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PhD Thesis

SHAKING TABLE TESTS ON RC SHEAR WALLS: SIGNIFICANCE OF NUMERICAL MODELING

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Paolo Martinelli
Summary

In the present study two different reinforced concrete shear walls, subjected to a sequence of input ground motions, have been studied. The walls are representative of a wide range of structures existing around the world. The efficiency of several structural modelings in the non-linear range have been investigated through the comparison with the experimental results.

The first wall, named CAMUS I, is a 1/3 scale 5-story lightly reinforced shear wall, tested on the shaking table Azalée (France) within the Camus Research Program. The structure, composed of two equal and parallel reinforced concrete (RC) shear walls, connected through rigid diaphragms, has been designed according to the multi-fuse concept, allowing the yielding of the longitudinal reinforcement at each story and not only at the structure base. The structure has been subjected to four accelerograms in sequence; a fifth accelerogram caused the structural collapse. The wall has been modeled at two different refinement levels, through a spread plasticity beam element (macro scale) and a fiber beam-column element (meso scale). The macro scale accounts for the material behavior in terms of moment-curvature relation, and relies on a flexibility approach. The meso scale follows the material constitutive behavior in terms of stress-strain relations. Dynamic step-by-step analyses have been carried out for all the modeling approaches. Modal and spectral analyses have been also performed.

The second wall, named NEES-UCSD, is a full scale vertical slice of a seven story reinforced concrete wall building. Part of the results that will be presented in this study have served as the UC Berkeley entry to the “Seven Story Building Slice Earthquake Blind Prediction Contest” of the shaking table tests that were performed at the UCSD’s Engelkirk Structural Engineering Center (San Diego). The shear wall building has been subjected to four earthquake ground motions of increasing intensity up to 0.93 g maximum acceleration, that has caused increasing damage in the structure up to pronounced non-linear hysteretic behavior. The strategy for the simulation of the non-linear behavior of the structure is based on a 2D model with a fiber beam-column element representing the shear wall. Three different types of analyses have been performed on the structural model: a standard linear modal and spectral analysis, a non-linear static pushover analysis, and finally a non-linear
dynamic analysis.

In this work it has been shown that this kind of experimental tests, characterized by a extremely high costs, are not limited to give insight into the behavior of the structures, through the comparison of the numerical simulation results with the measured experimental data, but can represent a unique occasion to measure the efficiency (capabilities and limitations) of analysis procedures and modeling strategies.
Sommario

Nel lavoro di ricerca qui presentato è stato studiato il comportamento di due pareti a taglio in cemento armato (CA). Le pareti sono state soggette ad una sequenza di input sismici su tavola vibrante; per le loro caratteristiche geometriche e meccaniche le pareti in studio possono essere considerate rappresentative di un’ampia classe di strutture esistenti. Il confronto tra simulazioni numeriche e dati sperimentali ha permesso di investigare l’efficienza di diverse modellazioni strutturali in campo non lineare.

La prima parete a taglio, chiamata CAMUS I, è un provino di 5 piani in scala 1/3 debolmente armato; la parete è stata testata in Francia all’interno del programma di ricerca Camus. La struttura, composta da due pareti uguali e parallele tra loro, connesse attraverso diaframmi rigidi presenti ad ogni interpiano, è stata progettata secondo la filosofia del multi-fuse design, che permette lo snergamento dell’armatura longitudinale ad ogni piano e non solo alla base della struttura. La struttura è stata sottoposta ad una sequenza di 4 accelerogrammi seguiti da un ultimo accelerogramma che ne ha causato il collasso. La parete è stata modellata a due diversi livelli di accuratezza attraverso un elemento a plasticità diffusa (macro scala) e un elemento a fibre (meso scala). La macro scala descrive in comportamento del materiale a livello sezionale attraverso una relazione momento-curvatura; l’elemento è formulato in termini di matrice di flessibilità. La meso scala dell’elemento a fibre, anch’essa formulata in termini di flessibilità, descrive il comportamento del materiale attraverso una relazione locale sforzo-deformazione. Per entrambi gli approcci di modellazione analizzati è stata eseguita un’analisi dinamica passo-passo; inoltre sono state eseguite analisi spettrali e modali standard.

La seconda parete, chiamata NEES-UCSD, è un provino in scala reale rappresentativo di una parte di edificio di 7 piani in CA. Parte dei risultati presentati in questo lavoro sono stati utilizzati per la partecipazione dell’UC Berkeley al “Seven Story Building Slice Earthquake Blind Prediction Contest”. Le prove sperimentali su tavola vibrante sono state eseguite al UCSD’s Engelkirk Structural Engineering Center (San Diego). L’edificio è stato sottoposto ad una sequenza di 4 accelerogrammi aventi intensità crescente fino al valore massimo di 0.93 g che ha causato danni crescenti alla struttura fino ad un pronunciato comportamento non lineare. La
strategia per la simulazione del comportamento non lineare della struttura è basata su un modello a fibre trave-colonna 2D. Tre diverse analisi sono state eseguite sul modello strutturale: un’analisi modale e spettrale, un’analisi statica non lineare di tipo pushover e un’analisi dinamica non lineare.

Lo studio ha mostrato come questo genere di prove sperimentali, caratterizzate da costi estremamente elevati, non si limita a fornire indicazioni sul comportamento delle strutture in esame con evidenti ricadute sul mondo della progettazione, ma rappresenta, attraverso il confronto tra risultati numerici e dati sperimentali, un’occasione unica per testare l’efficienza (potenzialità e limiti) delle procedure d’analisi e delle strategie di modellazione.
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Chapter 1

Introduction

The availability of large scale testing facilities all around the world has given a strong impulse to the experimental research in the field of earthquake engineering. Many laboratories, containing reaction walls and/or shaking tables, have risen in U.S., Europe, and Japan. The range of experimental data has covered not only structural elements but also large or full scale structures. In the same time, due to the development of powerful computers, refined numerical models have been derived to describe the non-linear cyclic behavior of materials, together with finite elements devoted to the analysis of the seismic response of structural elements. Finally, in the last decade, the codes for the design and assessment of structures in seismic zones have evolved towards a new design philosophy, the performance based-design, following a series of strong earthquakes (Northridge 1994, Kobe 1995, Kocaeli 1999). These events have shown that the protection of human lives is not sufficient to avoid the disruption of the economic and social life. Moreover, the analyses of the effects of significant earthquakes (since the early 1980s) have shown that the seismic risks in urban areas are increasing and are far from socio-economically acceptable levels. Hence, it becomes necessary to limit structural damage, explicitly controlling the structural behavior for different levels of seismic excitation. For this reason the modern codes consider also non-linear methods of analysis, both static (pushover analysis) and dynamic (time history analysis).

However, the modeling of structures in the non-linear range still presents high complications, also for the great number of parameters that must be accounted for. The problem is even more difficult for reinforced concrete (RC) structures, due to the computational problems tied to the reproduction of the non-linear behavior of concrete, steel and bond between the two. Different refinements level can be adopted in the discretization of RC structures, from the most accurate level of the finite elements (micro scale) to the intermediate level of fiber elements (meso scale) down to the lowest level of the macro scale of global beam models (macro scale). The micro and meso scale approaches are able to follow at local level the mater-
ial constitutive relations in terms of stress-strain, and differ mainly for the richness of the kinematic field, quite general for the micro scale and tied to geometric hypotheses on the cross-section behavior for the meso scale (Bernoulli or Timoshenko beam models). The macro scale approach tracks the material behavior in terms of sectional properties, as moment-rotation or moment-curvature relations, and often relies on a flexibility approach, with a less rich kinematic field. The reduction in accuracy is paired by the smaller computational effort required, in terms of CPU time, amount and simplicity of input data required and output data provided, that are usually in terms of quantities familiar to the design engineers.

In this work, the numerical efficiency of a macro and a meso scale model in the non-linear dynamic analyses will be investigated, from the point of view of their use inside analysis procedures adopted by modern seismic codes. The research will be focused on the numerical simulation of the outcome of a series of shaking table tests on two RC shear walls, named CAMUS I and NEES-UCSD, representative of a large class of existing structures. The wall CAMUS I, is a 1/3 scale mid-rise, lightly reinforced shear wall, designed according to the multi-fuse approach. The wall NEES-UCSD, is a slender full scale wall, designed with a displacement-based capacity approach. In this way the experimental tests, highly expensive, can offer not only useful indications about the structural behavior, but also a benchmark for the modeling and analysis procedures derived by theoretical research.

A review of the tests on slender and squat walls as well as the description of the macro and meso scale models existing in literature, is presented in Chapter 2. Chapter 3 describes the modeling approaches utilized and the main characteristics of the macro spread plasticity model and of the meso fiber beam-column element. Chapter 4 and 5 are devoted to draw the walls CAMUS I and NEES-UCSD, respectively. Both these chapters have the same construction: a first section that describes the wall, a second section that describes the numerical models and a third section that presents the numerical results and compares with those experimentally obtained. The conclusions will clarify the structural behavior of the walls and point out the shortcomings and the capabilities of the adopted modeling strategies.
Chapter 2

Literature review

2.1 Generality

The structural walls can be classified in different types (MacGregor and Wight, 2005):

- Shear walls: walls that resist to wind or earthquake loads acting parallel to the plane of the wall in addition to the gravity loads from floors and roof adjacent to the wall. These walls provide lateral support for the rest of the structure.

- Bearing walls: walls that are braced by the rest of the structure and laterally supported, that resist in-plane vertical load acting downward on the top of the wall.

- Non bearing walls: walls that in-plane resist only their own weight. These walls can resist shears and moments due to pressure or loads acting on one or both sides of the wall. Examples are basement walls and retaining walls (sometimes a system of counterfort is also inserted) used to resist lateral soil pressure.

- A walls assembly: is a group of walls that are interconnected to act as a single cantilever wall to resist lateral forces. They include stairwells and elevator shafts.

- Although they are not properly walls as such, plates that resist in-plane compression, as the compression flanges or the decks of box girders bridges, show some of the characteristics of the walls.

All these walls share two frequent characteristics: their slenderness, $h_w/t$ where $h_w$ is the total height of the wall and $t$ is the wall thickness, generally higher than
for columns, and the reinforcement ratio, generally about a fifth to a tenth of those in columns.

In the present study the behavior of the shear walls without openings under seismic excitation will be considered. Shear walls can be split into two classes, squat or short shear walls with height to horizontal length aspect ratios of 2 or less and slender or flexural shear walls with aspect ratios greater than 2. In the first class fall one or two story shear walls, which generally carry lateral loads acting as a D-regions, where D stands for discontinuity or disturbed, as shown in Figure 2.1a; they can be designed using a strut-and-tied model. D-regions are characterized by a complex flow of internal stresses and include regions adjacent to discontinuities caused by abrupt changes of cross-section or the presence of concentrated loads or reactions (ASCE-ACI-445, 1998). In the second class fall the shear walls with more than 3 or 4 stories in height, the lateral load are resisted mainly by flexural action of the vertical cantilever wall (Figure 2.1b), rather than by in-plane strut-and-tie forces.

![Figure 2.1: (a) Squat and (b) slender shear wall (MacGregor and Wight, 2005).](image-url)
In the squat walls the flexural strength may be very large in comparison with the lateral forces. Because of the small aspect ratio, relatively high shear forces must be generated to develop the flexural strength at the base. Therefore, the effects of shear often strongly influence the inelastic behavior of such walls. In the slender shear walls, lateral forces are introduced by means of a series of point loads through the floor acting as diaphragms. The floor slab also stabilizes the wall against lateral buckling, and this allows using relatively thin section. In such walls it is relatively easy to ensure, when required, the development of a plastic hinge at the base with adequate plastic rotational capacity.

2.2 Failure modes

Failure modes describe the physical reason for the rupture of a structural element. Because of the different mechanical behavior of reinforcing steel and concrete, a number of failure modes can occur depending on parameters such as type of cross-section, reinforcement detailing and quantities, properties of reinforcing steel, concrete compressive strength, and boundary conditions.

Different failure modes for squat shear walls that are likely to fail in shear were reported by Paulay et al. (1982). When a diagonal corner to corner crack forms in case of insufficient amount of horizontal reinforcement a diagonal tension failure can occur (Figure 2.2a). The diagonal tension failure can also occur along a steeper failure plane as shown in Figure 2.2b. These failure mechanisms are associated to the formation of diagonal cracks and reinforcement yielding. Walls with adequate horizontal reinforcement and with an high flexural capacity may fail in diagonal compression. The concrete crushes in the compression zone near the base of the wall (Figure 2.2c). Under reversed cyclic loading, two sets of diagonal cracks appear, and concrete crushing can extend over the entire length of the wall also at a much lower shear force (Figure 2.2d). The concrete’s compressive strength is considerably reduced by transverse tensile strain and intersecting diagonal cracks, which cyclically open and close. Another reported failure mode by Paulay et al. (1982) is sliding shear (Figure 2.2e). Originated by flexure, a continuous horizontal crack develops along the base of the wall. Due to degradation of aggregate interlock, with increase in number of cycles and by the yielding of longitudinal reinforcement, the crack slip increases, and hence the wall displacements include a significant portion due to sliding, especially at the load reversals. This phenomenon results in a significant reduction of stiffness at low force intensities (pinching) that reduces energy dissipation.

Capacity design procedures, which are based on hierarchy of the failure mechanisms, and appropriate detailing of the potential plastic region, should permit to avoid brittle failure or even those with limited ductility. In this regard, it is impor-
tant that in the design of ductile shear walls, flexural yielding in clearly defined plast-
ic hinge zones could control the strength, inelastic deformation, and hence energy
dissipation in the entire structural system. Paulay and Priestley (1992) have distin-
guished several failure modes in slender shear walls. When the principal source of
energy dissipation is the yielding of the flexural reinforcement in the plastic hinge
region, normally at the base of the wall, the failure mechanism is dominated by
flexure and is similar to that shown in Figure 2.3a. Between the failure modes to be
prevented, there are those due to diagonal tension or diagonal compression caused
by shear (Figure 2.3b) that can lead to a x cracks pattern and those due to sliding
shear along construction joints (Figure 2.3c).

Figure 2.2: Shear failure in squat walls (Paulay et al., 1982).

Figure 2.3: Failure modes in slender walls.
2.3 Tests on squat shear walls

The behavior of rectangular and barbell shaped shear walls subjected to monotonic and cyclic loading was studied by Maier and Thürlimann (1985). In their study, the specimens were considered as cantilevers with uniformly distributed vertical reinforcement and horizontal reinforcement ratios of 0 and 1.1%. The specimens S4 and S9, on which constant axial load and monotonically increasing lateral load were applied, resulted of particular interest. The aspect ratio of the specimens was equal to 1.02. Details of these specimens are listed in Table 2.1. Specimens S4 and S9 had the same geometry characteristics but the latter had not horizontal reinforcement. It was observed that the peak load was slightly influenced by the presence of horizontal reinforcement whereas the ultimate drift decreased in absence of horizontal reinforcement. Different failure modes were reported for the two specimens: specimen S4 failed in diagonal compression while specimen S9 failed in diagonal tension.

Lefas et al. (1990) studied the effect of parameters such as the height to width ratio, the axial load, the concrete strength, and the amount of web horizontal reinforcement (0.37%, 1.1%) on wall behavior with concentrated boundary reinforcement. The test set-up consisted of simple cantilevers with tip load. Although the amount of horizontal reinforcement was reduced almost by a factor of three, this reduction had not effect on failure mode, peak load, and achieved drift. The specimens failed in diagonal compression failure and it was concluded that the concrete compression zone contributes significantly to the overall shear strength of the wall associated with the development of triaxial compressive stress conditions.

The response of four squat walls with rectangular or flanged cross-section under static-cyclic load was examined by Paulay et al. (1982). The behavior of the specimen Wall1, which had an horizontal reinforcement ratio (1.6%) double than the vertical one (0.8%), is of particular interest. The specimen was designed without strong boundary reinforcement and axial force was not applied on it. The failure of this wall was dominated by sliding shear. Significant strength loss, due to degradation of aggregate interlock, was observed at displacement ductilities greater than $\mu_\Delta = 4$. Crossed diagonal reinforcement in two directions, effective in both tension and compression, was found to improve seismic response of squat shear walls.

Salonikios et al. (1999) carried out an experimental investigation of the validity of the design provisions both in Europe (EN-1998-1, 2005) and in the U.S. (ACI-318-05, 2005) for walls with $h_w/l_w$ ratios of 1.0 and 1.5. The wall specimens was reinforced against shear, either conventionally (orthogonal grids of web reinforcement), or with cross-inclined bars; the effects of web and edge reinforcement ratio, of axial load level, and of the quality of construction joints were also investigated. The specimens were tested as cantilevers. All specimens tested failed in a predominantly flexural mode, characterized by concrete crushing and reinforcement
<table>
<thead>
<tr>
<th>Ref.</th>
<th>Spec.</th>
<th>Load</th>
<th>$h_w$</th>
<th>$l_w$</th>
<th>$t$</th>
<th>$h_w/l_w$</th>
<th>$\rho_h$</th>
<th>$\rho_v$</th>
<th>$\rho_e$</th>
<th>$f'_c$</th>
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<td>1.13</td>
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<td>0.68</td>
<td>0.68</td>
<td>3.14</td>
<td>28.20</td>
</tr>
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</table>

[1]-Maier and Thürlimann (1985); [2]-Lefas et al. (1990); [3]-Rothe (1992);
[4]-Paulay et al. (1982); [5]-Salonikios et al. (1999); [6]-Hidalgo et al. (2002); [7]-Fouré (1993).

Applied loadings: mon-static monotonic, dyn-dynamic, st-static cyclic.

*Units: cm².

Table 2.1: Tests of squat shear wall from Greifenhagen (2006).
buckling at the confined edges showing displacement ductilities up to 5.3. Diagonal cracking of the web and sliding at the fixed base were observed for the specimens LSW1, LSW2, and LSW3 (Table 2.1) which had no diagonal reinforcement. Moreover, it was concluded that the absence of diagonal reinforcement anchored in the wall foundation leads to pinched hysteretic loops and diminution of energy dissipation.

Fouré (1993) reported static cyclic tests of walls with $h_w/l_w$ ratios of 0.5; the walls had a full rotational restraint at the top and were subjected to axial force ratios of almost 0.03. A diagonal tension failure was observed in the specimens. Strength and deformation capacity was marginally affected by horizontal reinforcement while vertical reinforcement was seen to be necessary for both flexure and shear. Also, it was found that the efficiency of horizontal reinforcement reduces as the aspect ratio of walls decreases.

The behavior of walls that exhibit the shear mode of failure was studied by Hidalgo et al. (2002), through the results of an experimental program that included the test of 26 full scale specimens, subjected to static cyclic loads. Test parameters were the aspect ratio of the walls, the amount of vertical (longitudinal) and horizontal (transverse) distributed reinforcement, and the compressive strength of concrete. The shear span ratio $M/Vl_w$ ranged from 0.35 to 1.0. Rotational and vertical restraining was applied to the top of the sections. Web horizontal reinforcement ratios ranged from 0 to 0.38% while vertical reinforcement ratios ranged from 0 to 0.26%. The properties of a selection of these specimens are reported in Table 2.1. The strength deterioration of wall specimens increased with decreasing values of the aspect ratio and of both horizontal and vertical reinforcement. The strength of the walls was restricted by diagonal tension failure so that the observed strength was between 36 and 73% of the base shear at nominal flexural strength. Vertical distributed reinforcement used in the test specimens had little or no influence on the maximum shear strength developed by the walls. This observation suggested that the test setup had a significant influence on the behavior of the vertically distributed bars. Figure 2.4 shows an example of the relation between the hysteretic shear behavior of shear wall and the resulting cracking pattern.

The behavior of cantilever walls under static-monotonic, static-cyclic, and dynamic loads was investigated by Rothe (1992). The specimens T01, T04, T10, and T11 showed different failure modes. The specimens had a shear span ratio of 1.5. The reinforcement details of all these specimens were the same ($\rho_h = 0.47$ and $\rho_v = 0.71$) except for the specimen T04, which did not have horizontal reinforcement. The rupture of some vertical rebars lead to collapse of specimen T01, while the failure of the specimen T04 was caused by diagonal tension. Both specimens were tested on a shaking table. Specimens T10 and T11 collapsed for sliding shear and diagonal compression, respectively. For both specimen static-cyclic test were
performed. A sliding shear mode of failure was not observed in dynamic tests; this was explained with the fact that dynamic sliding shear strength was considered to be significantly greater than the static value.

A shear wall database that include all the afore mentioned works regarding squat shear wall tests was created by Greifenhagen and Lestuzzi (2005). Static cyclic tests of four lightly reinforced concrete wall specimens that model shear walls of existing buildings in Switzerland were performed; the results were then compared with tests present in the database. Horizontal reinforcement, axial force ratio, and concrete compressive strength were varied in the tests. The shear span ratio \( M/Vl_w \) was for all four specimens equal to 0.69. The specimen characteristics are reported in Table 2.2.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( l_w ) [m]</th>
<th>( t ) [m]</th>
<th>( M/Vl_w )</th>
<th>( \rho_v ) [%]</th>
<th>( \rho_h ) [%]</th>
<th>( f'_c ) [MPa]</th>
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</thead>
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<td>0.69</td>
<td>0.3</td>
<td>0.3</td>
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<td>0.0</td>
<td>51.0</td>
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<td>0.69</td>
<td>0.3</td>
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Table 2.2: Specimen’s characteristics for the tests performed by Greifenhagen and Lestuzzi (2005).

Different crack patterns were observed varying the concrete compressive strength and the axial force ratio. The study showed that shear walls that were only designed...
for gravity loads can have drift capacity greater than 1% and that deformation capacity was not influenced by the ratio of horizontal reinforcement in lightly reinforced shear walls. The comparison with other tests from the literature pointed out that, for squat walls, the axial force ratio, vertical reinforcement arrangement, and the degree of restraining at the top of the wall influences the drift capacity. Cantilever shear walls without rotation restraints were less susceptible to brittle shear failure than walls with fixed top ends. Details of the specimens so far described are listed in Table 2.1. Figure 2.5 plots the nominal shear stress ratio against drift (Greifenhagen and Lestuzzi, 2005; Greifenhagen, 2006). The nominal shear stress ratio was derived from the maximum base shear while the drift was calculated from the ultimate top lateral displacements. Figure 2.5 shows as the shear stresses of all the specimens reported in the database are included between $0.3\sqrt{f_c'}$ and $0.9\sqrt{f_c'}$; the maximum drift was equal to 2.3%.

Figure 2.5: Experimental data for squat shear walls of rectangular cross-section (Greifenhagen and Lestuzzi, 2005).

Wood (1990) analyzed the results of 143 tests of low-rise shear walls subjected to lateral loads. The walls considered in his study were tested in Japan, Canada, New Zealand and United States. The cross-sections were symmetric, flanged, barbell and rectangular. Three-quarters of the 143 shear walls studied by Wood (1990) had a shear span ratio between 0.5 and 1.0. The test results were compared to the ACI-318-83 (1983) for the design of short walls for in-plane shear given by:

$$\nu_n = \alpha_c \sqrt{f_c'} + \rho h f_y$$

(2.1)

where $\alpha_c$ decreases linearly from 1/4 (MPa) for walls with an aspect ratio $h_w/lw = 1.5$ to 1/6 for $h_w/lw = 2.0$ and staying at 1/6 for $h_w/lw \geq 2.0$. The symbol
\( \rho_h \) represents the horizontal reinforcement ratio. A reasonable lower bound of the observed shear strengths of lightly reinforced walls with orthogonal shear reinforcement was found to be:

\[
\nu_n = \sqrt{f'_c}/2 \quad \text{ (in MPa)} 
\] (2.2)

The maximum failure shear stress tended to increase with in the amount of vertical reinforcement (both vertical reinforcement in the boundary elements and in the web). The increase in shear strength attributable to vertical reinforcement was approximated using a shear friction model. Wood proposed an upper strength limit of:

\[
\nu_n = 5\sqrt{f'_c}/3 \quad \text{ (in MPa)} 
\] (2.3)

### 2.4 Tests on slender shear walls

Cardenas et al. (1973) studied the behavior of slender shear walls. The specimen with the lowest amount of longitudinal reinforcement showed a sudden failure due to the rupture of the vertical reinforcement; just before failure a single crack appeared. At failure, the first layers of steel across the crack had no adequate tensile strength to replace the concrete tensile stresses lost when the concrete cracked.

Wood (1989) reviewed the results reported in the literature obtained from 37 tests of shear walls having the height to width ratio greater than 2. The specimens had rectangular, barbell and flanged cross-section. Only two specimen were tested dynamically, the other specimens were subjected to monotonic or cyclic load. The ratio of axial stress to compressive strength of concrete ranged from 0.3 to 14 percent. On the base of the observed crack patterns, two main categories of failure were identified: flexural and shear failures. Approximately 40% of the walls failed in flexure. Ten out of 37 specimens failed by the fracture of the flexural reinforcement. Reinforcing bars did not fracture in any of the walls that failed in shear. Wood (1989) defined two behavioral indices to organize the experimental data with respect to the observed mode of failure. Another index was also introduced as a simple parameter for determining the vulnerability to fracture of the reinforcement. The first index was the shear-stress index (SSI):

\[
SSI = \frac{\nu_{\text{max}}}{\nu_n} 
\] (2.4)

where \( \nu_{\text{max}} \) was the shear stress at failure and \( \nu_n \) was the nominal shear stress capacity from Eq. 2.1. The shear-stress index was used to distinguish between flexure and shear modes of failure.

Among the walls that failed in flexure, the steel strain was used to identify those susceptible to fracture of main reinforcement. The calculated strain \( \varepsilon_t \) in the extreme flexural bar at nominal flexural capacity was used as the second index. Sev-
eral assumptions were made to calculate the nominal flexural capacity: one of them assumed that the maximum strain at the extreme concrete compression fiber was equal to 0.003. Twenty of the 24 walls with a SSI greater than 0.75 failed in shear. Of the 13 walls that had a SSI less than 0.75, 12 failed in flexure. In ten of the 11 specimen that failed in flexure was observed a fracture of the main reinforcement; in addition these specimens had a calculated steel strains greater than 2.5% at the nominal flexural capacity. In the test specimens that were not susceptible to shear failures (SSI ≤ 0.75) and for which the calculated steel strains in the extreme tensile fiber was greater than 4%, fractured reinforcement was observed.

The third index, named flexural-stress index (FSI) was defined as:

$$FSI = \frac{\rho_t f_y + \frac{P}{A_g}}{f'_c}$$

(2.5)

where \(\rho_t\) represents the total reinforcement ratio in the wall, and \(A_g\) is the gross area of the cross-section. Failures caused by rupture of the reinforcement were observed in walls with FSI less than 15%. Wood (1989) also observed that slender shear walls with a total reinforcement ratio less than 1% were potentially susceptible to fracture of the tensile reinforcement.

Four tests were carried out by Zhang and Wang (2000) on rectangular reinforced concrete shear walls subjected to high axial loading. Two levels of axial load ratios \(N/(f_c A_g)\) were considered, equal to 0.25 and 0.35 respectively. The walls were subjected to combination of constant axial load and reversed cyclic lateral loading. The four walls had all the same geometric configuration, which were 700 mm wide, 1500 mm height and 100 mm thick with an aspect ratio of 2.14. For the four specimens, the design philosophy was based on the concept that the lateral load capacity is controlled by flexure avoiding an undesirable premature shear failure. The wall specimen with an axial load ratio of 0.25 and shear compression ratio of 0.11 failed in flexure at a load slightly above the calculated flexural strength, exhibiting a relatively high ductility. The shear compression ratio was defined as:

$$\lambda_v = \frac{V_{max}}{f_c A_g}$$

(2.6)

where \(V_{max}\) is the maximum shear acting on the wall and \(f_c\) is the concrete compressive strength, based on 28-day test. The specimens subjected to a high axial load ratio of 0.35 exhibited a premature failure at a low ductility level, due to an undesirable out-of-plane buckling mode in the post-yielding stage. As a general conclusion, it was found that the axial load ratio had a significant effect on the crack pattern, flexural strength, failure mode, and ductility of the walls.

Pilakoutas and Elnashai (1995b) tested six cantilever walls under severe cyclic loading up to failure. The walls had an aspect ratio of 2 and a scale of 1/2.5. The
shear reinforcement in excess with respect to the amount required to resist the maximum applied load did not affect the strength and deformational characteristics of the specimens. Pilakoutsas and Elnashai (1995a) demonstrated, through the decomposition of the total lateral deformation into flexural and shear components, that the bulk of the energy dissipation is due to flexure. Even if shear deformations were significant on the total amount, they cannot be considered to contribute significantly to the overall energy dissipation. It was found that suitable shear reinforcement is necessary to avoid early post-yield shear deterioration which can have adverse effects on the strength, ductility, and energy dissipation. Finally, it was found that an arch and tie mechanism was activated by the walls for the shear resistance. An example of crack pattern at different stages is shown for the wall SW9 in Figure 2.6, where the term MDL means maximum displacement level. The load-versus-top displacement for the same wall is depicted in Figure 2.7. It is important to point out that the cyclic response of the specimen is characterized by a medium degree of pinching; moreover the wall survived both cycles at MDL-24 and lost strength during the reverse cycle at the second forward cycle at MDL-26 reaching failure.

**Figure 2.6:** Crack pattern of wall SW9 at MDL-2, MDL-6 and failure (Pilakoutsas and Elnashai, 1995b).

Tansnimi (2000) studied the behavior and type of failure of four wall specimens subjected to various slow cyclic horizontal loading regimes. The shear walls were cast at 1/8 scale having constant thickness, rectangular cross-section and an aspect ratio equal to 3. All the walls were identical in size and reinforcement ratios as well as type of material. The only variable was the type of lateral load applied; the walls were not subjected to axial force. A plastic hinge at the base of the wall was found in all the specimens. The cyclic loading sequence had not influence on the strength and deformational responses of the specimens. An horizontal crack at the base (just above the foundation) was visible before ultimate state in all four specimens. This crack, probably due to the sliding of the vertical reinforcement, caused
2.4 TESTS ON SLENDER SHEAR WALLS

Figure 2.7: Load versus top displacement for the wall SW9 (Pilakoutas and Elnashai, 1995b).

A remarkable reduction in the strength, stiffness and energy dissipation of the specimens. Although two layers of reinforcement were provided to carry substantial shear forces, as recommended in ACI specifications (ACI-318-05, 2005), it was observed that the shear resistance of the wall, due to aggregate interlock degradation, was inadequate or absent at all.

A specimen in full scale, representative of a wall in a four story building with one underground floor, was tested by Riva et al. (2003) under cyclic loading up to collapse. The specimen was 15.5 m tall with a rectangular cross-section of 2800 × 300 mm. The wall was simply supported at the foundation and at the ground floor levels. The specimen was designed for moderate seismic action ($PGA = 0.20$) adopting EN-1998-1 (2005). Cycles following yielding were characterized by an insignificant strength and/or stiffness degradation. A single large crack near the base section, almost parallel to the ground floor diaphragm, was visible at collapse. This crack caused the tensile rupture (necking) of the longitudinal reinforcement; shear governed the failure mechanism. The shear reinforcement provided at the critical section was considered to be insufficient to avoid an early shear failure.

An example of stable hysteretic response of a cantilever wall subjected to incremental reversing displacement and variable axial compression is shown in Figure 2.8 (Paulay and Goodirs, 1985). The wall was designed following capacity design principles. A displacement ductility of about 4 was attained in a very stable manner. Failure occurred after two cycles to a displacement ductility of 6 with a deflection equal to 3.0% of the height of the wall. Failure was caused by inelastic instability.
2.5 Macro scale models

The analyses of reinforced concrete shear walls may be performed at three levels of refinements: the micro scale of the finite element method, the meso scale of a fiber model, and the macro scale of a global beam element. For the simplest approach, a single story of a wall can be modeled with a beam element. Many lumped plasticity models with the non-linearity concentrated at discrete plastic hinge located at the member ends were proposed in the past (Clough et al., 1965; Giberson, 1969; Takeda et al., 1970). The non-linear spread plasticity models provided a more accurate description of the inelastic behavior of reinforced concrete members (Otani, 1974; Soleimani et al., 1979; Meyer et al., 1983).

The first lumped and spread plasticity models, conceived to study the behavior of RC beams and columns subjected to large cyclic deformations, neglected the M-N interaction and the shear effects. The effect of axial load on the yield moment of RC columns was recognized by Kanaan and Powell (1973). However, the effect of axial force on the flexural stiffness of a member was first accounted for in the model proposed by Takayanagi and Schnobrich (1979) in their study of the seismic response of coupled wall systems. The beam elements were divided into a finite number of non-linear rotational springs connected in series. The walls and coupling beams were represented by one-dimensional beam elements. The interaction of bending moment, shear and axial forces was taken into account in the wall elements, while the axial stiffness of the coupling beams was assumed to be infinite, since the horizontal displacements of both walls were approximately equal. Otani’s model (Otani, 1974) was selected for modeling the coupling beams with the inflection point assumed fixed at midspan. The beams were connected to the wall elements
through a rigid link which accounts for the finite dimensions of the wall. A spring was inserted between the beam element and the rigid link to model the fixed-end rotations due to slippage of the reinforcing bars anchored in the wall. The effect of shear in the coupling beams was also taken into account. A modified Takeda’s model was adopted for the hysteretic behavior of the beam elements. The model accounted for the pinching effect during reloading and the strength decay due to loss of shear resistance after crack formation and yielding of the reinforcement in the coupling beams.

Several attempts to develop a model of the hysteretic behavior in shear were made to date. A qualitative model of the hysteretic shear force-deformation relation was proposed by Penzien and Penzien (1973). Another model was introduced by Ozcebe and Saatcioglu (1989). This model describes the experimentally observed stiffness degradation and the associated pinching of hysteretic loops. Empirically derived expressions were proposed which account for the effect of axial load on the hysteretic behavior.

Al-Sulaimani and Roessett (1985) defined a hysteretic model with softening to the aim to examine the effect of stiffness and strength degradation on the inelastic design spectra. They concluded that the stiffness would decrease during reloading and unloading (Figure 2.9). The reduction factor for stiffness during unloading was a function of the ratio of maximum displacement and yield displacement. The reduction factor for stiffness during the reloading was determined by connecting the load-reversal to the yield point or to the maximum load-deformation experienced if the system has already yielded in the new loading direction.

Roufaiel and Meyer (1987) proposed an extension of the spread plasticity model developed earlier by Meyer et al. (1983). The new model included the effect of shear and axial forces on the flexural hysteretic behavior based on a set of empirical rules. The hysteretic moment-curvature relation was described by Takeda’s model. The variation of axial loads due to overturning moments was not accounted for. The analytical results were compared with available experimental data and showed very good agreement. A set of new damage parameters was proposed which correlate well with the residual strength and stiffness of specimens tested in the laboratory.

A similar model was developed by Kunnath and Reinhorn (1990), along with a trilinear monotonic moment-curvature relation and a three-parameter hysteretic model. The values of these parameters determine the properties of stiffness degradation, strength deterioration and pinching. The results of the response analysis were expressed as damage indices using a calibrated damage model based on energy and ductility. However, the calibration of the parameters is definitely not easy and in this work, as in the previous ones, no mention is made of the problem of the element state determination, arising from the flexibility formulation of the model.

One of the limitations of previous studies dealing with the spread plasticity ele-
ment lies in the fact that the hysteretic model for the girder inelastic zones is assumed to be the same as that of the rotational springs modeling fixed-end rotations. No mention is made of the fact that the hysteretic behavior of these two critical regions is quite different. Mulas and Filippou (1990) extended Soleimani’s model (Soleimani et al., 1979) by inserting, at the beam ends, non-linear rotational springs. These describe the fixed-end rotations due to slippage of the reinforcement in the joint and follow a hysteretic rule which is independent of the rule describing the hysteretic behavior of the girder inelastic zone. In this model the problems posed by the flexibility formulation and by the introduction of two springs were solved completely, making use of an event-to-event strategy and of the static condensation procedures at the element level, so that the element is internally in equilibrium, at the end of each time step, whatever the position of contraflexure point may be.

A further flexibility model was proposed by Filippou and Issa (1988). It differs from Mulas and Filippou model in that the element was decomposed into different, parallel subelements, linked in series at the two ends, each describing a different deformation mechanism (Figure 2.10).

Three subelements were developed by authors: (a) an elastic subelement which models the flexural behavior of the frame member before yielding of the reinforcement; (b) a spread plastic subelement which describes the inelastic flexural behavior of the reinforced concrete member and accounts for the gradual spread of inelastic deformations at the member ends. The moment-curvature relation of this subelement is shown in Figure 2.11; and (c) a joint subelement modeling the fixed-end
rotation that arises at the beam-column interface due to bond deterioration and slippage of reinforcing bars along the joint anchorage. A Newton-Raphson iteration was used at the element level to achieve equilibrium. The model reproduces satisfactorily some experimental results.

Filippou et al. (1992) introduced several new subelements. The first is a shear subelement which describes the deformation due to shear distortion and, in particular, due to shear sliding in the inelastic regions of reinforced concrete members and, so, complements the list of girder subelements of the earlier study (Filippou and Issa, 1988). The model consists of a concentrated translation of zero dimension located at each member end. The two springs are connected by an infinitely rigid bar to form the subelement (Figure 2.12). In the case that a substantial axial force is present, the shear subelement is capable of describing the interaction of shear and axial forces with the opening and closing of shear cracks. The properties of the monotonic shear force-distortion relation of the element were obtained with the modified compression field theory (Vecchio and Collins, 1986). The hysteretic model of shear force-shear deformation relation, based on a set of rules which are shown in Figure 2.13, was derived from experimental results of beams subjected to flexure and shear. The other subelements refer to the hysteretic behavior of RC columns and are extensions of the corresponding girder subelements to account for the effect of axial load on the flexural and shear behavior of the member.
A simplified model of damage for RC member under hysteretic loading was proposed by Flórez-López (1995). A frame member was considered as the assemblage of an elastic beam-column and two inelastic hinges, as in the conventional lumped plasticity models. Three sets of internal variables, which measure plastic rotations and the state of damage of the member, were introduced. An expression for the flexibility matrices and the complementary strain energy of a member were proposed as a function of these internal variables. The model represents the following effects: (a) unilateral behavior and unsymmetrical cross-section with different yield capacities at positive and negative bending, (b) a simplified form of interaction of flexure and axial force in two forms: the so called P-∆ effect and the influence of the axial
force on the strength of the cross-section, (c) stiffness and strength degradation due to cracking of the concrete, (d) plastic deformations due to the yield of the reinforcement. However, the model does not take into account the pinching effect and low cycle fatigue effects.

A beam model based on a moment-curvature uniaxial cyclic law was proposed by Miramontes et al. (1996). The model is characterized by a trilinear envelope curve and a set of cyclic rules. The strength and stiffness degradation in the hardening and the softening branches as well as the hysteretic damping are directly dependent on the level and history of loading. Their continuous evolution was described with the aid of a new parameter based on energy criteria and phenomenological considerations. The principal phenomena taken into account by the hysteretic rules are: (a) stiffness degradation and strength reduction due to cycling loading; (b) pinching of loops due to shear stress; (c) softening behavior at failure and (d) the effect of axial force due to gravity loads. The evolution of cyclic behavior was controlled by a cyclic parameter defined in terms of a damage index. The model extends the earlier degradation rules (Takeda et al., 1970; Roufaiel and Meyer, 1987) to the unloading stiffness before yielding. Therefore, permanent strains reflect a more realistic behavior of RC cracked members and, at the same time, an uniform evolution of stiffness up to failure was assured.

Cofer (1999) modeled bridge columns with several types of analysis using a degrading plastic hinge model, a fiber beam model, and a detailed three-dimensional finite element model (Cofer et al., 2002). Reasonable results were obtained from

![Hysteretic behavior of shear subelement](image)
all models to varying degrees, but the three-dimensional finite element model was unable to provide meaningful solutions in the inelastic range due to numerical difficulties. Therefore, either the use of the degrading hinge model or the fiber beam model was recommended by the author for practical structural analysis. For the simplest approach, reinforced concrete beams and columns were modeled as traditional beam elements, with material non-linearity lumped at discrete plastic hinges. The moment-curvature envelope beyond a critical value $\phi_c$ was described by a softening branch representing severe damage in the element. At the softening stage, a damage coefficient $D$ was introduced; a value of 0 indicates no damage while a value of 1 means total damage. During the evolution of softening, the damage coefficient is equal to:

$$D = \frac{\theta_p - \theta_{cp}}{\theta_{max} - \theta_{cp}}, \quad 0 \leq D \leq D_{max} \leq 1 \quad (2.7)$$

where $\theta_p$ is total plastic rotation angle, $\theta_{cp}$ is the critical plastic rotation angle indicating the point at which softening begins, and $\theta_{max}$ is the maximum theoretical plastic rotation angle. The angles $\theta_{cp}$ and $\theta_{max}$ are functions of the plastic hinge length. Using a plastic flow theory, Cofer (1999) proposed a yield function that included softening and defined as

$$\left(\frac{M_{yu}}{M_{yp}}\right)^2 + \left(\frac{M_{zu}}{M_{zp}}\right)^2 = (1 - D)^2 \quad (2.8)$$

where $M_{yu}$ and $M_{zu}$ are the bending moment values about the y- and z-axis, respectively, on the interaction surface for a fixed value of axial force, $P_u$. $M_{yp}$ and $M_{zp}$ are the ultimate bending moment values about the y- and z-axis, respectively, for the same fixed value of $P_u$ when $M_y$ and $M_z$ are applied separately. The values $M_{yu}$ and $M_{zu}$ are a cubic function of axial force $P_u$; moreover being the quantity $(1 - D)^2$ less than or equal to one, the yield surface shrinks. It is worth to point out that the approach is also based on the assumption that the shear and axial capacities remain intact.

Kilar and Fajfar (1997) presented a simple method for the non-linear static analysis of complex building structures subjected to monotonically increasing horizontal loading (pushover analysis). The method was based on the extension of a pseudo-three-dimensional mathematical model of a building structure into the non-linear range. The structure consists of planar macro elements. For each planar macroelement, a simple bilinear or multilinear base shear-top relationship was assumed. By a step-by-step analysis an approximate relationship between the global base shear and top displacement was computed. The method is capable of estimating several important characteristics of non-linear structural behavior, as the real strength and the global plastic mechanism. It also provides an estimate of the required ductilities of the different macro elements in relation to target maximum displacement.
Galal and Ghobarah (2003) developed a global element, based on plasticity theory, to model the biaxial flexure and shear behavior of RC columns subjected to varying axial load. The model included stiffness degradation upon load reversals. The element consists of an elastic element with lumped plastic hinge at each end. The plastic hinge is composed of three flexure plastic subhinges and one shear subhinges in series. The three flexural plastic subhinges represent a stage of the non-linear behavior of RC members, namely the concrete cracking, steel yielding and ultimate condition, while the shear subhinge represent a shear failure. A quadratic function describes the yield surface for each subhinges and allows to represent the biaxial moment and shear-axial force interaction. The behavior of the element was verified using experimental results and was shown to be fairly accurate in predicting the response of axially loaded columns with an applied variable axial load.

A new hysteretic model, named the $\gamma$-model, was developed for RC structural walls by Lestuzzi and Badoux (2003). The model is based on the measured hysteretic loops of six RC structural walls recorded during dynamic tests. As illustrated in Figure 2.14 with sample hysteretic loops for two test walls, the empirical observation that the reloading curves tend to cross at the same point constitutes the basis of the $\gamma$-model. As shown in Figure 2.14, the force-displacement relationships of the elastoplastic model were modified in such a way that the reloading curves of large yield excursions cross the elastic portion of the envelope at a height of $1-\gamma$ of the yield force. Otherwise the reloading curves aim to the current peak displacement. The name of the model reflects the shape of the produced hysteretic loops, which looks like the symbol $\gamma$. Compared to more complex models such as Takeda-model and Q-model (Takeda et al., 1970; Saiidi and Sozen, 1979), it requires significantly less computational effort. This can be important in design situations where a large number of non-linear time history analyses must be conducted (i.e. design for many randomly generated design spectra compatible earthquakes). Unlike the Takeda-model, no numerical instability was observed, i.e. the $\gamma$-model also seems to be more robust numerically.

Kyakula and Wilkinson (2004) proposed a spread plasticity model that correctly identifies the initiation of yielding anywhere in the beam, takes into account the gradual spread of plasticity, the shift of the points of contraflexure, the variable location and actual length of the yield zones. The model assumed that columns and beam-column joints remain elastic. Beams were made up of elastic and spread plasticity sub elements connected in series. When a beam yields, its stiffness reduces and flexibility increases. The spread plasticity sub element has a null flexibility matrix before yielding; as the beam yields, the magnitude of its coefficients increase. At each time step, the model updates the flexibility matrices of the spread plasticity sub elements. Unlike existing spread and concentrated plasticity models, the moments within the span are monitored and the effect of their yielding or “unyield-
ing” taken into account. The authors concludes that spread plasticity models that only consider plasticity at the beam-column connections are only accurate for lower stories and structures where the applied-design gravity load $< 0.8$.

Sivaselvan and Reinhorn (2000) proposed a 1D smooth hysteretic model based on internal variables to study structures subjected to large and repeated cyclic deformation. The model included pinching effect, stiffness and strength deterioration. Oppositely to the polygonal hysteretic model based on piecewise linear behavior (Clough et al., 1965; Takeda et al., 1970), the smooth hysteretic models are characterized by a continuous change of stiffness. Several attempt were made to generalize the 1D models to three dimensions to take into account the interaction between various stress resultant (Casciati, 1989).

A lumped plasticity model accounting for the M-N interaction was utilized by Kaewkulchai and Williamson (2004) for progressive collapse analysis of planar frame. Stiffness and strength degradation were included into the model through the use of a damage dependent constitutive relationship. A damage index was used to determine the onset of member failure. The damage $D$, was given by the following formula:

$$D = \alpha U(\delta) + \beta W(\delta) \quad (2.9)$$

where $U(\delta)$ is a function that depends upon the maximum deformation, and $W(\delta)$ is a function that depends upon the accumulated plastic energy. Unlike the previous damage formulation, $\alpha$ and $\beta$ are not constant but functions of the properties of the
structural system. The most significant feature of the model was the capability to continue an analysis after the failure of a member.

Hidalgo et al. (2002) proposed a macro element with uncoupled hysteresis models for shear and flexure. Two different failure modes, a flexural and a shear one, were also included in the model. Pinching effects, stiffness degradation and strength reduction due to repeated cycles at the same deformation level were implemented in the hysteresis shear model. The model for the shear failure mode assumed independence of the shear strength of walls on both the bending moment and the axial force present in the wall. The definition of the envelope curves was based on the experimental results obtained from the cyclic test of 26 full scale, shear wall specimens.

Pincheira et al. (1999) extended the lumped plasticity model originally developed by Giberson (1969) to include shear behavior. To incorporate non-linear behavior in shear, the existing model was modified by adding a zero-length shear spring as shown schematically in Figure 2.15. Separate hysteretic laws were used in flexure and shear; the non-linear response of the element was obtained by the superposition of these uncoupled laws. The hysteretic laws were created specifically to model older RC construction under seismic ground motions and included strength and stiffness degradation in both flexure and shear. A backbone curve envelope was considered both in moment-rotation and shear force-shear displacement relations. The backbone of the non-linear shear spring allowed for strength degradation and was based on the modified compression field theory (Vecchio and Collins, 1986).

A special class of macroscopic phenomenological models was developed to study the non-linear behavior of RC shear walls and find origin in the Three-Vertical-Line-Element Model (TVLEM), proposed by Kabeyasawa et al. (1983). In the TVLEM
model, axial springs at each ends of the wall and a rotational spring at the center represented the flexural behavior, while a horizontal spring modeled the shear behavior. Since there several models were proposed (Vulcano and Bertero, 1987; Vulcano et al., 1988; Fischinger et al., 1990, 1992); among these the Multi-Component-in-Parallel-Model (MCPM) derived by Vulcano et al. (1988) added more axial springs and removed the rotational springs. Vulcano indicated that having at least four axial springs can improve the flexure response prediction. An update version of the MCPM, named the Multi-Vertical-Line-Element-Model (MVLEM) was proposed by Orakcal et al. (2004) in the intent to study the behavior of RC walls which response was governed by flexure. The MVLEM will be herein briefly discussed.

A 2D version of the MVLEM is illustrated in Figure 2.16. Each interstory of the wall can be modeled with a single MVLEM element; the whole wall is modeled as a stack of \( m \) MVLEM elements, which are placed on one another. Where intensive inelastic response is expected, more than one MVLEM for story can be used. Rigid beam at the top and bottom levels are connected by a series of trusses, which simulate the flexural response. At each truss is assigned a tributary area and force-displacement relationship are derived by uniaxial, cyclic constitutive models for concrete and steel. The point placed on the central axis of the element at height \( c h \) defines the relative rotation between top and bottom faces of the wall element. The value \( c \) is chosen on the base of the expected curvature distribution along the element height \( h \). The shear response is simulated by an horizontal spring placed at the height \( c h \). A non-linear shear force-shear deformation law can be assigned to the horizontal spring; nevertheless flexural and shear modes of deformation of the wall member are uncoupled. The MVLEM has six global degrees of freedom, three each located at the center of the rigid top and bottom beams (Figure 2.16). Plane sections remain plane hypothesis used together with the six nodal degrees of freedom allows to obtain the strain level in each uniaxial element. The model has the advantage to simulate the shifting of the neutral axis along the wall cross-section and the effect of fluctuating axial force, which are commonly ignored in macroscopic beam-column element.

In order to capture the experimentally observed shear-flexure interaction in RC walls under seismic input motions, Massone et al. (2006) proposed an analytical model that incorporates RC panel behavior described by a rotating-angle approach (Pang and Hsu, 1995), into the Multiple-Vertical-Line-Element-Model afore mentioned. The analytical model was based on the methodology developed by Petrangeli et al. (1999) to the macroscopic MVLEM element. Each shear spring in the MVLEM was treated as a RC panel element, with membrane actions, with uniform normal and shear stresses applied in the in-plane direction. The interaction between flexure and shear was incorporated at the uniaxial element level. This methodology involves the implementation of the finite element method together with a constitu-
A simple macro model similar to the TVLEM was proposed by Kim et al. (2005). The model consists of three vertical axial springs connected to rigid beams at top and bottom and a single horizontal spring. Axial springs represent flexural behavior which is based on the moment-curvature envelope from a wall cross-section analysis at the base of a story. The axial springs at the boundaries simply replace the moment-curvature relationship with force-displacement relationship. The horizontal spring represents shear behavior. Different hysteretic rules, including pinching effect, stiffness and strength degradation, were used for the flexural and shear behavior. The parameters for the shear hysteretic rule are from flexure-based calculations, except for the stiffness, which is based on shear properties. Moreover, the moment-shear interaction was obtained to enforce the shear spring to yield at the same time that the vertical springs reach flexural yield.

With the objective to study plastic hinge region in shear walls, a macro element consisting of four vertical steel, four vertical concrete and two diagonal shear spring connected by rigid truss was developed by Youssef and Ghobarah (2001). The steel springs present in the model intended to represent the behavior of a group of steel reinforcement bars and is not a pure material model. In the force-displacement steel spring model, bond slip softening is defined as the point at which force degradation starts. The concrete springs describe the relationship between the axial force on an identified concrete strut and the axial displacement of that strut. In the shear spring model, the shear force corresponding to a specified shear deformation is determined using the modified compression field theory for each cycle of the iteration process. Further details for the steel, concrete and shear spring models can be found in Ghobarah and Youssef (1999). The effects of the normal force and the bending moment

Figure 2.16: MVLEM element (Orakcal et al., 2004).
on the shear behavior of the shear spring were taken into account using sectional analysis.

2.6 Meso scale models - fiber elements

Today, a special class of FE, the so called “fiber beam element”, is considered particularly appropriate to study the flexural behavior of mono-dimensional structure as beams, columns or walls. The behavior of these elements is followed in a number of control cross-sections; each cross-section is subdivided into concrete and steel fibers where uniaxial stress-strain laws are used to describe the response of the material in the longitudinal direction (parallel to the beam axis). The constitutive relation of the section is not specified explicitly, but is derived by integration of the response of the fibers (Kaba and Mahin, 1984).

The first flexibility-based fiber element (Kaba and Mahin, 1984), even though yielded very promising results, presented some inconsistencies in the determination of the element flexibility matrix and in the state determination process that lead to numerical problems. Zeris and Mahin (1988, 1991) updated the original model; however the state determination procedure remained not clear.

A consistent force-based frame element formulation that strictly satisfies the equilibrium and enforces compatibility in an integral form along the beam element, was proposed by Spacone et al. (1996). The main shortcomings of flexibility-based elements is the difficult implementation in non-linear displacement-based analysis program, because the nodal forces cannot be computed from the section forces. Spacone et al. (1996) proposed an iterative procedure to solve this problem. The procedure produces the element stiffness and nodal forces for given element end deformations. The advantages of the force-based beam-column element over the traditional displacement-based element have been discussed by Neuenhofer and Filippou (1997). The force-based elements, characterized by exact force interpolation functions, permit the use of one element for the representation of the non-linear response of a frame member (beam or column) or a wall’s interstory. In contrast, displacement-based beam-column elements follow the standard finite element formulation, in which the element displacement field is derived from nodal displacements (Zienkiewicz et al., 2005). As result, the displacement field is approximate, thus several displacement-based elements are required along the length of a frame member or a wall’s interstory to represent the deformations in a plastic hinge region. Plane sections remain plane hypothesis was assumed and the effects of shear and bond slip were neglected.

Bažant and Bhat (1977) investigated the possibility of using and of extending fiber approach so as to include shear strain due to bending. Both curvature and transverse shear were assumed to contribute to deflections.
A displacement-based Timoshenko beam element based on uniaxial constitutive relations was proposed by Guedes et al. (1994). The shear strain was included in the model, but the axial force and the bending moment were uncoupled from the shear force.

A three node displacement-based beam-column fiber element including shear effects was proposed by Martinelli (2000) (Mulas et al., 2006a). The kinematic field assumes perfect bond between concrete and steel. This kind of element is potentially prone to “shear locking”. To avoid this phenomenon, the polynomials adopted to interpolate transverse displacement were one order greater than those used to interpolate the rotations. The “shear constraints” method was adopted, leading to a constant shear distortion along the element (Crisfield, 1986). The shear-bending interaction was reproduced linking, through proper kinematic assumptions, the modeled resisting shear force to the bending and axial behavior at the cross-section level. Axial force and bending moment are calculated as for the standard fiber elements by the resultants of the fiber contributions. The innovation of the element is represented by the procedure to determine shear forces, which depend on the description of the shear resisting mechanisms. Two shear mechanisms were included in the model through two mechanical formulations: the truss and arch mechanisms. The first mechanism was based on the Mörsch truss analogy which consider a RC beam composed of both the inclined concrete diagonal and the transverse reinforcement. The variation of the internal lever arm between internal compression and tension forces created the second mechanism. The principal stress present in the compression zone of the cross-section could be not aligned with the longitudinal beam axis. Part of the external shear force is balanced by the component lying on the cross-section of the resultant compression force. Non-linear material models were used for the concrete and steel components, with specific choices within each mechanism.

The 3D non-linear behavior of shear wall subjected to dynamic loading was studied by Kotronis and Mazars (2005) through a 3D Timoshenko fiber beam-column element. The element distinguishes from others element (Spacone et al., 1996; Petrangelo et al., 1999) to be displacement-based with a higher order of interpolation functions depending on material’s properties. Cubic and quadratic Lagragian polynomials for the transverse displacement and rotations were used respectively, to avoid shear locking phenomena. The displacement field $\mathbf{U}_s$ and the nodal displacement $\mathbf{U}$ are grouped in the following arrays:

$$\mathbf{U}_s = \mathbf{N} \mathbf{U}$$  \hspace{1cm} (2.10)

$$\mathbf{U}_s^T = \{ u_s(x) \, v_s(x) \, w_s(x) \, \theta_{sx}(x) \, \theta_{sy}(x) \, \theta_{sz}(x) \}$$  \hspace{1cm} (2.11)

$$\mathbf{U}^T = \{ u_1 \, v_1 \, w_1 \, \theta_x \, \theta_y \, \theta_z \}$$  \hspace{1cm} (2.12)
where 1 and 2 indicate the two nodes of the beam, \( x \) is the longitudinal axis of the beam, \( s \) is the subscript indicating “section variables”, \( u, v, w \) the displacements and \( \theta_x, \theta_y, \theta_z \) the rotations according to the \( x, y, z \) axis respectively. \( \mathbf{N} \) is matrix containing the interpolation function and has the form proposed by Friedman and Kosmatka (1993). The interpolation function depend on the stiffness ratio \( \phi \) or on the stiffness ratio \( \phi^* \):

\[
\phi = \frac{12}{L^2} \left( \int_S E_y^2 dS \right) \left( \int_S G dS \right)
\]

\[
\phi^* = \frac{12}{L^2} \left( \int_S E_z^2 dS \right) \left( \int_S G dS \right)
\]

being \( L \) and \( S \) the length and section of the beam, \( k_y, k_z \) the shear correction factor dependent upon the material definition and cross-section geometry, \( E \) and \( G \) the classical Young’s and shear moduli of the beam material. In the case of slender structures the stiffness ratios \( \phi \) and \( \phi^* \) are equal to zero and the stiffness matrix reduces to the one of the Euler-Bernoulli beam theory. Indicating with \( \mathbf{F} \) and \( \mathbf{D} \) the section stresses and strain respectively, the section stiffness matrix \( \mathbf{K}_s \) takes the following form (Guedes et al., 1994):

\[
\mathbf{F} = \mathbf{K}_s \mathbf{D}
\]

\[
\mathbf{F}^T = \{ N_T_y T_z M_x M_y M_z \}
\]

\[
\mathbf{D}^T = \left\{ u'_s(x) \ v'_s(x) - \theta_{sz}(x) \ w'_s(x) + \theta_{sy}(x) \ \theta'_{sx}(x) \ \theta'_{sy}(x) \ \theta'_{sz}(x) \right\}
\]

\[
\mathbf{K}_s = \begin{bmatrix}
K_{s11} & 0 & 0 & 0 & K_{s15} & K_{s16} \\
K_{s22} & 0 & K_{s24} & 0 & 0 \\
K_{s33} & K_{s34} & 0 & 0 \\
K_{s44} & 0 & 0 & \text{sym} \\
K_{s55} & K_{s56} & K_{s66}
\end{bmatrix}
\]

with the matrix coefficients equal to

\[
K_{s11} = \int_S E dS; \quad K_{s15} = \int_S E z dS; \quad K_{s16} = -\int_S E y dS; \quad K_{s22} = k_y \int_S G dS;
\]

\[
K_{s24} = -k_y \int_S G z dS; \quad K_{s33} = k_z \int_S G dS; \quad K_{s34} = k_z \int_S G y dS;
\]

\[
K_{s44} = \int_S G (k_y y^2 + k_z z^2) dS; \quad K_{s55} = \int_S E z^2 dS; \quad K_{s56} = -\int_S E y z dS;
\]
\[ K_{66} = \int_S E\gamma^2 dS; \]

As usually, \( N \), and \( T \) represent the axial and shear forces respectively, and \( M \) the bending moment. The constitutive model for concrete was based on damage mechanics and took into account some observed phenomena under cyclic loading such stiffness degradation due to cracking, stiffness recovery which occur at crack closure and inelastic strain induced by damage. The uniaxial version of the continuous damage constitutive law was used, with shear and torsion considered linear.

A 2D flexibility-based fiber beam element was proposed by Ranzo and Petrangeli (1998); the element strain state determination follows an equilibrium based approach presented in Petrangeli and Ciampi (1997). At the section level a hybrid formulation was used: the classical fiber method with plane sections remaining plane hypothesis describes the axial-flexural interaction while the shear response is taken into account with a non-linear truss model and described with a hysteretic stress-strain relationship. Bending and shear were coupled at section level: the shear constitutive relation contained a damage parameter dependent on flexural ductility. The flexural behavior was represented in a theoretical way, while shear response was described by empirical relationship calibrated on experimental results. As in the traditional fiber models, axial-flexural behavior was function of the section axial deformation \( \varepsilon \) and curvature \( \phi \), while the shear behavior was a function of section distortion \( \gamma \) and axial deformation \( \varepsilon \):

\[
\begin{align*}
P &= P(\varepsilon, \phi) \\
M &= M(\varepsilon, \phi) \\
V &= V(\varepsilon, \gamma)
\end{align*}
\] (2.19)

with the section stress and strain vectors being

\[
\mathbf{s}(x) = \begin{bmatrix} N(x) \\ M(x) \\ V(x) \end{bmatrix} \quad \mathbf{\varepsilon}(x) = \begin{bmatrix} \varepsilon(x) \\ \phi(x) \\ \gamma(x) \end{bmatrix}
\] (2.20)

The section stiffness matrix can be written as

\[
\mathbf{K}_s = \begin{pmatrix} K_{s11} & K_{s12} & 0 \\ K_{s12} & K_{s22} & 0 \\ \text{sym} & \text{sym} & K_{s33} \end{pmatrix}
\] (2.21)

The terms \( K_{s11}, K_{s12}, K_{s22} \) come from the fiber model and are obtained through the integration of concrete and steel fibers stiffnesses. The complete expression of these terms is reported in Eq. 3.5. The term \( K_{s33} \) represents the shear stiffness and is a function of shear distortion \( \gamma \) and of the maximum strain \( \varepsilon_{\max} \) which occurs during
the analysis. The terms $K_{s13}$, $K_{s23}$ are equal to zero because interaction between shear and flexure was not introduce explicitly in the definition of the section stiffness matrix. The term $K_{s33}$ is deduced by considering the current slope of the shear cyclic law and is defined as

$$K_{s33} = \frac{\partial [V(\gamma, \varepsilon_{max})]}{\partial \gamma}$$

(2.22)

The $V - \gamma$ primary skeleton curve was defined using a non-linear truss model, composed of a simple three-chord strut-and-tie mechanism. The response envelope under cyclic loading was obtained by modifying the primary monotonic curve as a function of the section ductility by means of section axial deformation.

An extension of the original fiber beam element to account the shear stresses, deformations and stiffness and their interaction with axial force and bending moment was implemented by Petrangeli et al. (1999). The fiber model required a reformulation of a biaxial concrete constitutive law that was based on the microplane theory. The shear mechanism was modeled at the cross-section level assuming that the strain field is given by the superposition of the longitudinal strain field obtained on the base of plane section hypothesis and an assigned distribution over the cross-section for the shear strain field. The transverse deformations were determined imposing equilibrium between the concrete fibers and transverse reinforcement. The result is a full 3 x 3 section stiffness matrix that couples axial, bending, and shear responses. This model, respect to the afore mentioned approaches based on truss and strut-and-tie analogies, describes axial, flexural and shear response and their interaction account on physical bases. The approach presented by Petrangeli et al. (1999) is accurate and rational; however, it demands a considerable additional computational cost respect to the traditional Euler-Bernoulli fiber element.

A force-based Timoshenko fiber beam element including shear effect was presented by Marini and Spacone (2006). The classical fiber approach for axial and bending effects was used together a simple non-linear shear force-shear deformation phenomenological law at section level. Shear and bending resulted uncoupled in the section constitutive law but the equilibrium along the element couples shear and bending forces at the element level. This approach, partly utilized in this work, will be discussed with greater details in the next chapter.
Chapter 3
Modeling approaches

Numerical models capable of describing the non-linear behavior of structural members (beams, columns, slabs, etc.) are required to study the damage process and the vulnerability to collapse of RC structures under earthquake loading. A brief description of different modeling scales, highlighting the diversity and complexity of the most common approaches, includes:

1. Micro scale models (Zienkiewicz et al., 2005), characterized by a continuous media approach where constitutive laws, expressed in local variables, are independent of geometry. Although the finite element method appeals for its accuracy and for its capacity to model different phenomena and their interaction, it requires the solution of a large system of equations, and the integration of stress in two or three space directions. In addition, the management of the input-output quantities is not a simple task because of the huge amount of data.

2. Meso scale models (Taucer et al., 1991), which are situated between a local and global formulation, are based on some structural theories (2D or 3D multi-fiber beam models). The equilibrium and kinematic conditions are handled at global scale whereas stresses and internal variables are calculated at local level; stresses are subsequently integrated over the section. This intermediate scale allows on one hand the capitalization of the simplified kinematic hypotheses of the theory of beams (Navier-Bernoulli or Timoshenko) with a consequent reduction of the size of the equation system, and on the other hand permits a fast integration of the constitutive laws thanks to the uniaxial hypothesis of the stress-strain field.

3. Macro scale models (Miramontes et al., 1996), which are formulated in generalized variables, as internal forces, rotations, curvatures, etc. The integration of stresses at the cross-section level is removed and the number of
points defining the constitutive law is usually reduced (trilinear or bilinear constitutive laws). These models are formulated considering simplified kinematic hypothesis. The concrete-steel behavior is described in generalized variables as uniaxial constitutive laws based on experimental observations \((N - \varepsilon), (V - \gamma), (M - \phi)\) or based on classical theories of plasticity or damage mechanical models (i.e. yielding surface). The loss of local information represents the principal drawback of these models. Furthermore they should be specialized for each cross-section and particular loading histories (earthquakes, explosions, impacts). The reduction in accuracy is paired by the smaller computational effort required, in terms of CPU time, amount and simplicity of input data required and output data provided, that are usually in terms of quantities familiar to the design engineers.

To combine the advantages of different scales of analysis a mixed approach can be utilized. For the micro and meso scale the mixed approach was explored by Coronelli et al. (2005), for the micro and macro scale by Coronelli and Mulas (2001), for the meso and macro scale by Davenne et al. (2003) and for all three scale by Coronelli et al. (2006). In the present work, the meso and macro scale will be adopted and their description will be presented in the next section. Even though at different levels of refinement, in both approaches the beam element was derived to represent a whole monodimensional structural element, as a beam, a column or a shear wall; moreover both modeling scales adopt a flexibility approach.

3.1 The spread plasticity model

The beam spread plasticity model here adopted (Coronelli and Mulas, 2001) is shown in Figure 3.1. The model consists of three components in series: (a) the central beam, itself subdivided into three sub-zones, an elastic zone in the centre and two inelastic zones at the two ends, taking into account yielding and cracking spread along the beam; (b) two non-linear rotational springs describing the fixed-end rotations (FER) at the beam-column interface; (c) two rigid offset zones accounting for the finite dimensions of the beam-column joint. The components (b) and (c) will be not activated in this work.

A flexibility approach is adopted to derive the element tangent stiffness matrix, with a twofold purpose: to avoid both the explicit insertion into the structure stiffness matrix of the DOFs due to the springs’ relative rotations and the definition of the beam shape functions after the onset of steel yielding and the spreading of concrete cracking. The element tangent stiffness matrix is calculated by inverting the flexibility matrix, which is obtained by summing the contributions of the central beam and of the springs. The former is derived by means of an approximate integra-
tion of the curvatures $\phi$ along the beam, assuming a linear bending moment increment diagram; the latter is directly derived from the hysteretic relation between the moment $M$ and the FER $\theta$. The inelastic zones, initially of zero length, can spread when the end sections of the central beam are in the strain hardening range. Further details on the flexibility formulation and on the computational aspects pertaining to the introduction of the springs can be found in Mulas and Filippou (1990).

The original version of the beam model makes use of the Clough’s model (Clough, 1966) depicted in Figure 3.2, based on a bilinear primary curve with ascending post-yielding branch; different values for the yielding moment $M_y$ on the positive and negative sides are possible. The elastic stiffness $K$ is the secant value from the origin to the yielding point; the yielding branch has a slope $pK$, where $p$ is the strain-hardening ratio, governed by the steel reinforcement properties. Unloading takes place with the elastic slope; reloading after a partial unloading maintains the elastic slope up to the unloading point, without causing any energy dissipation (line $IJ$ in Figure 3.2). The reloading branch has a reduced stiffness $sK$ (being $s < 1$), governed by the introduction of two focal points (for positive and negative bending), which are the points corresponding to the yield point (line $DE$ in Figure 3.2) or to the maximum excursion previously experienced in the inelastic range (line $GC$ in Figure 3.2). The reloading branch is the segment connecting the end of the last, complete, unloading branch to the focal point. The beam model does not account for the M-N interaction.

### 3.1.1 Hysteretic models with stiffness deterioration

The original Clough’s model does not take into account phenomena that are important for reproducing correctly the response of structures subjected to seismic excitation. To this aim, the Clough’s model was improved in this work introducing two aspects: the stiffness deterioration taking place in the unloading branches and
the pinching behavior due to shear effects in the reloading branches.

The first aspect was introduced in this work through the Q-Hysteresis model (Saiidi and Sozen, 1979). In this model, when the yielding moment is exceeded, the unloading stiffness takes the value $cK$ (being $c < 1$), as depicted in Figure 3.3. To determine the non-dimensional parameter $c$ a reference value of the ratio between the yield $\phi_y$ and the maximum $\phi_m$ value of the curvature is defined, given by the minimum value of the ratio on the positive and negative side:

$$\left(\frac{\phi_y}{\phi_m}\right)_\text{ref} = \min\left(\frac{\phi_y^+}{\phi_{\text{max}}}, \frac{\phi_y^-}{\phi_{\text{min}}}\right) \quad (3.1)$$

The parameter $c$ depends on the previous loading history according to the relation:

$$c = \left(\frac{\phi_y}{\phi_m}\right)_\text{ref}^\alpha \quad (3.2)$$

The exponent $\alpha$ controls the slope of the unloading branch after yielding; in this work was taken equal to 0.5 as suggested by Saiidi and Sozen (1979).

The description of pinching effects due to shear behavior in repeated cycles at the same deformation level follows the proposal of Ibarra et al. (2005). The hysteretic model in Figure 3.4 differs from the Clough’s model in the reloading branch, divided in two parts by a “break point”; its position depends on the maximum perma-
3.1 THE SPREAD PLASTICITY MODEL

The spread plasticity model is a hysteretic model that describes the relationship between moment and curvature, accounting for plastic deformations and load experienced in the direction of loading. In Figure 3.4 the abscissa of the break point (point I) is defined by the parameter $\kappa_d$, which modifies the value of the permanent curvature after unloading (point D). If the yielding moment is still not reached, the abscissa of the break point is zero (point E). The slope $K_{rel,a}$ of the first part of the reloading branch is defined by the parameter $\kappa_f$, which modifies the maximum “pinched” strength (points $E'$ and $I'$ of Figure 3.4). The abscissa of the points $E'$ and $I'$ is governed by the yielding and maximum curvatures. The parameters $\kappa_d$ and $\kappa_f$, having an upper bound equal to 1, govern the amount of pinching that will be present in cyclic loading; the upper bound corresponds to the absence of pinching, lower values correspond to an increasing stiffness degradation. Once the break point is reached (points E and I), the reloading path is directed towards the maximum deformation of previous cycles in the direction of loading, with a slope $K_{rel,b}$. If the absolute deformation at reloading (point O, Figure 3.5) is larger than the absolute value of $(1 - \kappa_d)\phi_{per}$, the reloading path consists of a single branch that is directed towards the previous maximum deformation in the direction of loading.

Finally, a third hysteresis model was adopted, encompassing both the stiffness deterioration in the unloading branches and the pinching effects.
3.2 The fiber beam-column element

The formulation of the fiber beam-column element here adopted is based on the Navier-Bernoulli’s hypothesis: the sections remain plane and normal to the reference axis during the deformation history (Spacone et al., 1996). In the case of reinforced concrete members this assumption implies a perfect bond between reinforcing steel and concrete. Based on a flexibility approach, the beam-column elements determine the section forces (moment and axial load) from interpolation of the element end forces and integrate the resulting section deformations (curvatures and axial strains) over the length of the element to determine the element end deformations (rotations and axial lengthening). In case that the axial load is acting on the structure, the fiber elements are able to model the axial lengthening of the columns resulting from lateral displacements, by capturing the moment-axial load interaction in terms of stiffness and strength. The second order $P - \Delta$ effect is also incorporated in the element formulation (Filippou and Fenves, 2004). The element does not include the buckling of the longitudinal reinforcing bars. Two main advantages stem from the adoption of a force-based elements: (a) equilibrium between nodal forces and section forces can be enforced exactly and this permits using fewer elements for the representation of the non-linear behavior of a structure; (b) no numerical difficulties arise from the possibility of softening and strength loss of individual sections (Spacone et al., 1996).

The non-linear behavior of the materials is tracked in three cross-sections, located...
at the Gauss-Lobatto integration points along the length of the element. This choice provides the critical section forces and deformations at the ends of the element. The sections are subdivided into concrete and steel fibers; the mechanical and geometric characteristics of each fiber are required. Figure 3.6 shows a representation of the section subdivision into fibers for the beam element in the local reference system. In the formulation adopted the material models are uniaxial. The material models adopted for representation of the behavior of concrete and steel are described in the following sections.

A force-based two-dimensional element based on the Timoshenko beam theory is also used in this study. The element formulation follows the force-based formulation presented in Spacone et al. (1996) with the modifications proposed by Marini and Spacone (2006) to include shear effects. A phenomenological shear force–shear deformation law is used at the section level, combined with a classical fiber section model for the axial and bending effects. Shear and bending are decoupled in the section constitutive law, but equilibrium between them is enforced, thus bending and shear are coupled at the element level. The section force and deformation vectors $s(x)$ and $\varepsilon(x)$ for the uniaxial bending case are represented by the following equation:

$$
\begin{align*}
\mathbf{s}(x) &= \begin{bmatrix} N(x) \\ M(x) \\ V(x) \end{bmatrix} \\
\mathbf{\varepsilon}(x) &= \begin{bmatrix} \varepsilon_0(x) \\ \phi(x) \\ \gamma(x) \end{bmatrix}
\end{align*}
$$

Figure 3.5: Pinching hysteretic model: modification if reloading deformation is to the right of break point.
The resulting section resisting forces and the tangent stiffness matrix are

\[
\mathbf{s}(x) = \begin{cases} 
  \sum_{\text{fiber}=1}^{n(x)} \sigma_{\text{fiber}} A_{\text{fiber}} & \\
  - \sum_{\text{fiber}=1}^{n(x)} \sigma_{\text{fiber}} A_{\text{fiber}} y_{\text{fiber}} & \\
  V = V(\gamma) 
\end{cases}
\]  

(3.4)

\[
\mathbf{k}(x) = \begin{cases} 
  \sum_{\text{fiber}=1}^{n(x)} E_{\text{fiber}} A_{\text{fiber}} & \\
  - \sum_{\text{fiber}=1}^{n(x)} E_{\text{fiber}} A_{\text{fiber}} y_{\text{fiber}} & \\
  0 & \\
  \sum_{\text{fiber}=1}^{n(x)} E_{\text{fiber}} A_{\text{fiber}} y_{\text{fiber}}^2 & \\
  0 & \\
  V = V(\gamma) 
\end{cases}
\]  

(3.5)

where \(n(x)\) is the number of fibers in the section, \(\sigma_{\text{fiber}}\) is the fiber stress, \(E_{\text{fiber}}\) is the fiber tangent modulus, \(A_{\text{fiber}}\) is the fiber area, and \(y_{\text{fiber}}\) is the distance from the fiber centroid to the section reference axis. \(V = V(\gamma)\) indicates that the shear force
is computed directly from the shear deformation $\gamma$ via the selected shear response law, and $dV/d\gamma$ indicates that the shear tangent stiffness is the derivative of the shear law. A simple bilinear $V - \gamma$ law is used and will be discussed later.

### 3.2.1 Concrete stress-strain relation

In order to compute the concrete stress in each layer, a material law describing the concrete stress-strain relation under arbitrary strain histories is needed. Many studies have been conducted on the stress-strain relationship of concrete confined by transverse reinforcement under compression. Observations and laboratory tests have shown that if the compression zone of a concrete beam or column is confined by closely-spaced stirrup ties, hoops or spirals, the ductility of concrete is significantly enhanced and the member can sustain deformations of large curvature (Kent and Park, 1971; Park et al., 1982; Ozcebe and Saatcioglu, 1987). The monotonic envelope curve of concrete in compression follows the model of Kent and Park (1971) that was later extended by Scott et al. (1982) to include high strain rate. The so-called modified Kent and Park concrete model offers a good balance between accuracy and simplicity even though more accurate and complete models have been published since (Mander et al., 1988). The formulations of the stress-strain relations of confined and unconfined concrete based on the modified Kent and Park model are summarized here.

The constitutive model consists of an ascending branch represented by a second-degree parabolic curve and a descending linear part, as shown in Figure 3.2. The ascending parabola is expressed by Eq. (3.6)

$$f_c = k f'_c \left[ \frac{2 \varepsilon_c}{\varepsilon_0 k} - \left( \frac{\varepsilon_c}{\varepsilon_0 k} \right)^2 \right] \quad \varepsilon_c \leq k \varepsilon_0$$

(3.6)

where $\varepsilon_c$ is the longitudinal concrete strain, $f'_c$ is the compressive strength of concrete, $\varepsilon_0$ is the strain of unconfined concrete corresponding to $f'_c$, and $k$ is a confinement coefficient greater than or equal to 1. For unconfined concrete, the parameter $k$ is equal to one.

For strain greater than the value corresponding to the peak stress, the softening branch of the stress-strain relationship is approximated by a straight line having equation:

$$f_c = k f'_c \left[ 1 - Z_m (\varepsilon_c - \varepsilon_0 k) \right] \quad \varepsilon_c > k \varepsilon_0$$

(3.7)

where $Z_m$ is the strain softening slope.
Figure 3.7: Constitutive model for concrete.

The definition of the parameter $k$, as proposed by Scott et al. (1982) is

$$k = 1 + \rho_v \frac{f_{syt}}{f'_c}$$

(3.8)

where $f_{syt}$ characterizes the yielding stress of the transversal reinforcement, and $\rho_v$ stands for the volumetric confinement ratio

$$\rho_v = \frac{A_{sw}l_w}{b_c h_c s}$$

(3.9)

In this equation $A_{sw}$ defines the cross-sectional area of the stirrups that provide confinement (with perimeter $l_w$ and separation $s$), and $b_c \times h_c$ measures the area of the concrete core effectively confined.

As for parameter $Z_m$, an estimation of the form:

$$Z_m = \left[ \frac{0.5}{\left( \frac{3 + 0.29 f'_c}{145 f'_c - 1000} \right) + 0.75 \rho_v \sqrt{\frac{b_c}{s}} - \varepsilon_0 k} \right]$$

was assumed.

For confined concrete, a perfectly plastic residual behavior was assumed at high strain levels, to account for the load carrying capacity of crushed concrete still effectively confined by the transverse steel. The confinement effect on the strength of concrete represented by the confinement coefficient $k$ increases the concrete peak stress from $f'_c$ to $k f'_c$. It was assumed that confined concrete can sustain a constant stress of $0.2 k f'_c$ at strains greater than $\varepsilon_{20c}$. 
On the tension side, a linear elastic branch with slope equal to the initial Young modulus in compression was assumed up to the stress peak $f_{ct}$. The stress peak $f_{ct}$ is followed by a linear softening branch with slope $E_{ct}$ up to a strain value corresponding to no-stress transfer.

The hysteretic behavior of the concrete model includes the gradual degradation of stiffness under unloading and reloading in compression, as shown in Figure 3.8. The concrete material model does not include loss of strength under cyclic load. More details are provided by Yashin (1994).

![Figure 3.8: Hysteretic concrete stress-strain behavior.](image)

### 3.2.2 Steel stress-strain relation

The reinforcing steel stress-strain behavior is described by non-linear model of Menegotto and Pinto (1973), as modified by Filippou et al. (1983) to include isotropic strain hardening.

The model, as presented in Menegotto and Pinto (1973), takes on the form

$$\sigma^* = b\varepsilon^* + \frac{(1 - b)\varepsilon^*}{\varepsilon_0 - \varepsilon_r} \left[1 + (\varepsilon^*)^R\right]^{1/R}$$  \hspace{1cm} (3.11)

where

$$\varepsilon^* = \frac{\varepsilon - \varepsilon_r}{\varepsilon_0 - \varepsilon_r}$$  \hspace{1cm} (3.12)

and

$$\sigma^* = \frac{\sigma - \sigma_r}{\sigma_0 - \sigma_r}$$  \hspace{1cm} (3.13)

Eq. (3.11) represents a curved transition from a straight line asymptote with slope
$E_0$ to another asymptote with slope $E_1$. The quantities $\sigma_0$ and $\varepsilon_0$ are the stress and strain at the point where the two asymptotes of the branch under consideration meet (point A in Figure 3.9); similarly, $\sigma_r$ and $\varepsilon_r$ are the stress and strain at the point where the last strain reversal with stress of equal sign took place (point B in Figure 3.9); $b$ is the strain hardening ratio, that is the ratio between $E_0$ and $E_1$, and $R$ is a parameter that influences the shape of the transition curve and allows a good representation of the Bauschinger effect. As indicated in Figure 3.9, $(\varepsilon_0, \sigma_0)$ and $(\varepsilon_r, \sigma_r)$ are updated after each strain reversal.

\[ R = R_0 - \frac{a_1 \xi}{a_2 + \xi} \]  \hspace{1cm} (3.14)

where $\xi$ is updated following a strain reversal. $R_0$ is the value of the parameter $R$ during first loading and $a_1$, $a_2$ are experimentally determined parameters to be defined together with $R_0$.

The steel model is computationally efficient and agrees well with experimental results from cyclic tests on reinforcing steel bars; it remains one of the most accurate model in the literature.
Chapter 4

Lightly reinforced shear wall (CAMUS I)

4.1 Description of the specimen

4.1.1 Geometry and mass

The first RC shear wall at study, named CAMUS I, is a specimen with limited longitudinal and shear reinforcement ratio, as commonly used in France in seismic zones. The design of the specimen, based on design spectrum $S_1$ of the French code PS92 (NF-P-06-013, 1995), follows the concept of a structural multi-fuse (Bisch and Coin, 1998); yielding of the vertical reinforcement is allowed at several floors, opposite to the usual concentration of damage at the lowest story.

The 1/3$^{rd}$ scale model at study, 5.1 m tall and having a total mass of 36000 kg, is composed of two parallel 5 floor RC walls without openings, linked by 6 square floors, anchored to the shaking table through a heavily reinforced concrete footing. The wall has an aspect ratio equal to 2.65. A view and the geometry of the wall are depicted in Figures 4.1 and 4.2; the nominal wall thickness is 6.00 cm. The total mass of the specimen is such as to give approximately a vertical stress value of 1.6 MPa at the base, typical for this kind of structures. The distribution of the additional masses necessary for the similitude law is shown in Figure 4.3.

The shaking table has a total mass of 25000 kg and is restrained through four vertical rods. Two rods are situated at the level of the centre of the wall and two at the extremities (7.06 m between the extreme rods). The rods are connected to the table at a level 1.02 m under the walking level of the shaking table. The axial stiffness $k$ of each rods has been estimated equal to 400 MN/m in the plane of the mock-up model.
4.1.2 Materials

The wall was cast in a C20 micro-concrete; an average concrete strength of 35 MPa was obtained from cylinder tests. Thickness of the cover concrete is 1 cm. The diameters of the reinforcement steel bars used are 4.5, 5.0, 6.0 and 8.0 mm. The bars of diameter 4.5 and 5 mm have small variations of the cross-section to ensure bond, while the remaining ones are ribbed high-bond bars. The bars were positioned in three groups, at the centre of the cross-section and at the two ends. The nominal yielding stress of steel reinforcement was $f_y = 500$ MPa. The distribution of the longitudinal reinforcement bars for the wall is described in Figure 4.3. The transversal steel used for the wall follows the design recommendations (the stirrups spacing is about 60 mm with a diameter equal to 3.0 mm) and is placed around the bar layers. Two types of stirrups were used: closed stirrups from the wall base to below the third floor curtailment and open U-shaped stirrups on the upper part of the structure. Tensile tests were performed for the different reinforcing steel bars except for the 3 mm diameter stirrup bars; actual yielding stress are reported in Table 4.1. No transverse reinforcement is placed throughout the whole width of the wall, but only around the longitudinal bar layers. The steel reinforcement ratio changes 10 cm just below each floor, as the walls were cast producing construction
4.1 DESCRIPTION OF THE SPECIMEN

Figure 4.2: The overall geometry of the wall CAMUS I (Combescure and Sollogoub, 2004).

joints in correspondence of each storey and the longitudinal bar curtailments are placed below each floor close to this location.

<table>
<thead>
<tr>
<th>$\phi$ [mm]</th>
<th>Type</th>
<th>$f_y$ [MPa]</th>
<th>$f_r$ [MPa]</th>
<th>$\varepsilon_{gt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>With diameter variation</td>
<td>465</td>
<td>520</td>
<td>2.5%</td>
</tr>
<tr>
<td>5</td>
<td>With diameter variation</td>
<td>570</td>
<td>605</td>
<td>2.5%</td>
</tr>
<tr>
<td>6</td>
<td>High bond</td>
<td>515</td>
<td>565</td>
<td>5.5%</td>
</tr>
<tr>
<td>8</td>
<td>High bond</td>
<td>430</td>
<td>450</td>
<td>5.0%</td>
</tr>
</tbody>
</table>

Table 4.1: Steel bar characteristics (Combescure and Sollogoub, 2004).

4.1.3 Loading program

The specimens of the walls were tested on the Azalée shaking table of the CEA at Saclay (France) and subjected to a sequence of five seismic inputs. Two tests adopted the artificial signal Nice, representative of a far field earthquake, at 0.25 g
Figure 4.3: Reinforcement and masses for the wall CAMUS I (Combescure and Sollogoub, 2004).

(RUN1) and 0.41 g (RUN4); two other tests adopted the natural signal San Francisco, representative of a near field earthquake at 0.13 g (RUN2) and 1.11 g (RUN3). The accelerograms applied at the base of the wall, are depicted in Figure 4.5; the time axis describes the temporal sequence used in the numerical analyses. An ultimate RUN5, (Nice 0.72 g) lead to the rupture of some rebars and to maximum displacement corresponding to 1% drift.

The acceleration and displacement response spectra of the input motions (5% damping) are presented in Figure 4.6. RUN1 and RUN4 (from artificial NICE signal) are rich in terms of frequencies. RUN3 (from San Francisco earthquake) is a short signal; it has a thinner bandwidth of higher accelerations. In Figure 4.5 RUN2 is not showed due to its low peak acceleration (0.15 g).

The first two natural frequencies of the mock-up before the first test were 7.24 Hz and around 33 Hz (in-plane bending) respectively; the first vertical mode has a natural frequency of 20 Hz. The structure showed almost no damage after the first two runs; in spite of the PGA values, RUN4 was more damaging than RUN3, but it was applied to a partially damaged structure. The results of RUN5 have not been analyzed here, since the wall was so seriously damaged by the previous runs that
4.1 DESCRIPTION OF THE SPECIMEN

4.1.4 Specimen instrumentation

The instrumentation was designed in order to give information on the motion of the shaking table and on the global and local behavior of the wall; to this aim 64 instruments were used during each tests. The horizontal displacement, velocity and acceleration of the shaking table were measured in its centre. Two vertical accelerometers were used to check a possible rocking or vertical motion of the shaking table. Six transducers, fixed at a vertical beam, were employed to measure the horizontal displacements of each levels of the left wall. The floor horizontal displacements relative to the wall base were deduced directly from these measurements. Six accelerometers, positioned in the middle of each wall, were necessary to detect the horizontal floor accelerations. An accelerometer positioned on a corner at the top allowed to check the out-of-plane motion of the wall. Accelerometers placed at both ends of the left wall measured the vertical accelerations; at the fifth level a single accelerometer fixed at the center cross-section was used. Since the accelerations absolute and not relative to the shaking table were measured, the inertial forces and so the bending, normal and shear forces can be directly computed from the product of the floor masses and the horizontal accelerations recorded.

Figure 4.4: Central and lateral reinforcement details.

the reproduction of this run is out of the scope of this work.
**Figure 4.5:** Input ground motions: (a) Nice 0.25 g; (b) San Francisco 0.13 g; (c) San Francisco 1.11 g; (d) Nice 0.41 g.

**Figure 4.6:** (a) Acceleration and (b) displacement response spectra (5% damping).
4.1 DESCRIPTION OF THE SPECIMEN

Information about the local behavior were provided by the readings of the displacement sensors (LVDTs) and strain gauges. Several strain gauges served to measure the strains of the steel reinforcements at the base of the 3 first levels of the left wall; they were placed just above the floor at the level of the construction joints. In order to detect potential opening of cracks, several LVDTs were fixed on both extremities of the left wall, from base level to fourth level, and at the center of the base level.

4.1.5 Experimental results

Table 4.2 lists a summary of the envelope of the experimental values that will be compared to the numerical results (Combescure and Sollogoub, 2004); only global response parameters are here considered. The internal forces presented in Table 4.2 were computed from the recorded horizontal absolute accelerations and the estimate of the masses of each floor. The values of axial force listed in Table 4.2 represent the maximum dynamic excursion in compression and in tension.

In the presentation of the numerical results, particular attention will be devoted to the response of the wall to the strong motions of runs 3 and 4, where the struc-
Table 4.2: Envelope of the experimental response parameters.

<table>
<thead>
<tr>
<th>RUN</th>
<th>$u_{top}$ [mm]</th>
<th>$a_{top}$ [g]</th>
<th>$V_{base}$ [kN]</th>
<th>$M_{base}$ [kNm]</th>
<th>$N_{base}^-$ [kN]</th>
<th>$N_{base}^+$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7</td>
<td>0.677</td>
<td>65.9</td>
<td>211</td>
<td>36.5</td>
<td>44.3</td>
</tr>
<tr>
<td>2</td>
<td>1.54</td>
<td>0.284</td>
<td>23.5</td>
<td>75.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>13.2</td>
<td>1.16</td>
<td>106</td>
<td>280</td>
<td>105</td>
<td>102</td>
</tr>
<tr>
<td>4</td>
<td>13.4</td>
<td>0.933</td>
<td>86.6</td>
<td>280</td>
<td>51.9</td>
<td>50</td>
</tr>
</tbody>
</table>

- Compression
+ Tension

ture reaches the maximum values of displacements and internal forces. The time intervals from 94 s to 96 s (RUN3) and from 119 s to 121 s (RUN4), respectively, where the maximum values are attained, are analyzed in detail in Figures 4.8 and 4.9 showing the time history of displacement at the top, of the bending moment, shear force and dynamic axial force at the base of the wall. The axis time is referred to the complete time sequence of the four runs. Each parameter is normalized to the corresponding maximum value reached during the four input motions.

Figure 4.8: Top displacement, bending moment, shear force and dynamic axial force at the base of the wall for the peak of RUN3.

The experimental results show a remarkable variation of the axial force at the base of the wall. This phenomenon can be explained in the following manner: as the cracks close, a shock is induced; the flexural stiffness changes suddenly recovering partially its uncracked value and the second mode (vertical mode) is excited (Combescure and Sollogoub, 2004; Kotronis et al., 2005). The variation of the
vertical dynamic forces can even double or cancel the axial force due to the dead weight of the specimen. The crack closure (displacement or moment equal to zero) is characterized by a sudden and high compression. The dynamic axial compression is then followed by a tension force of the same order of magnitude. From Figures 4.8-4.9 it is possible to observe that the displacement is in phase with the bending moment and both follow the first frequency of the wall. Figure 4.9 shows a 1/3 ratio between the peaks of the top horizontal displacement and the base axial force, in accordance with the frequency ratio between the first flexural and extensional modes (7.24 Hz and 20 Hz, respectively).

**Figure 4.9:** Top displacement, bending moment, shear force and dynamic axial force at the base of the wall for the peak of RUN4.

### 4.2 Numerical model of the wall

For both the modeling approaches herein considered a structural model was prepared. Nevertheless, the two models share some characteristics. Due to the symmetry of the specimen, only a single wall was analyzed; a stick model, depicted in Figure 4.10 was adopted. The model has only one element (spread plasticity or fiber beam-column element depending on the case we are considering) for each story. The masses were introduced at level of each story. The basement of the wall, having a larger thickness and percentage of reinforcement, was modeled through an elastic beam element, with an uncracked concrete gross-section and Young’s modulus equal to 28 GPa. Finally, the shaking table was modeled with one linear elastic element with high stiffness, which was fixed by a few springs having a stiffness equivalent to that of the rods of the foundation system. Since only half of the...
structure was analyzed, only half of the stiffness of the rod support was considered.

### 4.2.1 Spread plasticity model

In the spread plasticity model, the inertial forces associated to the masses act only in correspondence of the horizontal degrees of freedom of each floor. The stiffness of the foundation system of the shaking table was simulated through two springs, one translational (in vertical direction) and one rotational.

The initial stiffness of a RC wall depends on several parameters, as material properties, dimensions, reinforcement quantities, stress levels, concrete cracking, axial compression, boundary conditions and bar slippage. Hence, uncertainties are present in the stiffness values needed to characterize the predominant dynamic wall properties under seismic loads and to ensue peak displacements and forces. Moreover, the behavior of reinforced concrete sections is characterized by a tri-linear monotonic moment-curvature relationship. However, the macro scale model adopts a bilinear relation; therefore a single effective initial stiffness needs to be selected for the elements. This stiffness, used for both positive and negative bending moments, has to represent the initial inelastic response of the wall.

The value of $K_{eff} = 0.7EI_g$, where $I_g$ is the moment of inertia of the uncracked concrete gross-section, corresponds to the Canadian state-of-the-practice as recommended in CPCA (1995) and used by several researchers (Harries et al., 1998; Stonehouse et al., 1999; Tremblay et al., 2001). Paulay and Priestley (1992) recommend the following expressions:

$$K_{eff} = E \left[ I_g \left( \frac{100}{f_y} + \frac{P_u}{f'_c A_g} \right) \right]$$

(4.1)

where $P_u$ is the axial load considered to act on the wall during an earthquake, taken positive in compression, and $f_y$ is in MPa. The values provided by Eq. 4.1 vary from $0.35EI_g$ at the base of the wall to $0.26EI_g$ in the upper stories, in agreement with the values recommended in NZS (New Zealand Standards, 1995) for $N/f'_c A_g \approx 0.1$. The United States state-of-practice, as indicated in FEMA (1997), recommends $K_{eff} = 0.5EI_g$. Two different modal analyses, both accounting for the presence of the shaking table, have been performed with $K_{eff} = 0.5EI_g$ and $0.35EI_g$, providing for the first natural frequency 7.31 Hz and 6.32 Hz respectively. Even though the highest value of the flexural stiffness gives a better match with the experimental value of 7.24 Hz, in this study the value $K_{eff} = 0.35EI_g$ was adopted for the wall ($E = 30600$ MPa and $I_g = 2.81 \times 10^{10}$ mm$^4$). In fact, the highest value matches better the initial stiffness but the lowest value represents better the stiffness after cracking and the yielding of the bars, that strongly affects the seismic response of the wall.
The shear deformability was subsequently added to the model with the lowest flexural stiffness. The shear effect was considered in an approximate way, only in the linear range and based on the hypothesis of constant shear over each element; however, the effective area of the cross-section was reduced by a factor 1.4 (instead of 1.2 arising from the usual beam theory) to roughly account for cracking. The introduction of shear deformability into the numerical model brings the first frequency down to 6.15 Hz.

The moment at yielding, $M_y$, is obtained from the moment-curvature relationship at a steel strain equal to 0.20%, as depicted in Figure 4.11. Since the M-N interaction is not accounted for by the spread plasticity model, two different values for $M_y$ were determined and adopted. One corresponds to the value $N_{stat}$ of the axial force due to the presence of static load; the other corresponds to the minimum value of the compressive force $N_{dyn}$ detected during the experimental tests. The strain hardening ratio $p$ (Figure 4.11) was selected equal to 2%. The values of the axial forces $N_{stat}$ and $N_{dyn}$ and the corresponding yielding moments $M_{y,stat}$ and $M_{y,dyn}$ at each floor, are listed in Table 4.3.

### 4.2.2 Fiber model

The analytical model adopted for the numerical simulation was developed using the finite element platform OpenSees (Mazzoni et al., 2002), created for earthquake engineering analyses. The inertial forces associated to the masses act in horizontal and
vertical directions. The cross-sections were subdivided into 72 concrete fibers and differ at the different levels for the quantity and position of the longitudinal bars. Each bar was described with a steel fiber having the property given by Combescure and Sollogoub (2004). In the model, two different zones of concrete material response were identified: confined concrete at the outer layers of the cross-section, and unconfined concrete in the interior. Due to the very low percentage of transverse steel reinforcement present in the central part of the wall, no increase was considered in the uniaxial strength of concrete due to confining effects. The confined and unconfined zones in the CAMUS I wall are shown in Figure 4.12. Table 4.4 summarizes the material properties used in the analyses for these two concrete zones. The mechanical properties of the materials were derived from compression, tension and splitting test results.

The shaking table was included in the model using a beam element with relevant high stiffness; the support rods were modeled with three translational springs, one

<table>
<thead>
<tr>
<th>Level</th>
<th>(N_{\text{stat}}) [kN]</th>
<th>(M_{y,\text{stat}}) [kNm]</th>
<th>(N_{\text{dyn}}) [kN]</th>
<th>(M_{y,\text{dyn}}) [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>163.0</td>
<td>357.5</td>
<td>61.0</td>
<td>266.7</td>
</tr>
<tr>
<td>2</td>
<td>130.0</td>
<td>260.7</td>
<td>45.4</td>
<td>189.7</td>
</tr>
<tr>
<td>3</td>
<td>97.6</td>
<td>165.3</td>
<td>31.6</td>
<td>108.7</td>
</tr>
<tr>
<td>4</td>
<td>64.8</td>
<td>87.0</td>
<td>15.7</td>
<td>50.6</td>
</tr>
<tr>
<td>5</td>
<td>32.0</td>
<td>65.5</td>
<td>5.0</td>
<td>34.1</td>
</tr>
</tbody>
</table>

Table 4.3: Axial forces and corresponding yielding moments at each floor.
located in the middle and the other two at the extremity of the shaking table.

A first structural model (considered as the “standard” model) was set up, based on the Bernoulli kinematics. Its first two eigenfrequencies are 9.1 Hz and 23.3 Hz for the first flexural and vertical mode respectively, in agreement with the results of an elastic model without shear deformability.

To investigate the effects of the shear modeling, a structural model adopting at each story a Timoshenko fiber element was developed. The model, named NLT, is based on the same mechanical and geometric properties just described. The shear force-deformation law to be inserted in each fiber element was obtained through a FE pushover analysis of the first story of the wall, considered as fixed at the base and subjected to a horizontal load applied at the top under displacement control. The numerical results for both the FE and the fiber models are depicted in Figure 4.13. The shear contribution to the top displacement is derived as the difference between the total and the flexural displacement. The first story rotation due to the flexural contribution is obtained as:

\[ \theta = \frac{V_1 - V_2}{l} \]  

(4.2)

where \( l \) is width of the wall and \( V_1 \) and \( V_2 \) are the vertical displacement at the top of the story level on the left and right corner respectively. To obtain the contribution of the flexural deformations of the first story to the top lateral displacements, the location of the center of rotation (centroid of the curvature distribution) over the first story must be assumed. The flexural displacement \( U_f \) at the first top story can be represented as

\[ U_f = \alpha \theta h \]  

(4.3)

where \( \theta \) is the first story rotation obtained from Eq. 4.2, \( h \) is the height of the first story and \( \alpha \) is the distance from the top of the first story to the centroid of the curvature distribution. Thus, \( \alpha \) varies from 0.5 for the rectangular distribution to 0.67 for the triangular distribution. A value of 0.67 was assumed based on the use of this value by others authors (Ali and Wight, 1990; Thomsen and Wallace, 1995). Finally, a constant distribution of shear strains is assumed, by dividing the displacement due to shear for the story height \( h \). The shear and the flexural contribution to

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>( f'_c )</th>
<th>( \varepsilon_0 \times 10^{-3} )</th>
<th>( \varepsilon_u \times 10^{-3} )</th>
<th>( k )</th>
<th>( Z_m )</th>
<th>( f_{ct} )</th>
<th>( E_{ct} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined</td>
<td>-35.0</td>
<td>-2.00</td>
<td>-4.45</td>
<td>1</td>
<td>408</td>
<td>3.0</td>
<td>3000</td>
</tr>
<tr>
<td>Confined</td>
<td>-40.0</td>
<td>-2.28</td>
<td>-10.8</td>
<td>1.14</td>
<td>74</td>
<td>3.0</td>
<td>3000</td>
</tr>
</tbody>
</table>

Table 4.4: Concrete material properties assumed in the analyses for the CAMUS I shear wall.
the total lateral displacement for the first story wall are depicted in Figure 4.13. A bilinear approximation was assumed for the numerical $V - \gamma$ relation, having an initial stiffness $K_0$ equal to $1290 \times 10^3$ kN up to $V_0 = 150$ kN, followed by a branch having slope $K_1$ equal to $216 \times 10^3$ kN. To simulate a higher shear deterioration, from the NLT model a further model named NLTD was created, adopting a bilinear shear force-deformation relation with the same stiffnesses $K_0$ and $K_1$ but with a value $V_0 = 50$ kN, equal to $1/3$ of the previous one. This choice can be motivated by the observation of the experimental results. A maximum shear value of 66 kN was recorded at the base during the first run, leading the wall out of the elastic range; so, it is reasonable to assume that the value of 50 kN can represent a separation value between elastic and inelastic response.

4.3 Numerical results

The non-linear dynamic analyses of the spread plasticity element were performed with a computer code named RCDYNA, *Reinforced Concrete DYNamic Analysis,*
previously developed (Mulas et al., 2000) and upgraded in this work with the modifications to the hysteretic relations shown in Chapter 3 (Martinelli and Mulas, 2006). The dynamic analyses for the fiber model were performed with the computation platform OpenSees (Mazzoni et al., 2002).

Some analysis features are common to both the codes. A viscous damping was used for the dynamic analyses, where the damping matrix is of the Rayleigh-type and expressed as a linear combination of the mass and initial stiffness matrices

$$
[C] = \alpha [M] + \beta [K_0]
$$

The two coefficients $\alpha$ and $\beta$ are calculated in order to obtain a damping ratio equal to 2% of critical damping for the first two modes (Chopra, 2001). The dynamic analysis was performed after a static analysis for gravity loads; the incremental equations of motion were integrated by the Newmark’s method of constant acceleration in which $\beta = 0.25$ and $\gamma = 0.5$. The Newton-Raphson method was adopted to eliminate the unbalanced loads at the end of each step. In the spread plasticity model the masses are only associated to the horizontal degree of freedoms; as a consequence the first two modes are to be intended as the first two flexural modes of the wall. In the fiber model, the first two modes of the model correspond to those of the wall, the first one flexural and the second one vertical.

Dynamic non-linear analyses were performed applying to the base of the shaking table the four accelerograms, from RUN1 to RUN4. These were applied reproducing exactly the experimental sequence, each separate by a 3 seconds of null signal which allows for a 95% reduction of the amplitude of free vibration at the end of the muted interval. The structural response to RUN4 can only be captured if this

Figure 4.13: Base shear force vs top displacement for the first story.
run is applied to the structure previously damaged by RUN3.

4.3.1 Spread plasticity models

For the spread plasticity model, a time step $\Delta t = 0.005$ s was used. In this study, five different modeling approaches using the spread plasticity model were adopted, differing in the value of the yielding moment and in the constitutive relation adopted. Two models of the wall, adopting the Clough’s model but differing in the value of $M_y$, as explained in previous section, were analysed. The model named $C_s$ and the one named $C_d$ makes use of the values $N_{\text{stat}}$ and $N_{\text{dyn}}$ of the axial force respectively. The structural response obtained through the $C_s$ model has a frequency content similar to that corresponding to the $C_d$ model. However, the response amplitude is not simulated in a satisfactory way. As an example of the relative performance of the two models, the time history of the bending moment and shear force at the wall base is plotted in Figure 4.14 for the strongest part of RUN1 and RUN3; the overestimation produced by the model $C_s$ is quite apparent especially for the RUN3. For this reasons the remaining models were based on the value of $M_y$ corresponding to $N_{\text{dyn}}$.

The third structural model ($Q$ model), adopts the Q-Hysteretic constitutive relation; the fourth structural model is based on the pinching hysteretic model ($P$ model) and the last one utilizes the pinching model combined with the Q-hysteretic model ($PQ$ model). In the last two models, the parameters $\kappa_f$ and $\kappa_d$ necessary to determine the position of the break point in the pinching branch were set equal to 0.8. Even though this value produces only a limited shear degradation, the introduction in the model of the pinching behavior has a positive effect in the response, as it will be shown in the discussion on the results.

The five structural models share the hypotheses concerning the plastic zones: the formation of the plastic hinge is allowed at both ends (top and bottom) of each element, and thus of each interstory. To avoid numerical problems during the solution, the plastic hinge cannot spread beyond a length of $0.06L$, $L$ being the interstory height. Some of these hypotheses are due more to a shortcoming of the code than of the model itself. No rigid-ends were considered in the model, thus extending the element deformability also to the zone where the floors frame into the wall. The non-linear rotational springs at the ends of the element were not activated in this work. In fact there was no experimental evidence of bar slipping phenomena in the wall during the tests, as in the beam-column joints of RC frames under earthquake excitation.
Figure 4.14: Comparison between $C_s$ and $C_d$ models: (a) RUN1, base bending moment; (b) RUN3, base bending moment and (c) RUN3, base shear force.
Prediction of extreme values

The capability to reproduce the seismic cycles can be first of all investigated by the prediction of the extreme values of the global parameters listed in Table 4.2, namely the horizontal displacement and acceleration at the top, the bending moment and the shear force at the base. The extreme values of the response are summarized in Table 4.5 for the five modeling approaches. The results due to RUN2 have been here omitted, since this is a low intensity event, leaving the structure mainly in the linear range, thus of limited significance in this work.

<table>
<thead>
<tr>
<th>Model</th>
<th>RUN</th>
<th>( u_{\text{top}} ) [mm]</th>
<th>( a_{\text{top}} ) [g]</th>
<th>( V_{\text{base}} ) [kN]</th>
<th>( M_{\text{base}} ) [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_s )</td>
<td>1</td>
<td>6.72</td>
<td>0.97</td>
<td>89.6</td>
<td>288.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.7</td>
<td>2.07</td>
<td>120.8</td>
<td>377.2</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>5.05</td>
<td>0.69</td>
<td>70.1</td>
<td>204.2</td>
</tr>
<tr>
<td>( C_d )</td>
<td>1</td>
<td>6.55</td>
<td>0.93</td>
<td>77.3</td>
<td>247.7</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.6</td>
<td>1.69</td>
<td>104.0</td>
<td>288.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6.05</td>
<td>0.70</td>
<td>76.9</td>
<td>195.8</td>
</tr>
<tr>
<td>( Q )</td>
<td>1</td>
<td>6.60</td>
<td>0.95</td>
<td>79.0</td>
<td>251.8</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.8</td>
<td>1.74</td>
<td>103.0</td>
<td>300.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6.68</td>
<td>0.80</td>
<td>83.8</td>
<td>218.5</td>
</tr>
<tr>
<td>( P )</td>
<td>1</td>
<td>6.55</td>
<td>0.93</td>
<td>79.8</td>
<td>248.5</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.6</td>
<td>1.74</td>
<td>102.1</td>
<td>288.8</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6.49</td>
<td>0.80</td>
<td>78.7</td>
<td>202.5</td>
</tr>
<tr>
<td>( PQ )</td>
<td>1</td>
<td>6.58</td>
<td>0.96</td>
<td>81.1</td>
<td>250.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.8</td>
<td>1.74</td>
<td>100.1</td>
<td>291.8</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>7.41</td>
<td>0.93</td>
<td>86.9</td>
<td>225.4</td>
</tr>
</tbody>
</table>

Table 4.5: Time histories extreme values.

To further investigate this point, the results of these models were examined in terms of percentage errors, both to understand the predictive capacity of the models and to compare their performance. Percentage errors with respect to the experimental results for the generic parameter \( P \) is computed as:

\[
e^\% = 100 \frac{P_{\text{numerical}} - P_{\text{experimental}}}{P_{\text{experimental}}} (4.5)
\]

Thus, positive and negative values of the percentage errors indicate overestimates and underestimates, respectively, of the experimental values. The results for the
percentage errors in terms of displacement and acceleration at top, bending moment and shear force at the base, are shown in Figure 4.15. The results for RUN2 were omitted also in this case. Local parameters as strains, even though experimentally measured, were not considered too. Their simulation is considered out of the scope of macro scale models.

All of the models underestimate the top displacement; the error is small for RUN1, around 20% for RUN3 but over 40% for RUN4. It must be kept in mind that the simulation of this run is quite a difficult task, due to the presence of significant damage induced by RUN3. The error is much larger when RUN4 is applied to the virgin structure; a similar finding was found by Coronelli et al. (2005) making use of a more refined fiber model. The top horizontal acceleration suffers from the highest error; however in this case an overestimation is found for RUN1 and 3, while an underestimation for RUN4. The same trend is shown by the shear force and the bending moment. The beam model is more accurate in reproducing the internal forces than in reproducing the displacement and acceleration; the maximum error does not exceed in any case 25%.

The results in terms of maximum errors further justify the choice of the axial force \( N_{dyn} \) as a base for the determination of the yielding moment \( M_y \). With the exception of top displacement for RUN1 and RUN3 and of the bending moment at the base for the RUN4, the model \( C_s \) always shows the worst performance and appears unable to capture the response of the wall.

The insertion of the stiffness degradation in the unloading branch, through the Q-hysteresis model, produces different effects on the \( C_d \) and \( P \) models. The results of the \( C_d \) and Q model are very close; for RUN1 and RUN3 the adoption of the Q model results in a decrease of precision; the opposite is true for RUN4. The differences between the \( P \) and \( PQ \) models appear to be higher, especially for RUN4, where the \( PQ \) model provides a very good approximation of all the values, with the exception of top displacement. The results of the four models seems on the whole comparable when RUN1 and RUN3 are at concern; however, the \( P \) and \( PQ \) models have a better performance when RUN4 is analysed.

The envelope along the height of the wall of the maximum displacements and interstory drifts is depicted in Figure 4.16 for the \( PQ \) model. The black line represents the experimental results while the grey line represents the numerical results; different markers have been used for each input motions. The envelopes point out the underestimation of the \( PQ \) model with respect to the experimental values. The experimental envelope of the interstory drift for the RUN1 suggests that the damage is concentrated at the first two stories; above the second story the structure behaves as a rigid body. The structure remains nearly undamaged during the RUN2. The high damage induced by RUN3 is quite apparent; during this input motion the damage spreads up to the fourth level. For this run, the agreement between measured
Figure 4.15: Percentage errors on maximum values: (a) top displacement; (b) top horizontal acceleration; (c) base shear force and (d) base bending moment.

and predicted displacement and interstory drift values is good. In the last run the damage increases in the first three stories in spite of the fact that the PGA of this run is half of the RUN3; however, this input motion was applied after the high damage induced by RUN3.

Figure 4.17 shows the maximum envelopes along the height of the wall of the numerical results produced by PQ model in terms of: (a) the horizontal floor acceleration, (b) story shear force and (c) overturning moment. The agreement between experimental and numerical horizontal floor acceleration values is more than satisfactory. The numerical horizontal acceleration overestimates the experimental values even though the profile along the height fits well the experimental one. The match between measured and numerical shear force and overturning moment is also satisfactory. It is worth noting that the experimental envelopes of the moments
4.3 NUMERICAL RESULTS

Figure 4.16: Measured and numerical PQ values envelopes: (a) horizontal displacement; (b) interstory drift.

along the height of the wall for RUN3 and RUN4 (Figure 4.17c) are coincident notwithstanding the RUN4 has a PGA that is less than half of that of RUN3.

Time histories and frequency content

The extreme values and the envelopes of the response provides a good indicator of the model performance; however, it is very important to investigate also the frequency content of the response, and the match between the experimental and numerical time history of the significant parameters. To this purpose, only the $C_d$ and $PQ$ model have been considered. In fact, these are considered the less and the most accurate, respectively, of the models having the yielding moment $M_y$ determinate by $N_{dyn}$. The results for the top displacement, base shear force, base bending moment and top horizontal absolute acceleration are shown in Figure 4.18-4.21, re-
respectively; black line denote experimental results, dashed and continuous grey lines denote $C_d$ and $PQ$ model results, respectively. Each figure contains four subplots, one for RUN1 (a) and RUN3 (b), and two for RUN4 (c-d). The results of RUN1 and RUN3 are analysed over a time interval of 6 s, while the results of RUN4 are analyzed over a larger time interval of 12 s. The frequency content of the response for all the four quantities is reproduced correctly by the two models. In the RUN1, the extreme value for all the parameters is reached at around 13 s, with an underestimation for the displacements and on overestimation for the internal forces and accelerations. A larger error appears at around 14 s, but in this case the numerical response is overestimating all parameters. The numerical results for RUN3 show a frequency content similar to that obtained experimentally; however, the top displacement and the base bending moment are fitted better than the shear base force and top acceleration. This result indicates that the first flexural mode is well represented by the model, while the model has some difficulties to represent the second and the higher modes. The successful simulation of this run is tied to the reproduction of the effects of the strong pulse in the signal at around 95 s. Also in this case the performance of the two models at study are similar. For the last run, the time history in terms of displacement (Figures 4.18c and d) shows a permanent distortion of the wall both at the beginning and at the end of the interval analysed. In the RUN4 both models fail in predicting the large response peak taking place at around 120 s, in terms of all the response parameters. However, the remaining response is reproduced very well, especially in terms of base shear (Figure 4.19d), indicating that, even though the model fails to reproduce the phenomena taking place at around 120 s, the evolution of the experimental frequency content has been fit well by the numerical model.

The analysis of the experimental results has shown a damage concentration at the third level (Figure 4.16b), differently from shear walls designed according to capacity design philosophy, where the damage is concentrated at the base of the walls. The experimental values of the steel strain confirm the main damage concentration at the third level. It is interesting to analyze the response in term of numerical moment-curvature for the numerical model (experimental moment-curvature data were not recorded). The plot of the moment-curvature relation for the cross-section at the base of level 3 is shown for RUN1 (Figure 4.22a, b), RUN3 (Figure 4.22c, d) and RUN4 (Figure 4.22e, f). The outcomes of the $C_d$ (Figure 4.22a, c, e) and $PQ$ model (Figure 4.22b, d, f) are compared. In this figure the demand posed over the structure by RUN3 can be observed also at local level. Even though a reduced amount of pinching was selected, the reduction of the area enclosed by the seismic cycles, accounting for shear effects, can be easily appreciated.
Figure 4.17: Measured and numerical PQ values envelopes: (a) horizontal floor acceleration; (b) shear force and (c) overturning moment.
Figure 4.18: Comparison of experimental and numerical values for the top displacement: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.19: Comparison of experimental and numerical values for the shear force at wall base: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.20: Comparison of experimental and numerical values for the overturning moment at wall base: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.21: Comparison of experimental and numerical values for the top absolute horizontal acceleration: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.22: Moment-curvature at the 3rd level of the wall, RUN1, RUN3 and RUN4: (a), (c), (e) $C_d$ model; (b), (d), (f) PQ model.
4.3 NUMERICAL RESULTS

4.3.2 Fiber models

For the fiber model, a time step $\Delta t = 0.001$ s is necessary to integrate the vertical oscillations depending on the second mode. This value of $\Delta t$ does not lead any way to satisfactory results in the reproduction of the axial force and consequently of the rocking effect, typical of the wall, due to the neutral axis shift. However, the overall structural response depends mainly on the flexural behavior and is almost unaffected by the variation of the time step from 0.005 s to 0.0001 s, as long as the first mode is correctly integrated.

Based on the Bernoulli kinematics two different cases have been analyzed, the standard case where all the gravity load is applied to the wall (100% model) and another case where only 20% of the gravity load is acting (20% model). Based on the Timoshenko kinematics, other two cases have been considered, differing for the $V - \gamma$ relation (NLT and NLTD models). A total of four different models were analyzed with the fiber elements discretization.

In the previous section, the wall response was simulated via a spread plasticity model, describing the material properties in terms of sectional variables, namely the moment-curvature relation. The overall performance of the spread plasticity model was satisfactory, partly based on an accurate set-up of the numerical model. In the moment-curvature relation a reduced stiffness $0.35EI_g$ was chosen, following literature data, and the yielding moment $M_y$ corresponding to the minimum value of the compressive axial force recorded during the tests was assumed. However, this model simulated poorly the response peak in RUN4. In a previous work (Martinelli and Mulas, 2006), the poor performance of the PQ model in correspondence of the peak of the RUN4, was attributed to the absence of the M-N interaction in the element formulation. However, the 100% fiber model, fully accounting for this phenomenon, shows an even worst performance both in terms of extreme values and in terms of time histories, as will be discussed in this section.

The 20% model was derived to simulate, in accordance with the results obtained in the previous section and in Martinelli and Mulas (2006), a reduced flexural stiffness, obtained in a fictitious way through the reduction of the gravity loads. Even though the initial frequencies of 100% and 20% models are the same, the model 20% experiences a higher reduction of stiffness during the time history, due to the reduced amount of gravity loads.

In a similar way to that done for the spread plasticity model, the comparison between experimental and numerical results is here performed in terms of the global response parameters, namely the top displacement, the top horizontal acceleration, the base bending moment, the base shear force. The formulation of the fiber model allows also for the evaluation of the dynamic variation of the axial force at the base cross-section. The extreme values of the parameters in study for the four modeling approaches are listed in Table 4.6, where the results of RUN2 have been omitted.
since the structure remains practically in the linear range. The results in terms of percentage errors of these models for the afore mentioned parameters are shown in Figure 4.23. The presentation of the results will highlight the rationale for the modeling choices.

<table>
<thead>
<tr>
<th>Model</th>
<th>RUN</th>
<th>$u_{top}$ [mm]</th>
<th>$a_{top}$ [g]</th>
<th>$V_{base}$ [kN]</th>
<th>$M_{base}$ [kNm]</th>
<th>$N^{-}_{base}$ [kN]</th>
<th>$N^{+}_{base}$ [kN]</th>
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</thead>
<tbody>
<tr>
<td>100%</td>
<td>1</td>
<td>7.6</td>
<td>1.19</td>
<td>104.7</td>
<td>324.2</td>
<td>99.1</td>
<td>93.7</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.1</td>
<td>1.96</td>
<td>166.3</td>
<td>401.5</td>
<td>189.5</td>
<td>145.6</td>
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<td></td>
<td>4</td>
<td>4.7</td>
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<td>67.2</td>
<td>197.1</td>
<td>91.9</td>
<td>83.6</td>
</tr>
<tr>
<td>20%</td>
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<td>5.5</td>
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<td>75.7</td>
<td>229.1</td>
<td>92.3</td>
<td>93.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10.7</td>
<td>1.59</td>
<td>126.2</td>
<td>275.5</td>
<td>141.2</td>
<td>115.2</td>
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<td></td>
<td>4</td>
<td>10.4</td>
<td>0.97</td>
<td>91.2</td>
<td>284</td>
<td>129.8</td>
<td>95.6</td>
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<td>101.3</td>
<td>323.5</td>
<td>81.5</td>
<td>78.3</td>
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<td></td>
<td>3</td>
<td>10.5</td>
<td>1.97</td>
<td>133.5</td>
<td>391.4</td>
<td>159.4</td>
<td>165.8</td>
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<tr>
<td></td>
<td>4</td>
<td>3.4</td>
<td>0.62</td>
<td>58.9</td>
<td>170.0</td>
<td>30.3</td>
<td>30.4</td>
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<tr>
<td>NLTD</td>
<td>1</td>
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<td>305.5</td>
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<td>39.8</td>
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<td>3</td>
<td>9.8</td>
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<td>129.6</td>
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<td>4</td>
<td>3.3</td>
<td>0.61</td>
<td>57.8</td>
<td>167.8</td>
<td>24.3</td>
<td>18.7</td>
</tr>
</tbody>
</table>

- Compression
+ Tension

Table 4.6: Extreme values of response parameters.

The results predicted by the standard fiber model denote a poor performance of the 100% model. The displacements are overestimated for RUN1, and grossly underestimated for RUN3 and RUN4 (Figure 4.23a). The horizontal acceleration, the bending moment and the shear force are overestimated for RUN1 and RUN3 and underestimated for RUN4 (Figure 4.23b-d). The standard model overestimates the dynamic axial force for all the runs, both in tension and compression, showing the highest errors (Figure 4.23e, f).

The standard model has a bad match with the experimental results also in terms of the maximum envelope along the height of the wall. The envelopes of the maximum displacement and interstory drifts are shown in Figure 4.24, while the envelopes for the horizontal acceleration, shear force and bending moment are depicted in Figure 4.25. The envelope of the maximum axial force in tension and compression, where zero is referred to the static value, is presented in Figure 4.26. The black line represents the experimental results, the darker grey line represents the 100% model results and the lighter grey line represents the 20% model results; different
markers have been used for different input motions. The envelopes of the maximum values confirms the percentage error results, pointing out the bad performance of the standard fiber model. The reproduction of the dynamic axial force, both in tension and in compression, is particularly unsatisfactory.

To investigate the frequency content of the response, the time histories of the parameters in study are analyzed for the Runs 1, 3 and 4. The results for the top displacement, base shear force, base bending moment, top horizontal absolute acceleration and dynamic axial force are shown in Figures 4.27-4.31, respectively; black line denote experimental results, dashed and continuous grey lines denote 100% and 20% model results, respectively. As for the spread plasticity model, each figure contains four subplots, one for RUN1 (a) and RUN3 (b), and two for RUN4 (c-d). In every plot a time interval of 6 s is analyzed, except for the axial force, where a shorter time interval of 3 s is considered.

The results depicted in the Figures 4.27-4.31 confirm the results obtained in terms of extreme values, highlighting the different frequency content between the standard model and the experimental results. In particular, the large error in RUN4 is due to failing in the reproduction of the response peak taking place at around 120 s (in the complete time sequence of the four runs). An explanation for this poor performance can be found in the comparison, depicted in Figure 4.32, between the amplitude spectra of the top displacement for the experimental and numerical results, showing the overestimation of the structural stiffness for the standard 100% model during the load history. In these plots two time windows of 4 s each have been considered for runs 1, 3 and 4.

The response peak in RUN4 in terms of top displacement, base bending moment and base shear force (Figures 4.27-4.29c) is correctly reproduced by 20% model; however the subsequent response (Figures 4.27-4.29d) is affected by a significant error, presumably for an excessive stiffness degradation due to the former high peak. Figure 4.32 depicts the satisfactory result in terms of frequency content, that is believed to be the reason for the successful performance of this model. Finally, the Figure 4.23 provides an immediate and global idea about the better performance of the 20% model with respect to the traditional fiber model. The match between experimental and numerical results increases dramatically, in terms of both extreme values (Table 4.6 and Figures 4.24-4.25) and time histories (Figures 4.27-4.30).

In fact, it must be noticed that the reduction of the gravity load implies also a strength reduction: due to the M-N interaction, the maximum bending moment that each cross-section can transmit decreases. This, in turn, implies a reduction of the maximum shear, due to the statically determinate scheme of the wall.

To analyze the wall in a more realistic way, the shear deformability in the non-linear range was included in the NLT model. The results, however, are not satisfactory; the percentage errors (Figure 4.23) as well as the extreme values (Table
Figure 4.23: Percentage errors on maximum values: (a) top displacement; (b) top horizontal acceleration; (c) base shear force; (d) base bending moment; (e) base compression axial force and (f) base tension axial force.
4.3 NUMERICAL RESULTS

4.6) are very close to those predicted by the 100% model. For this reason, only a few time histories regarding the NLT model have been presented here. Figure 4.33 shows: (a) the base shear force for RUN1, (b) the base bending moment for RUN3 and (c) the top displacement for RUN4. As for the 100% model, the NLT model shows a heavy overestimation for the base bending moment and shear force in runs 1 and 3 (Figure 4.33a, b) while underestimates the response peak in RUN4 (Figure 4.33c). A reason for this can be the high value of the shear force $V_0$ marking the transition from the first to the second branch of the bilinear $V - \gamma$ relation approximating the FE results. A comparison with the experimental results shows, in fact, that this value is never reached in the tests. For this reason the model NLTD, having a bilinear shear force-deformation relation with the same stiffnesses $K_0$ and $K_1$ but with a value $V_0 = 50$ kN, equal to $1/3$ of the previous one was finally derived. In
Figure 4.25: Envelope of experimental and numerical results for 100% and 20% models: (a) horizontal floor acceleration; (b) shear force and (c) overturning moment.
4.3 NUMERICAL RESULTS

Figure 4.26: Envelope of experimental and numerical results for 100% and 20% models: dynamic axial force.

spite of the fact that this model describes a higher shear deterioration, the results are comparable to that obtained with the model NLT, both in terms of extreme values (Table 4.6), of percentage errors (Figure 4.23) and of time histories (Figure 4.33).
Figure 4.27: Comparison of experimental and numerical values for the top displacement: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.28: Comparison of experimental and numerical values for the shear force at wall base: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.29: Comparison of experimental and numerical values for the overturning moment at wall base: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.30: Comparison of experimental and numerical values for the top absolute horizontal acceleration: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.31: Comparison of experimental and numerical values for the dynamic axial force at wall base: (a) RUN1; (b) RUN3; (c-d) RUN4.
Figure 4.32: Comparison between experimental and numerical frequency spectrum for (a,b) RUN1; (c,d) RUN3; (e,f) RUN4.
Figure 4.33: Time histories: (a) RUN1, base shear; (b) RUN3, base bending moment and (c) RUN4, top displacement.
Chapter 5

Slender shear wall (NEES-UCSD)

Part of the results herein presented were performed within the context of the “Seven Story Building Slice Earthquake Blind Prediction Contest” that served as UC Berkeley entry (Martinelli and Filippou, 2007). The contest was organized by the School of Engineering at the University of California at San Diego (UCSD), the Portland Cement Association, and the NEES Consortium. The competition was open to teams from the structural engineering community, the academic and research community, and the undergraduate engineering student community who were invited to perform a “blind” numerical simulation of the structure. The relevant material and geometric properties of the wall and the input base excitation records were provided to the participants, but the experimental results were kept secret, thereby challenging each team to produce the best possible numerical predictions of the structure’s non-linear response within the time frame of approximately 2 months.

5.1 Description of the specimen

5.1.1 Geometry and mass

The test specimen is a 7 story full scale wall structure (Figure 5.1). The test was conducted at the NEES-UCSD Large High-Performance Outdoor Unidirectional Shake Table. The structure was designed with a displacement-based capacity approach for a site in Los Angeles. This resulted in design lateral forces that are considerably smaller than those currently specified in the building codes used in the United States. The total height of the specimen is 19.96 m and the total weight is 226 tons. The building is the tallest structure ever tested on a shaking table. The specimen is made up of a web wall 3.65 m wide and two transverse structural elements: a flange wall 4.88 m wide and a precast segmental pier column. The thickness of the web wall is 203 mm at the first and seventh floor and 152 mm at all other floors.
The thickness of the flange wall is 203 mm at the first floor and 152 mm at all other floors. Both the web and the flange wall are fixed at the base. The web wall provides lateral resistance in the direction of loading, while the two other structural elements provide transverse and torsional resistance to the test structure.

Figure 5.1: View of the test structure and sketch of the system elevation (U.S. Customary Units).

A 3.65 m by 8.13 m simple supported slab on gravity columns is present at each level. The flange wall is connected to the slab with a pin-pin connection. The segmental pier column is connected to the slab through a pin-pin horizontal steel truss. The pier was pinned at the base in the east-west (E-W) direction and fixed in the north-south (N-S) direction (Figure 5.2). For the gravity columns high strength steel pin-pin rods grouted in 102 mm pipes were used. A foundation and floor plan view of the system as well as the direction of the input ground motion are shown in Figure 5.2 while the mass of each element is given in Table 5.1. A particular of the slotted connection is depicted in Figure 5.3.
Figure 5.2: Foundation and floor plan view of the specimen (U.S. Customary Units).
<table>
<thead>
<tr>
<th>Structural element</th>
<th>Mass of each element [kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation</td>
<td>21448</td>
</tr>
<tr>
<td>Slabs</td>
<td>110185</td>
</tr>
<tr>
<td>Web Wall</td>
<td>26073</td>
</tr>
<tr>
<td>Flange Wall</td>
<td>33237</td>
</tr>
<tr>
<td>Precast Pier Column + Foundation</td>
<td>30155</td>
</tr>
<tr>
<td>Gravity Columns + Foundation</td>
<td>4716</td>
</tr>
<tr>
<td>Braces</td>
<td>181</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>225995</strong></td>
</tr>
</tbody>
</table>

Table 5.1: Mass balance.

![Flange wall and slotted connection](image)

Figure 5.3: Zoom of the slotted connection (U.S. Customary Units).

### 5.1.2 Materials

Tunnel steel forms were used for the construction of the walls and slabs. Concrete with specified compressive strength of 28 MPa and A615 grade 60 steel were used. The concrete material characteristics were established with cylinders tests whose results are given in Table 5.2, where $\varepsilon_{cu}$ is the strain corresponding to the maximum stress. The construction sequence included casting a level of the web and flange walls as well as the slab at a time. The segmental pier column was precast in three pieces and assembled afterwards using mortar bed joint and post tensioning. The concrete cast sequence is shown in Figure 5.4. For the concrete placements c4, c6, c8, c10, c12, c14 and 16 measured data are not available. These concrete pour placements correspond to the 4 in starting curb of floors 2, 3, 4, 5, 6, 7 of the flange wall. Tensile test were performed for the different reinforcing steel bars and the results are reported as the average of three specimens. The results are shown in Table 5.3 where $f_y$ and $\varepsilon_y$ are the yield stress and strain, respectively, $f_{su}$ denotes the maximum stress, and $\varepsilon_{su}$ is the strain corresponding to $f_{su}$. 
Concrete placement & $f'_c$ [MPa] & $E_c$ [MPa] & $\varepsilon_{cu}$
\hline
c2 & 54.26 & 23091 & -0.00281 \\
c3 & 37.44 & 24469 & -0.00269 \\
c5 & 39.30 & 26000 & -0.00229 \\
c7 & 42.13 & 34839 & -0.00214 \\
c9 & 41.58 & 30199 & -0.00236 \\
c11 & 39.99 & 28896 & -0.00225 \\
c13 & 39.85 & 32136 & -0.00233 \\
c15 & 43.09 & 33536 & -0.00210 \\
c17 & 38.75 & 28917 & -0.00234 \\
c18 & 37.58 & 30323 & -0.00220 \\
\hline

Table 5.2: Characteristics of the concrete cylinders.

The reinforcement details of the web and flange wall are shown in Figure 5.5. The reinforcement of the web wall in the first and seventh floor includes two layers of vertical reinforcement (8#5) and confinement mesh (#3@4 in Baugrid) at each end. A single layer of reinforcement (13#4@10 in) exists between them. For the other levels, a single layer of vertical reinforcement with 4#7 at each end and 11#4@10 in exists in between. No confinement reinforcement is provided in levels two to six. A single layer of horizontal reinforcement (#4@8 in) was also provided at all levels. The reinforcement longitudinal ratio is $\rho_l = 0.66\%$ in levels one and seven and $\rho_l = 0.81\%$ in levels two to six. It should be noted that additional transverse reinforcement was used at the lap splice location in levels two to six. A capacity based design was followed for the design of the lap splice region. The total steel reinforcement area of the lap splice is equal to the reinforcement of the first level and slightly exceeds the total reinforcement area of the second level. The reinforcing steel of the first level is anchored in the foundation and stops at the top of the first level slab. For the other levels the reinforcement that protrudes from the bottom level continues and is lap spliced with the reinforcement of the top level.

5.1.3 Loading program

The experimental program investigates the response of the web cantilever wall to different levels of excitation. The input motion consists of 4 accelerograms applied in the direction parallel to the web wall (E-W direction) denoted with EQ1, EQ2, EQ3 and EQ4, respectively. The first two records are the longitudinal and transverse component of the 1971 San Fernando earthquake (California) recorded at Van Nuys station. The third record is the longitudinal component of the Northridge 1994 earthquake.
Bar size | Bar name | $f_y$ [MPa] | $f_{su}$ [MPa] | $\varepsilon_y$ | $\varepsilon_{su}$
--- | --- | --- | --- | --- | ---
#4 | b1 | 449.7 | 750.9 | 0.0054 | 0.1009
#4 | b2 | 435.3 | 710.5 | 0.0074 | 0.1096
#5 | b3 | 454.0 | 696.4 | 0.0078 | 0.1128
#7 | b4 | 476.4 | 765.8 | 0.0079 | 0.1117
#7 | b5 | 452.6 | 775.5 | 0.0070 | 0.1053
#6 | b6 | 493.4 | 784.1 | 0.0052 | 0.1046
#6 | b7 | 457.6 | 720.0 | 0.0054 | 0.0836
#3 | b8 | 455.2 | 724.9 | 0.0086 | 0.1169
- | b9 | 438.0 | 704.2 | 0.0087 | 0.1098
- | b10 | 443.7 | 725.4 | - | 0.0911
- | b11 | 489.1 | 749.7 | 0.0053 | 0.0800

Table 5.3: Steel bar characteristics.

An earthquake (California) recorded at the Oxnard Boulevard station in Woodland Hill. The last record is the Sylmar Olive View Medical Center 360° component from the 1994 Northridge earthquake. The ground acceleration time histories are shown on Figure 5.6. The test program included also low intensity white noise in between earthquake tests. The response spectra of the first three earthquake records is slightly higher than the site response spectra for 50% probability of exceedance in 50 years. The large intensity earthquake record has a spectral acceleration in the period range of interest above the site response spectra for 10% probability of exceedance in 50 years. The acceleration and displacement response spectra of the input motions (5% damping) can be seen in Figure 5.7. For the white noise tests 2%g, 3%g and 5%g root mean square (RMS) motions were used. Ambient and white noise vibrations measured using both accelerometers and linear variable differential transformers (LVDTs) were used for system and damage identification before and after every earthquake test, helping to establish the different levels of damage after each earthquake record. For the high intensity earthquake significant damage was expected at the base of the structure.

5.1.4 Specimen instrumentation

A dense network of instrumentation was deployed to measure the dynamic response of the structure. The instrumentation includes internal as well as external instrumentation: 139 accelerometers, 58 displacement transducers, 28 string potentiometers, 314 strain gages and 23 pressure transducers were used. For measuring displacements a video grammetric system and an array of 50 Hz, 3 mm resolution, GPS
devices were deployed. The total number of channels, including the sensors used for monitoring the hydraulic components of the shaking table and the response of the reaction block and the surrounding soil, exceeds 600.

In more detail, at each of the two lower levels eight pairs of displacement transducers were used at the ends of the wall for obtaining the local strains and curvature profile. The network of displacement transducers was denser at the two lower levels because of the expected inelastic response in this region. In addition, two diagonal string potentiometers were placed at every side of the wall for measuring the shear deformations. In levels three to seven two pairs of strain pots were used at the ends of the web wall for measuring the average strains of these regions, which are expected to remain in the elastic range. On the post tensioned segmental pier, 18 additional displacement transducers were placed in different parts. The instrumented parts of the segmental pier were the connecting regions of the segmental pieces as well as the connection regions with the horizontal truss.

A dense network of accelerometers is also deployed. Four horizontal, two transverse and two vertical accelerometers were placed on top of every slab. Additionally, nine vertical accelerometers were placed at the top of the slab at every other
Figure 5.5: Test structure reinforcement details: (a) first level; (b) levels 2-6 (U.S. Customary Units).
5.1 DESCRIPTION OF THE SPECIMEN

Figure 5.6: Acceleration time histories for EQ1, EQ2, EQ3 and EQ4 respectively.

Figure 5.7: (a) Acceleration and (b) displacement response spectra (5% damping).
floor. Two horizontal, one transverse and two vertical accelerometers were placed at the top of the post tensioned pier and the flange wall. An additional horizontal accelerometer was placed at mid height of every level of the web wall. Finally sixteen accelerometers were placed at the foundation level and at the platen. The layout of the horizontal and transverse accelerometers at the top of every slab can be seen in Figure 5.8.

![Figure 5.8: Longitudinal and transverse accelerometer layout.](image)

5.1.5 Experimental results

Table 5.4 contains a summary of the envelope of the experimental values that will be compared with the numerical results: shear force and overturning moment at the base and horizontal displacement and absolute horizontal acceleration at the top. The internal forces presented in Table 5.4 were computed from the recorded horizontal absolute accelerations and from the estimate of the masses of all elements of the structure. The accelerations was obtained from accelerometer Hi-1 placed on top of each slab (Figure 5.8).

5.2 Numerical model of the wall

The modeling approach in the study of this wall uses a non-linear fiber beam-column element for each story. The numerical model for the shaking table test
Table 5.4: Envelope of the experimental response parameters.

<table>
<thead>
<tr>
<th>EQ</th>
<th>$u_{top}$ [mm]</th>
<th>$a_{top}$ [g]</th>
<th>$V_{base}$ [kN]</th>
<th>$M_{base}$ [kNm]</th>
</tr>
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<tbody>
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<td>146</td>
<td>0.593</td>
<td>628.2</td>
<td>8093.1</td>
</tr>
<tr>
<td>3</td>
<td>160</td>
<td>0.728</td>
<td>703.9</td>
<td>8490.0</td>
</tr>
<tr>
<td>4</td>
<td>395</td>
<td>1.078</td>
<td>1184.7</td>
<td>11839.4</td>
</tr>
</tbody>
</table>

Table 5.5: Concrete material properties assumed in the analyses.

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>$f'_c$ [MPa]</th>
<th>$\varepsilon_0 \times 10^{-3}$</th>
<th>$\varepsilon_u \times 10^{-3}$</th>
<th>$k$</th>
<th>$Z_m$</th>
<th>$f_{ct}$ [MPa]</th>
<th>$E_{ct}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined</td>
<td>-37.9</td>
<td>-2.70</td>
<td>-5.09</td>
<td>1</td>
<td>376</td>
<td>3.8</td>
<td>2800</td>
</tr>
<tr>
<td>Confined</td>
<td>-45.8</td>
<td>-3.26</td>
<td>-30.3</td>
<td>1.21</td>
<td>30</td>
<td>3.8</td>
<td>2800</td>
</tr>
</tbody>
</table>

The specimen was prepared with OpenSees, the finite element analysis platform (Mazzoni et al., 2002).

A two-dimensional lumped mass model was created for the seven story test specimen. The layout of the nodes and elements for the numerical model is shown in Figure 5.9. The 3D effects of the compatibility deformation between the web wall, the slab and the flange wall were neglected in this study. The web and flange walls were modeled with one non-linear fiber beam-column element at each story. In the model, two different zones of concrete material response were identified in the web wall for levels 1 and 7: confined concrete at the outer layers of the cross-section, and unconfined concrete in the interior. By contrast, the concrete at levels 2-6 of the web wall and in the flange walls was modeled as unconfined. The web wall cross-section at level 1 and 7 was subdivided into 56 concrete layers. The outermost six layers have one third the area of the remaining layers. The web wall cross-section from level 2 to level 6 was subdivided into 44 concrete layers. The confined and unconfined zones in the web wall are shown in Figure 5.9. Table 5.6 summarizes the material properties used in the analyses for these two concrete zones. For the unconfined concrete, the experimental value of the zone $c_3$ (Figure 5.4) was taken as the reference value. Table 5.6 shows the material properties assumed for the steel reinforcement. For simplicity, only one average value was considered in this study.

The footings of the web and flange wall were represented with linear elastic beam elements because of their large strength on account of size and prestressing with longitudinal tendons. The cross-sectional properties of these linear elements were determined with the assumption of uncracked response based on the gross area and
a Young modulus of concrete of $E_c = 27500 \text{ MPa}$.

The sole purpose of the precast column element is the torsional stability of the specimen. The bracing between the precast column and the slab was modeled as a truss element that was designed to remain elastic during testing. The slotted connection between the flange wall and the web wall was represented with a truss element of axial stiffness $E_A = 550000 \text{ kN}$. This assumption neglects the small value of bending moment and shear force transferred by the slotted connection from the flange to the web wall. In order to connect the centerline of the web wall with the centerline of the precast element and that of the flange wall, rigid end off-sets were used in the numerical model between the web wall and the truss elements. Before starting with the non-linear analyses, a preliminary linear model was used to understand the influence of modeling assumptions on the first natural frequency. In this model linear elastic beam elements were used for modeling the walls and the post tensioned pier column. The sections were considered uncracked and the concrete modulus was taken as $E_c = 27500 \text{ MPa}$.

The first model of the present study considered only the web wall with lumped mass at each floor that accounted for the weight of the slab. This resulted in a first natural frequency of 2.29 Hz, as compared with that measured in the tests under ambient vibration, which was 1.91 Hz. A second model was then constructed to include the flange wall with the slotted connections between web and flange walls modeled as rigid links. This assumption resulted in a very stiff model with a first natural frequency of 4.17 Hz. After correcting the model to allow for axial force transfer only by the slotted connections, the first natural frequency of the model decreased to 2.06 Hz. A third model was then created to include the post tensioned pier. The first mode of this model has a natural frequency of 1.95 Hz as compared with the measured frequency of 1.91 Hz. Finally, the inclusion of the rotational inertia of the floor mass resulted in a slight reduction of the first natural frequency of the model to 1.94 Hz. This model was used for the non-linear response simulations.

With the modeling assumptions and material properties given before, the initial frequency of the model with the fiber beam-column elements representing the web wall was equal to 2.05 Hz in good agreement with the experimental value. Since the tensile strength is included in the material model, the structural model frequency corresponds to the uncracked section stiffness. The first three longitudinal (L) natural frequencies based on ambient vibration data from accelerometers from the un-

<table>
<thead>
<tr>
<th>$E_0$</th>
<th>$E_1$</th>
<th>$f_y$</th>
<th>$f_{su}$</th>
<th>$\varepsilon_y$</th>
<th>$\varepsilon_{su}$</th>
<th>$b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>$\times 10^{-3}$</td>
<td>$\times 10^{-3}$</td>
<td></td>
</tr>
<tr>
<td>200000</td>
<td>3100</td>
<td>450</td>
<td>750</td>
<td>2.24</td>
<td>100</td>
<td>0.015</td>
</tr>
</tbody>
</table>

Table 5.6: Steel material properties assumed in the analyses.
5.3 Numerical results

5.3.1 Static non-linear analysis

The pushover analysis has several advantages. The procedure is simple, does not rely on an estimate of the site-specific ground motion and can identify the critical regions where large deformations are expected (Elnashai, 2001).

The OpenSees platform program was used to conduct the pushover analysis for damaged state are compared with the frequencies of the non-linear model in Table 5.7. The first experimentally measured coupled longitudinal-torsional (L-T) frequency is also reported. These experimental data were available only after the competition (Moaveni et al., 2006).

Figure 5.9: Numerical model of the 7 story building.
the tested structure. A pattern of horizontal forces is defined, acting at each level of the structure, possibly of different intensity and scaled by the same factor. As the scaling factor increases from zero the displacement of a characteristic point on the structure (termed “control node”) is observed and related to the shear at the base of the structure. A key point in this type of analysis is the necessity to define the horizontal forces pattern. Since the shape of this distribution has arbitrary nature and should represent the inertia forces acting on the structure during the seismic excitation, generally the seismic codes prescribe the construction of the pushover curve for several different forces patterns. It is generally accepted that the “true” behavior will be bounded by a distribution representing the behavior of the elastic structure and one representing the behavior when the structure is well into the inelastic range. It is allowed to represent the behavior of the elastic structure through

---

**Figure 5.10:** Confined and unconfined concrete zones in the web wall.
### 5.3 Numerical Results

<table>
<thead>
<tr>
<th>non-linear model</th>
<th>Measured frequencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>1(^{st}) L-mode</td>
<td>2.05 Hz</td>
</tr>
<tr>
<td>1(^{st}) L-T mode</td>
<td>-</td>
</tr>
<tr>
<td>2(^{nd}) L-mode</td>
<td>12.2</td>
</tr>
<tr>
<td>3(^{rd}) L-mode</td>
<td>19.8</td>
</tr>
</tbody>
</table>

Table 5.7: Measured and numerical natural frequencies.

a distribution of horizontal forces proportional to inertial forces when the structure responds only in the first mode; moreover, this distribution is further simplified becoming triangular. The behavior of the structure wall into the inelastic range is described by a constant distribution of horizontal forces. In the application of this procedure, both the triangular and constant distributions of horizontal forces were used in computing the pushover curve, and the control node was assumed at the centre of the top of the web wall. In the static and dynamic analyses it was assumed that the web wall was loaded by self weight and by half of the weight of the slabs (the other half of the load of the slab was attributed to the elastic columns).

**Force-displacement curves**

The numerical relation base shear vs top displacement is depicted in Figure 5.11 together with the experimental values recorded during the four input ground motions. The information presented in Figure 5.11a-c reveals that the structure remained in the elastic range under the first ground motion, whereas showed a light inelastic behavior in the second and third input ground motions. A large deformation was measured during the last input motion (Figure 5.11d). The maximum base shear was 1185 kN at 250 mm top floor displacement, which represents 1.2% drift. The pushover plot shown in Figure 5.11d does not envelope the experimental maximum data generated by last input motion. This can be explained in the following manner: the base shear force in the tests was not measured directly, but calculated using the story masses and the measured accelerations; on the other hand, the value of base shear in the OpenSees model, for the static analysis, was computed by summing up the support reactions at each level along the height of the wall. In the earthquake base excitation case, the general form of the equation of dynamic equilibrium is:

\[
[M] \cdot \dot{u} + [C] \cdot \ddot{u} + \{R(u)\} = - [M] \cdot \{1\} \ddot{u}_g \quad (5.1)
\]

where \(u\) is the relative displacement between the wall and the shaking table and \(\{R(u)\}\) is the internal resting force vector. From the Eq. 5.1 \(\{R(u)\}\) should be
equal to $[M] \cdot \{\ddot{u} + \ddot{u}_g\} + [C] \cdot \dot{u}$ and not only equal to $[M] \cdot \{\ddot{u} + \ddot{u}_g\}$.

Figure 5.11: Pushover curves vs experimental results: (a) EQ1; (b) EQ2; (c) EQ3; and (d) EQ4.

Thus, static analysis results for base shear vs top displacement can not directly be compared to the dynamic test results for the product of floor mass and floor acceleration. Viscous damping forces should be subtracted from the product of floor masses and acceleration, to obtain a comparable results. Damping effects are present in structure as that here studied, where the presence of bracing or slotted connections at several floor can act as dissipating devices. In addition, other factors can have contributed to the fact that numerical static result underestimates the maximum dynamic experimental data (Mulas et al., 2006b): (a) the absence of the rocking effects, (b) the absence of 3D effects, (c) the effect of the material cyclic degradation and (d) the effect of the higher modes.
5.3 NUMERICAL RESULTS

Moment-curvature curves

The moment-curvatures curves obtained with OpenSees non-linear model can help to provide a good insight in the behavior of the wall. The curves for the cross-sections of the web wall at the different levels, obtained considering the axial force produced by gravitational loads and the material properties listed in the Paragraph 2.2, are depicted in the Figure 5.12. The moment-curvature relationships were determined using a standard section analysis of the web wall (assuming plane sections remain plane), with the concrete and steel models as shown in Figure 3.7 and 3.9.

Let consider the moment-curvature diagram at the base: initially, the wall was uncracked and the moment-curvature diagram for this stage was linear (moment up to 2500 kNm). When the stress at the extremity of the wall reached the tensile strength of the concrete, cracking occurred. As a result, less of the concrete section was effective in resisting moments and the stiffness of the wall decreased (from 2500 to around 4000 kNm). Around 4000 kNm the outermost bar reached the yield point; once yielding had occurred, the curvatures increased rapidly with very little increase in moment.

5.3.2 Dynamic analysis

The OpenSees platform was used for the non-linear time history simulations by imposing the acceleration measured by the shaking table accelerometer in the E-W direction at the base of the footing of the web and flange walls. The simulation to the four recorded motions was performed with a single continuous sequence of concatenated acceleration records from EQ1 through EQ4.

All non-linear time history analyses adopted the Newmark time integration method of constant acceleration, with a time step equal to $\Delta t = 0.01$ s. Rayleigh damping was assumed with the damping matrix proportional to the mass and initial stiffness matrix. The constants were calibrated to give a damping ratio of 1.5% for the first two modes in the plane of excitation. The same damping matrix was used for all input ground motions.

Global response

The assessment of the quality of the numerical results can be based on the envelope of maximum values for floor lateral displacement, interstory drift, residual lateral displacement, horizontal floor acceleration, story shear force and overturning moment, which were required for the competition. These quantities are depicted in Figure 5.13 and 5.14, with a black line representing the experimental values and a grey line representing the numerical results; different markers have been used for each input motion. The agreement between measured and predicted displacement
Figure 5.12: Moment-curvature relation for web wall cross-section from base level to level 5.
and interstory drift values is excellent. The experimental shear force and overturning moment were evaluated at the centerline of the web wall and took into account the inertial effects of all elements of the structure. The numerical shear values in Figure 5.14 are obtained from the deformations. The observed underestimation of interstory shear values and overturning moment may be due to the fact that the model does not include the effect of the slab and the slotted connection, or it may be due to the different way of determining the interstory shear between analysis and experiment. The agreement between measured and predicted floor acceleration values is satisfactory, but not as good as for the interstory drift, particularly in the lower floors of the test specimen for input motion EQ4. The increasing deformation caused by the increasing intensity of the input motions is apparent in the envelope values of Figure 5.13 and 5.14. Permanent inelastic displacements at the end of the last input motion were observed in the simulations and these were in good agreement with the measured values (Figure 5.13c).

In addition to the envelope values the response time history for top displacements, internal forces and top horizontal accelerations offers an insight into the damage evolution of the numerical model during the four input ground motions. Figure 5.15 compares the measured response with the numerical time history of the top displacement of the web shear wall relative to the base. The agreement between analysis and experiment is quite remarkable for the strong motion, as is the case for the other three motions. The time histories of the overall system base shear and overturning moment for all four input ground motions are depicted in Figures 5.16 and 5.17. Again, the numerical model underestimates the maximum measured shear value especially during the last input ground motion (Figure 5.16d). The agreement of the numerical and experimental results for the last motion is especially noteworthy when one accounts for the specimen damage incurred during the preceding input motions. The time histories for top floor absolute horizontal acceleration are shown in Figure 5.18. There is a fairly good match between the experimental and numerical top horizontal acceleration during the four input ground motions.

For a more accurate assessment of the ability of the proposed model to track the damage evolution of the test specimen during the four input motions of increasing intensity, the frequency spectrum of the displacement time history of the model is compared with the spectrum of the measured response. Figure 5.19 shows the frequency spectrum for a 6 s window at the beginning and at the end of each input motion. The spectrum at the beginning of each motion is represented with a dotted line, while that at the end of each motion is represented with a solid line. Each frequency spectrum is normalized to the corresponding maximum value for the input motion. It is worth noting that the fundamental frequency of the model and the test specimen changed from 1.9 Hz (fundamental period of 0.52 s) at the beginning of the first input motion to 0.67 Hz (1.49 s) at the end of the last input motion. Thus,
Figure 5.13: Measured and numerical envelopes: (a) displacement; (b) interstory drift and (c) residual displacement.
5.3 NUMERICAL RESULTS

Figure 5.14: Measured and numerical envelopes: (a) overturning moment; (b) shear force and (c) floor acceleration.
Figure 5.15: Comparison of experimental and numerical values for the top displacement during the four input motions.
Figure 5.16: Comparison of experimental and numerical values for the shear force at wall base during the four input motions.
Figure 5.17: Comparison of experimental and numerical values for the overturning moment at wall base during the four input motions.
Figure 5.18: Comparison of experimental and numerical values for the top absolute horizontal acceleration during the four input motions.
Local response

Moment-curvature responses for the web wall at base level obtained from the numerical model are depicted in Figure 5.20 for all four input ground motions. For these plots, as well as for those presented in the previous section, the bending moment was derived from displacements. Figure 5.20 points out the damage progress and as the non-linearity becomes pronounced after the second input motion. Figure 5.21 shows the moment-curvature response for 6 levels, from wall base level to level 5. From these Figures it is clearly visible that the maximum damage is localized at the wall base level. For the first three signals the damage is concentrated at the wall base level, with the other levels stay in the elastic range; during the last input ground motion the inelastic behavior spreads to the first level.

The final point of significant interest in performance-based seismic evaluation regards the determination of local damage in the concrete and in the reinforcing steel. The cyclic loops of the outermost confined concrete fibers at the wall base are depicted in Figure 5.22. Fiber “A” and “B” indicate the outermost fibers in east (close to the flange wall) and west (close to the precast pier column) direction, respectively. The maximum strain value from the numerical model agrees well with the experimental result. The Sylmar record from the 1994 Northridge earthquake (EQ4) caused an important damage to the structure with cracking at the wall extremities and a small permanent displacement (about 13 mm), confirming the presence of residual cracks and yielding of the reinforcement bars. Figure 5.23 shows the web wall corner before and during the last input motion pointing out the crack pattern during the strongest input motion (Figure 5.23b).

Figure 5.24 shows the envelope of steel strains in an outer reinforcing layer along the height of the web wall. The agreement with the single experimental value near the base of the wall is quite good. The hysteretic behavior of two outer bars (ST2 and ST18) is also considered. Stress-strain response for the first three levels are plotted in Figure 5.25. As it was expected, the “critical” region is located on the lower level. The maximum strain values of the model agree well with isolated measurements of steel and concrete strains at the base of the wall. The model fails to capture the lap splice failure of the specimen because perfect bond between concrete and steel is assumed, neglecting the effect of bond-slip.
5.3 NUMERICAL RESULTS

Figure 5.19: Evolution of the frequency spectrum during the four input motions: (a) measured; (b) numerical.
Figure 5.20: Numerical moment-curvature response of the web wall at the base level for the input motions from (a) EQ1 to (d) EQ4.
5.3 NUMERICAL RESULTS

Figure 5.21: Numerical moment-curvature response of the web wall from base level to fifth level (a-f) for the four input motions.
Figure 5.22: Stress-strain relationship for concrete fibers at wall base level.

Figure 5.23: Web wall corner view at base level: (a) before and (b) during EQ4.
Figure 5.24: Maximum strain envelope for steel bar ST2.
**Figure 5.25:** Stress-strain relationship for steel bars ST2 and ST18.
Chapter 6

Conclusions

6.1 Summary of the results

In the present study, two different reinforced concrete shear walls subjected on a shaking table to a sequence of input ground motions have been studied. The efficiency of the structural models in the non-linear range, at the macro and meso scale, have been investigated through the comparison with the experimental results. The first wall, named CAMUS I, is a 1/3 scale mid-rise, lightly reinforced shear wall, designed according to the multi-fuse approach. The second wall, named NEES-UCSD, is a slender full-scale wall, designed with a displacement-based capacity approach. Both the shear walls are representative of a wide range of structures existing around the world. The main findings can be summarized as it follows. With reference to the CAMUS I wall:

- The simulation of the experimental response with the macro scale model has been tackled with a twofold strategy, based on an accurate set-up of the numerical model and on the modification of the hysteretic constitutive relation of the plastic hinges, to better account for the stiffness degradation due to shear effects in cyclic loading. Concerning the first issue, the selection of an appropriate initial stiffness has been a key point: due to the adoption of a bilinear skeleton curve, the initial stiffness needs to represent the secant behavior up to yielding. Another crucial point, in a model not accounting for the M-N interaction, is the determination of the level of axial force corresponding to the yielding moment. A large variation of dynamic axial force is experienced by the wall during the test. Hence the value of the yielding moment must be selected so to represent, on the average, the strength of the wall during the entire seismic excitation. This fact could explain the good fit of RUN3, where the strength is likely to be close to this average value, and is evidenced by the poor behavior of the model $C_s$. This model has a moment
at yielding \((M_y)\) corresponding to the value of the axial force due to the presence of static load. The input parameters \(K_{eff}\) (initial effective stiffness) and \(M_y\) should be chosen on the base of the average values expected during the seismic excitation.

- The performance of the different hysteretic models seems on the whole comparable, even though the results of RUN4 are simulated at best by the pinching model. RUN4 was applied to an already damaged structure: it can be concluded that the pinching model is able to capture both the damage induced by RUN3 and its effects on RUN4. In this respect, the choice of phenomenological laws allows to insert the effect of physical phenomena like the shear behavior.

- The performance of the macro model, also considering its simplicity, can be considered more than satisfactory. The insertion of the strength degradation as proposed for example by Ibarra et al. (2005), could improve the performance and fix the problem evidenced in the simulation of the response peak in RUN4. A few shortcomings of the numerical code have been evidenced by this work. Numerical problems arose when the stiffness of the wall decreases in a significant way, with plastic hinges spreading and forming on the two sides of the same floor. In this respect it could be helpful both to add inertia terms on rotational degrees of freedom and/or prevent the formation of one of the plastic hinges.

- The standard formulation of the flexibility-based fiber element used, regardless to the adoption of a Bernoulli or a Timoshenko kinematics, appears unable to reproduce the initial stiffness and the structural behavior. A good result is obtained by fictitiously reducing the gravity loads acting on the wall (20% model), and thus its flexural stiffness and strength, due to the level of the static axial load. It can be concluded that the structural response is characterized by a marked stiffness deterioration, and the numerical results can be explained assuming that a shear-flexure coupling has taken place during the seismic tests, impairing the flexural behavior. This phenomenon cannot be captured neither by the monotonic pushover analysis on the FE model nor by the shear modeling based on Timoshenko kinematics and uncoupled from the flexural behavior, as it was presented here.

- The reproduction of the response of the wall, due to the low reinforcement ratio, the particular loading sequence and the critical shear behavior, has been a challenging task. For this wall, the choice of the element has been a crucial issue, while the structural modeling resulted relatively simple.

With reference to the NEES-UCSD wall:
• The wall was subjected to four consecutive table motions with increasing maximum acceleration from 0.15 g to 0.93 g. This excitation history caused increasing damage in the structure up to pronounced non-linear hysteretic behavior and lap splice failure during the last input motion.

• For reinforced concrete walls of medium to high slenderness, undergoing primarily flexural response with insignificant shear effects, the Euler-Bernoulli fiber beam-column element is a good compromise between accuracy and computational efficiency. The ability of the proposed model to predict the measured earthquake response of a reinforced concrete shear wall specimen has been demonstrated by the good agreement of the blind prediction results with the measured data. This comparison covered the time histories of top displacements, top horizontal acceleration, base shear force, base overturning moment and the envelopes of floor displacement, interstory drift, floor acceleration, story shear and overturning moment over the height of the structure. The ability of the proposed model to track the damage evolution of the specimen was confirmed after the competition with the comparison of the frequency spectrum evolution during the four input motions. The maximum strain values of the model agreed well with isolated measurements of steel and concrete strains at the base of the wall. The model, for his formulation assuming perfect bond between concrete and steel, did not attempt to predict material failure and did not capture the lap splice failure of the specimen.

• In this wall, contrarily to what happens in CAMUS I wall, the hypotheses of structural modeling was quite important; in fact, the specimen is not a single statically determinate wall but statically indeterminate system composed of different elements.

A final comment can be drawn on the design philosophy that was adopted for the CAMUS I wall. Regarding the prediction of the structural behavior, the adoption of a design philosophy, as in the seismic code EC8, that prefigures a clear deformative mechanism (plastic hinge at the base), seems preferable with respect to the choice of a code as PS92 that allows for a damage distribution along the height of the wall. A few analyses, not presented here, concerned the response of the wall CAMUS III, which has the same geometry and masses of the wall CAMUS I, but was designed according to EC8. A greater predictability of the results was found, confirming the above statement. Moreover, the greater reinforcement ratio present in the CAMUS III wall, improves the flexural behavior of the wall and therefore reduces the moment-shear interaction effects very difficult to simulate.
6.2 Outlook

The results of this work have pointed out the necessity of several elaborations. The first necessity is to develop modeling instruments simple enough to be accessible to structural designers but yet accurate to simulate the moment-shear interaction. An improvement appears necessary both at the macro and at the meso scale of modeling. At the macro scale, shear effects have been introduced only on a phenomenological base, through a modification of the hysteretic moment-curvature relation for the plastic zones. A further step in this direction can be the implementation of a strength degradation that, tied to the value of the shear force, can produce a coupling between shear and flexural behavior. A different choice can be represented by the insertion of a translational spring, in place of the rotational springs describing the fixed-end rotations, able to describe the non linear behavior in shear of a finite part of the beam element. This approach could help in correctly recognizing the fact that shear in concrete does not involve a sectional behavior but the response of several cross-sections. At the meso-scale, the approach through the Timoshenko beam has not produced positive results. In fact, the uniaxial behavior of concrete in flexure is not affected by the insertion of the shear deformability: there is no rotation of principal stresses, that remains normal to the cross-section plane as the fiber direction, and no reduction in the strength of the concrete in compression, due to the transverse tension induced by shear. Both these phenomena are quite important in shear dominated elements, as the numerical analyses of the CAMUS I wall have shown. Maintaining unchanged the element formulation, a first modification could be inserted at the constitutive relation level, relating the concrete strength in compression to some indicator tied to the shear distortion. This modification could be inserted and tested on the OpenSees platform in a relatively simple way.

The second necessity is that to extend the modeling work here done also to structures with a marked 3D behavior, subjected to torsional effects if excited by earthquakes. This is the case of those class of structures in which the walls are tied or interact each others. Examples of these structures can be find in stairs or elevators. For this reason the author will participate to the first phase of the international Benchmark “SMART-08” (Seismic design and best-estimate Methods Assessment for Reinforced concrete buildings subjected to Torsion and non-linear effects), supported by Commissariat à l’Energie Atomique (CEA), Electricité de France (EDF) and AREVA-NP companies. The goal of the benchmark is the response prediction of a 1/4th scale model of a 3 storey building with torsional effects (representative to a nuclear building), that will be built and tested on CEA’s Azalée shaking table.
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