfi and Shing[9].

On live from National University of Engineering, Faculty of Civil Engineering, Structural Division & CISMID Structural Laboratory.

Related to confined masonry walls, due to an enormous number of brick units, masonry should be considered as a material with uniform characteristics. FEM studies in this field have been recently started; in Peru at National University of Engineering[14] in 1993, plastic models were included for masonry and concrete, following Drucker Prager yield criteria; in Obayashi-gumi Corporation[8] in 1993, a more refined analysis was carried out having a special care in the connection between masonry and a surrounding frame.

Macroscopic models based on the plasticity theory had been recently included in AIJ Guidelines for Reinforced Concrete Structures[1]. Researches in FEM microscopic analysis are being done in order to verify, modify and propose macroscopic models. One of these studies was given by Shohara R., Noguchi H. and Shirai N.[16], they verified the AIJ Guidelines by using FEM analysis.

Full scale experimental tests of confined masonry walls have already been done at the Japan Peru Center for Earthquake Engineering Research and Disaster Mitigation (CISMID) of the National University of Engineering, in Lima-PERU. On the other hand nonlinear finite element analysis of reinforced concrete have been making great strides. The current study has been carried out by using FEM program for RC structures developed at Noguchi Laboratory of Chiba University. The test results were used for the comparison with the FEM analysis.

## 2 TEST PROGRAM

### 2.1 MATERIALS

Nominal dimensions of the clay brick units used were 250mmx125mmx90mm with a net area of 61% as shown in Figure 1. They were laid with a 1:4 (cement-sand) mortar, with an average joint thickness of 10mm. The nominal masonry prism strength was 50 kg/cm$^2$.

The concrete used for columns and beam had a nominal strength of 210 kg/cm$^2$ and was reinforced with 4#3 longitudinal bars, stirrups in columns were #2 at 25cm spacing except in the top and bottom parts at 10cm spacing was used, while in the top beam stirrups were distributed at 20cm spacing and in the ends parts at 10cm spacing. The yielding stress of longitudinal bars was 4200kg/cm$^2$ and for stirrups was 2800kg/cm$^2$.

### 2.2 SPECIMENS

Confined masonry is a composite structure by masonry, established first and surrounded by concrete elements which are casted at last.

Three kinds of confined masonry walls were tested, two specimens per each one, remaining the height (H) the aspect ratio (H/L) was the main objective of the mentioned experimental program. The full scale specimens (tested at CISMID) were based on Peruvian Standards for Confined Masonry. Configuration and bar arrangements of the specimens are shown in Figure 2 and dimensions in Table 1.

### 2.3 LOAD APPLICATION

The experiment was carried out under static loading control. The lateral forces were applied by reversed cyclic loading, remaining constant the vertical load for all specimens which was applied by using a steel beam connected to a concrete beam in order to distribute the load application. The loading setup is shown in Figure 3 and vertical load application in Table 2.

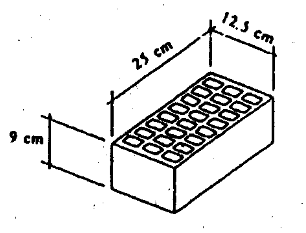



Figure 1. Clay brick unit.

Table 1. Specimen dimensions (cm).

| TYPE | h | L | t |
|---|---|---|---|
| MEL | 240 | 180 | 12.5 |
| MCI | 240 | 240 | 12.5 |
| MEC | 240 | 360 | 12.5 |

Table 2. Vertical Load (tf).

| TYPE | MEL | MCI | MEC |
|---|---|---|---|
| VERTICAL LOAD | 13.5 | 18.0 | 27.0 |

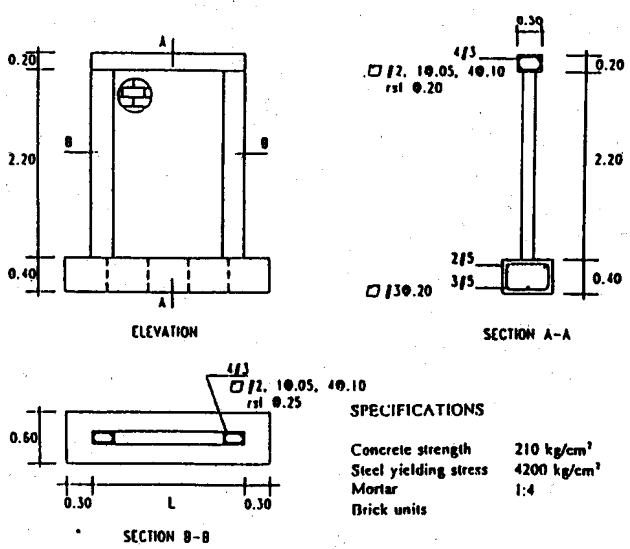



Figure 2. Plan of specimens.

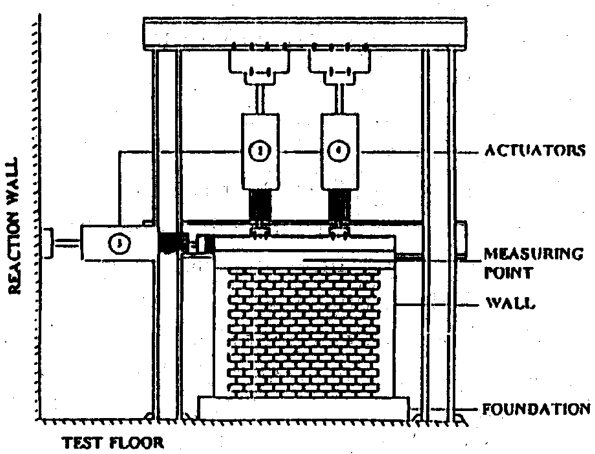



Figure 3. Load set up.

# 3 ANALYTICAL MODELS

## 3.1 MODELLING OF A TEST SPECIMEN

The confined masonry walls were modelled by using the FEM program developed at Noguchi Laboratory (Department of Architecture in Chiba University). The FEM program which has been applied to static nonlinear finite element analysis of reinforced concrete structures, takes account of the material nonlinearities. When the program is applied to confined masonry structures it is necessary to model reasonably.

As was described before, based on the shape of the specimens and the loading characteristics, the plane stress condition was assumed.

Due to an enormous number of elements units which would be needed to model the bricks and joints individually, the masonry itself was modelled as a material with uniform characteristics (masonry pile). Regarding to the mesh selected for these analysis as shown in Figure 13, the masonry elements in horizontal direction were jointed two by two making a stairs shape in the vertical direction and simulating masonry assembly. This mesh was adopted because this configuration provided larger horizontal displacement compared with the vertical one as was clearly observed in the test.

The joints in the masonry wall were replaced by two-node orthogonal spring elements and distributed in both directions, vertical and horizontal. Similarly the surrounding frame and brick walls were connected by two-node linkage elements consisting of two orthogonal springs, because some separation and slippage were expected in the boundary surface between brick walls and the surrounding frame.

An elastic element which did neither crack nor yield were adopted for the foundation slab. The effects of the foundation slab was included in the analysis because flexural cracking in the bottom part of the walls was observed in the test.

For applying the vertical load, a rigid element was added above the concrete beam. Then the axial loads were applied to the rigid element (Fig.13). This was included because the masonry strains are larger than concrete strains then against same condition of axial loads excessive deformation in the masonry would be found.

## 3.2 MODELLING OF MATERIAL CHARACTERISTICS

### 3.2.1 Models for Concrete

The complicated stress-strain behavior of concrete under different stress states can be divided in four main groups: representations of established stress strain curves by mathematical procedures, linear and nonlinear elastic theories, perfect and work hardening plasticity theories and the endochronic theory of plasticity. Within nonlinear elastic theories two models are known, hypoelasticity model such as orthotropic model of Darwin and Pecknol (incremental stress and strain, tangent stiffness) and hiperelasticity model (total stress and strain, secant stiffness).

**Orthotropic model of Darwin and Pecknold**

The constitutive law is obtained assuming an "equivalent" Poisson's ratio for principal stresses equal to $\nu^2 = \nu_1 \nu_2$ and shear modulus is an invariant.

$$D_c = \frac{1}{1 - \nu^2} \begin{bmatrix} E_1 & \nu\sqrt{E_1 E_2} & 0 \\ \nu\sqrt{E_1 E_2} & E_2 & 0 \\ 0 & 0 & 1/4(E_1 + E_2 - 2\nu\sqrt{E_1 E_2}) \end{bmatrix} \tag{1}$$

This model introduces the uniaxial equivalent strain concept for each principal direction (i=1,2)

$$\sigma_{iu} = \int \frac{d\sigma_i}{E_i} = \sum_{j=1}^{n} \frac{\Delta\sigma_i}{E_i} \quad \text{or} \quad \Delta\varepsilon = \frac{\sigma_{inew} - \sigma_{iold}}{E_i}$$

where Ei is calculated by equation (2) suggested by Saenz for concrete with normal strength.

$$\sigma_i = \frac{E_o \varepsilon_{iu}}{1+(\frac{E_o}{E_s}-2)\frac{\varepsilon_{ic}}{\varepsilon_{iu}}+(\frac{\varepsilon_{ic}}{\varepsilon_{iu}})^2} \tag{2}$$

As for the maximum values Kupfer and Gerstle's failure surface is used.

$\alpha = \sigma_1/\sigma_2$ and $\sigma_1 > \sigma_2$

1.Compression-Compression

$$\sigma_{2c} = \frac{1+3.65\alpha}{1+\alpha^2} f_c, \quad \sigma_{1c} = \alpha\,\sigma_{2c} \tag{3}$$

2. Tension-Compression

$$\sigma_{1t} = (1-0.8\frac{\sigma_2}{f_c})f_t \tag{4'}$$

$$\sigma_{2c} = \frac{1+3.28\alpha}{1+\alpha^2} f_c, \quad \sigma_{1c} = \alpha\,\sigma_{2c} \tag{5}$$

3.Tension-Tension

$$\sigma_{1t} = f_t \qquad \sigma_{2t} = f_t \tag{6}$$

Determination of $\varepsilon_{ic}$

if $\sigma_{ic} < f_c$

$$\varepsilon_{ic} = \varepsilon_{cu}[\frac{\sigma_{ic}}{f_c}R-(R-1)], \quad R=3.15$$

$$\varepsilon_{ic} = \varepsilon_{cu}[-1.6(\frac{\sigma_{ic}}{f_c})^3+2.25(\frac{\sigma_{ic}}{f_c})^2+0.35(\frac{\sigma_{ic}}{f_c})] \tag{7}$$

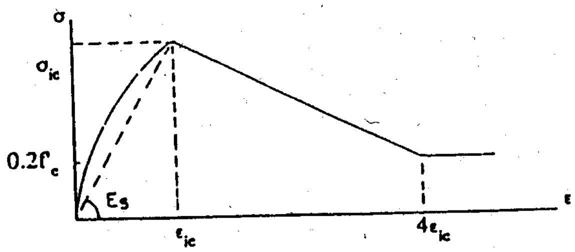



Figure 4. Uniaxial stress-strain relationship for concrete.

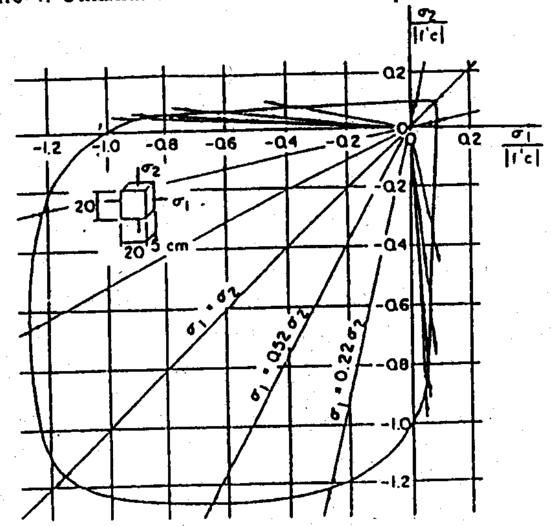



Figure 5. Biaxial strength envelope of concrete.

Unificated Poisson's ratio: ( $\nu < 0.99$ )

$\nu = 0.2$ for tension-tension and compression-compression

$$\nu = 0.2+0.6(\frac{\sigma_2}{f_c})^4+0.4(\frac{\sigma_1}{\sigma_{1t}})^4 \text{ for uniaxial compression and tension-compression} \tag{8}$$

**3.2.2 Models for steel**

As steel elements compared with concrete elements have a small transversal section, steel is considered as uniaxial element, dowel effect or another transversal effects will be considered in other way.

**Discrete model.** Steel bars are connected by mean of links to finite element nodes, without through elements. Link represents bond and dowel effect, and springs are not necessary for perfect bond. This model is appropriate when many amount of steel is concentrated in some zones.
**Embedded model.** Steel is regard as bars and its distribution is independent of FEM mesh, so that steel bars can pass through FEM elements.
**Smeared (multi layered element).** In this case steel is regarded distributed in zones. Stiffness contribution is only along steel bars direction.

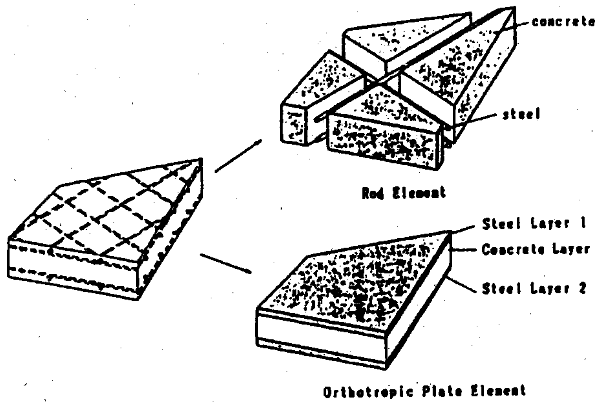



Figure 6. Modelling of a reinforced concrete element.

Related to steel behavior, several models are also established, elasto plastic model, where the behavior is considered elastic up to the yielding point and then after completely plastic, another models such as a trilinear model, which follows the steel behavior in three straight lines or Steven's model, similar to the trilinear model, where the second line obey an exponential curve.

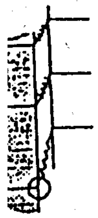
### 3.2.3 Link Elements

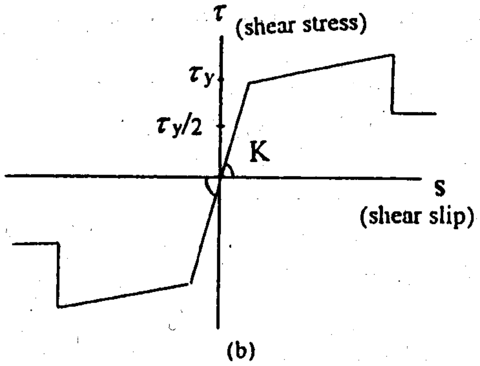
#### Bond and Slip

How the bond stresses are transferred from the concrete to the steel is established mainly making basic tests. This effects have been investigated by many researchers such as Noguchi H. [6] where bond stress and shear slip relationships were determined thus;: when the bond stress reaches the maximum bond strength or when a longitudinal bars yields, half of the bond stress is released and the bond stiffness is set to zero, as shown in Figure 7. The spring stiffness of a bond link normal to the bar axis is assumed to be linear an elastic. Then these two orthogonal link elements assemble a stiffness matrix thus:

$$\begin{bmatrix} Fr \\ Fs \end{bmatrix} = \begin{bmatrix} Kr & 0 \\ 0 & Ks \end{bmatrix} \begin{bmatrix} dr \\ ds \end{bmatrix} \qquad (9)$$

where following the bar direction Kr represent bond effect and Ks the dowel effect.



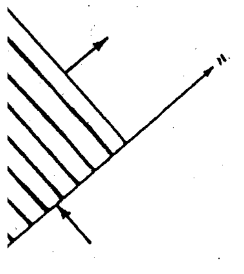

nal link element. (a) Idealization (b) Bond stress - shear slip relationship.

:oncrete is characterized by a gradual growth of cracks, being one of ies. Cracks are perpendicular to the tensile principal stress and open en under monotonic loads when stress redistribution occur.

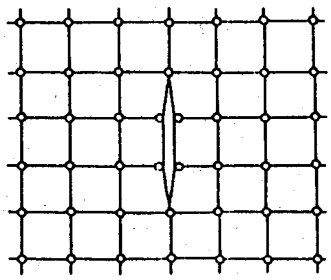
**el.** where cracks implies an infinite number of parallel fissures across ient, this model is appropriate for structures where many cracks are

**l.** cracks are prelocated, then one obvious difficulty in such approach rientation of the cracks are not known in advance. Previous elastic ests are required.

ıg model, the cracking criteria is judged in different ways. (1)Crack stress attains its tensile strength. (2)When the principal strains attains ıg different rules after stress reaches the tensile strength.



:racking.



(b) Discrete cracking.

Figure 8. Idealization of cracking models.

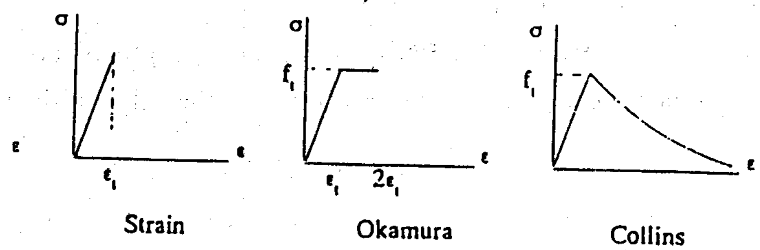



Figure 9. Cracking criteria of concrete