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Realistic simulation of reinforced concrete structural systems with combine of simplified and rigorous component model

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Abstract. This study presents the efficiency of simulating structural systems using a method that combines a simplified component model (SCM) and rigorous component model (RCM). To achieve a realistic simulation of structural systems, a numerical model must be adequately capturing the detailed behaviors of real systems at various scales. However, capturing all details represented within an entire structural system by very fine meshes is practically impossible due to technological limitations on computational engineering. Therefore, this research develops an approach to simulate large-scale structural systems that combines a simplified global model with multiple detailed component models adjusted to various scales. Each correlated multi-scale simulation model is linked to others using a multi-level hierarchical modeling simulation method. Simulations are performed using nonlinear finite element analysis. The proposed method is applied in an analysis of a simple reinforced concrete structure and the Reuipu Elementary School (an existing structure), with analysis results then compared to actual onsite observations. The proposed method obtained results very close to onsite observations, indicating the efficiency of the proposed model in simulating structural system behavior.

Keywords: structural engineering; realistic simulation; multi-level hierarchical modeling; finite element analysis; simplified component model; rigorous component model.

1. Introduction

Advances in computer technology and numerical methods today allow the simulation of complex problems previously addressable only through experimentation and theoretical modeling in engineering science. Sophisticated engineering systems based solely on computer simulation have already been designed and put into practice in various industries. Complex phenomena such as airplane crashes and car accidents can now be analyzed by computer simulation and programs are now able to represent the detailed behavior of structural systems. In the realm of structural engineering, predicting global response and damage variation to structures during a major earthquake points out a crucial application for computer simulation technology.

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In structural engineering, structural systems (e.g., bridges and buildings) are typically large-scale and are affected by multiple structural components and materials at many scales. Thus, in order to create a realistic simulation of a structural system, a model must be able to capture all detailed behaviors of the real system at various scales. Modeling the whole structural system in every detail with very fine meshes using high-performance computing represents one way to achieve such a simulation. However, resultant models are enormous and still difficult to process within the limitations of computational power currently available.

Many researchers have proposed structural simulations that reduce computational demands by combining a simplified global model with several detailed component models at various scales. They have developed simplified models that can generally achieve this objective for various structural components. One such example is the model for beam-column elements proposed by D'Ambrisi and Fillippou (1999).

A realistic simulation of structural systems achieved by combining a simplified component model (SCM) and a rigorous component model (RCM) is proposed and investigated in this paper. Critical components are modeled in detail and then integrated into a larger model, which is applied to simulate the complete structure. These correlated multi-scale simulation tasks are linked together through a multi-level modeling simulation method so that the resultant simulation of the entire system incorporated several detailed simulations of individual components. To evaluate the feasibility, the authors employed the proposed method in simulations of a simple reinforced concrete structure and a building on the campus of Reuipu Elementary School. Obtained results are compared with the results of actual experiments conducted onsite. Analyses are carried out using commercial finite element software such as ETABS (2005) and ABAQUS (2004).

2. Related research

Studies related to this research involve three different structural engineering problems: (1) structural modeling and analysis subjected to pushover loading, (2) detailed modeling and analysis of structural components such as columns and beam-column connections and (3) structural engineering via computer simulations can be one under the other. With regard to structural modeling and analysis, Martino *et al.* (2000) presented a nonlinear pushover analysis of reinforced concrete structure based on provisions in FEMA 273 related to the seismic rehabilitation of buildings. They further developed a pushover analysis tool using a finite element program capable of accomplishing these tasks. Akkar and Metin (2007) presented their evaluations of a nonlinear static procedure presented in FEMA 440 for reinforced concrete structures. Evaluations were based on peak single degree of freedom displacement, peak roof, and estimations of inter-story drift. Statistics describing lateral loading patterns that are used in pushover analysis to idealize the building systems affect the accuracy of nonlinear static investigation procedures. Mwafy and Elnashai (2001) presented a comparison of the results of nonlinear static pushover analysis and dynamic collapse analysis for reinforced concrete buildings. Chintanapakdee and Chopra (2004) presented the seismic response of vertically irregular frames using response history and modal pushover analysis. The modal pushover analysis procedure for estimating seismic demands can be extended to analyses of unsymmetric-plan buildings (Goel and Chopra 2005). Sung *et al.* (2005) and Tu *et al.* (2006) proposed a realistic pushover analysis model for reinforced concrete columns, shear walls, and brick walls using a determined plastic hinge based on failure modes often observed in experiments and earthquakes.

As critical parts of the structure subjected to lateral seismic loading, columns and beam-column joints play a determining role on structural failure. Many studies have proposed the detailed modeling and analysis of response and behavior for this component. Silva *et al.* (2001) proposed a failure surface and criterion in three-dimensional (3D) space for a reinforced concrete column subjected to biaxial bending and axial load, with the failure criterion governed by a combination parameter between strain-based failure criteria and the exponential coefficient using optimization techniques. Koksal (2006) proposed a failure criterion for confined concrete columns that modified Mander's theoretical single stress-strain model into a multi-parameter form of Drucker-Prager type-criterion. For the beam-column joint component, Lowes and Altoontash (2003) proposed a model that presented detailed response under reversed-cyclic loading, which represents the primary inelastic mechanisms determining joint behavior, i.e., failure of the joint core under shear loading and anchorage failure of beam and column longitudinal reinforcement embedded in the joint. An approach to modifying joint core shear stress-strain response proposed by Mitra and Lowes (2007) transfers joint shear through a confined concrete strut and simulates strength loss due to load history and joint damage following yielding in beam longitudinal reinforcement.

With regard to computer simulation for structural engineering applications, D'Ambrisi and Fillippou (1999) proposed an analytical method to simulate the nonlinear static and dynamic responses of reinforced concrete frames that isolates the basic mechanisms controlling the behavior of a reinforced concrete member into individual sub-elements that are connected in series to form the reinforced concrete member element. Kwan and Billington (2001) and Palermo and Vecchio (2007) simulated of the nonlinear behavior of reinforced concrete structures using finite element models published in commercial finite element codes. Further advancements have made feasible the analysis of arbitrary conditions, including reverse cyclic loading or earthquake-type loading. Strain-softening in compression and the Bauschinger effect in reinforced steel are now accurately considered improving the cyclic response. Modeling bond-slip, concrete shear distortion, and steel buckling are the other necessary topics for full failure prediction.

3. Proposed multi-level hierarchical modeling and simulation method

Finite element analyses are often carried out to simulate the behavior of a system as a whole. In such analysis, simplistic models are often used for structural components such as connections. While simplistic models can adequately simulate the global response of structural systems, they are inadequate to describe the detailed responses of individual components. The significant amount of analysis done on structural components has modeled such in a very fine mesh to assess detailed behavior. However, such models are considered in isolation, with no consideration given to the relationship between their behavior and that of the rest of the system. Therefore, there is a need to integrate these two levels of knowledge under a single platform. Such integration will permit a more realistic simulation of structural engineering systems. To achieve this, each structural component must be modeled twice at two different scales: (1) a simplified model of a macro/global system to obtain the forces applied by the rest of the system, and (2) a rigorous model to analyze each isolated component to obtain detailed behavior.

As two separate models represent the same structural component in a real system, synchronization through communication and coordination is required during the simulation process. In addition, a rigorous component model may include simplified models of other components in order to produce

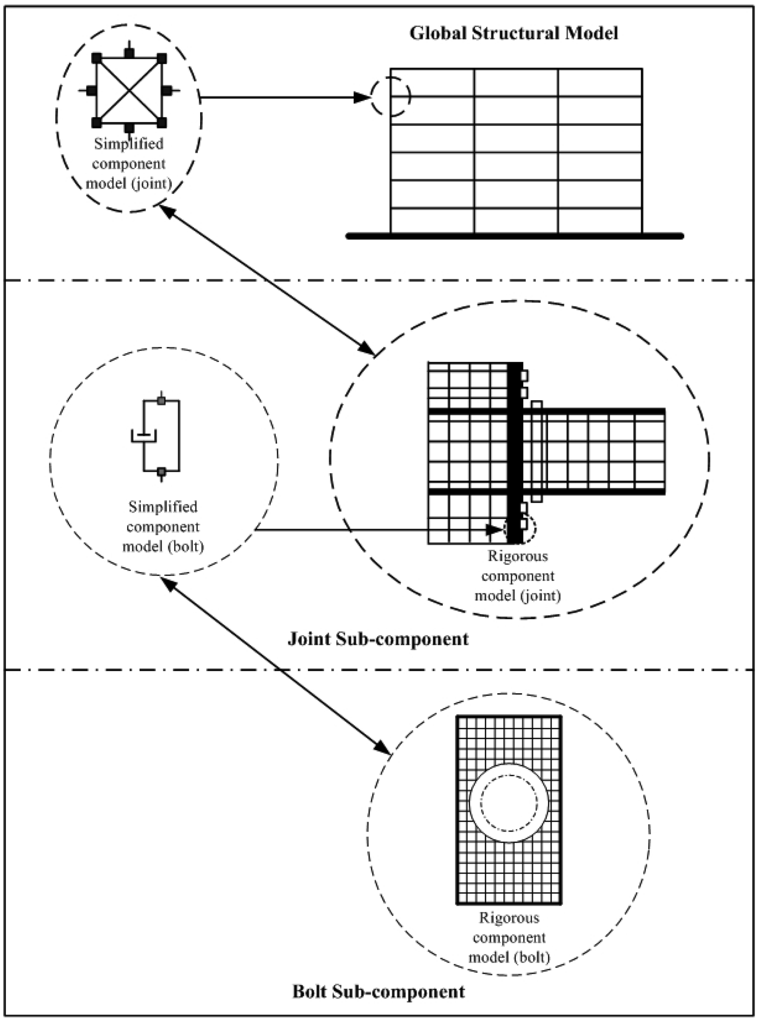


Fig. 1 Multi-level modeling and simulation method

a multi-level modeling architecture. This is shown conceptually in Fig. 1.

To derive a realistic simulation of structural behavior using the proposed multi-level modeling approach, the following three requirements must be fulfilled:

1. The simplified component model (SCM) must accurately describe the global behavior of the component modeled. Otherwise, resultant global actions upon it would be invalid.
2. The rigorous component model (RCM) should present the detailed behavior of the component modeled. For reinforced concrete structural components, it is necessary to develop separate material models for concrete and steel and to combine those models at the structure level through the use of different elements for each material.
3. The analysis algorithm must be able to handle the transition between these two distinct levels by translating a component's global behavior obtained from its macro-level model to action on its micro-level model. Moreover, localized effects on a component obtained from its micro-level model must be related back to the behavior of its macro-level model in the global system.

Thus, mechanisms are required to extract displacement and stress values in a SCM and then to translate these quantities into the known values obtained in the analysis of the corresponding RCM, and vice versa.

4. Finite element analysis for a realistic simulation of structural systems using SCM

According to ATC-40 (1996), the seismic resistance capability of a building can be evaluated using nonlinear static analysis. The main purpose of this method is to simulate the nonlinear response of a building experiencing a seismic lateral load that gradually increases in a specific direction. Nonlinear static analysis requires an analytical model that can simulate the overall nonlinear behavior of a structure. The global nonlinear behavior of a structure can be presented in a model which simulates each component of the structure with precision. Therefore, simplified models for each structural component type (e.g., beams, columns, beam-column connections, walls, bracings) should simulate their nonlinear responses accurately. A typical SCM is composed of simple nonlinear elements and bounds on element validity. Usually, each nonlinear model element describes a deformation mechanism that affects the overall nonlinear behavior of structural components. The behavior of a nonlinear element is described by one or a few performance curves or constitutive models, which may be established by codifying experimental results or theorems.

For nonlinear structural analysis, the software program ETABS offers a beam-column element with plastic hinges able to form various simplified components models (ETABS 2005). The program also provides four kinds of default plastic hinge properties to be used, including (1) an axial hinge (Default-P), (2) an interaction between axial and moment hinges (Default-PMM), (3) a moment hinge (Default-M) and (4) a shear hinge (Default-V). Unfortunately, these default hinge properties have been shown to be too conservative to analyze non-ductile reinforced concrete structures in Taiwan (Lin 2006).

Tu *et al.* (2006) proposed a solution to the unique hinge characterization. They assumed that the pushover curve for a reinforced concrete column has four critical points, namely cracking point, yielding point, ultimate point, and failure point. They further assumed that reinforced concrete column deformation points consist of three substantive components, namely flexural deformation, shear deformation, and bond slippage in the longitudinal reinforcement.

Moreover, moment and shear capacities should be compared at every point, smaller (controlled) capacity should be designated as the lateral load, and deformation should be calculated at every point as induced by controlled capacities. By introducing unique plastic hinge properties that allow the failure mode to be determined for each member, the pushover analysis can be performed using ETABS.

5. Finite element analysis for a realistic simulation of structural components using RCM

Recent rapid advances in computing science and methodological approaches (e.g., finite element method) have greatly enhanced the ability of numerical models to simulate and predict behavior in structural components. Effectiveness is particularly enhanced when a 3D model with different elements and materials is employed (Lowes 1999). Numerical modeling further provides a powerful tool with which to study new systems and explore possible new concepts in structural analysis and design. This type of modeling was adopted in the present work to simulate the detailed behavior of structural components.

After two decades of developing finite element methods for reinforced concrete structures, a number of general-purpose finite element codes, such as ABAQUS, ANSYS, ETABS, and SAP2000 have been developed and made commercially available for research and professional applications. However, it is widely recognized that a more accurate and computationally stable material constitutive law for concrete is necessary in order to represent the behavior of reinforced concrete structures accurately in terms of localized interactions between concrete and reinforcing elements.

5.1 Development of reinforced concrete material models

Most materials of engineering interest initially respond elastically. However, the materials models that are to be developed for materials such as concrete and steel bar are incremental theories in which the mechanical strain rate is decomposed into elastic and plastic (inelastic) parts.

5.1.1 Concrete

This study adopts the model proposed by Chen *et al.* (1991) to predict concrete behavior. It is a continuum, plasticity-based, damage model for concrete that assumes tensile cracking and compressive crushing to be the main two failure mechanisms in concrete material. The evolution of the yield (or failure) surface is controlled by two hardening variables, $\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$, which are tensile and compressive equivalent plastic strains, linked to failure mechanisms under tension and compression loading, respectively. The model assumes that the uniaxial tensile and compressive response of concrete is characterized by damaged plasticity, as shown in Fig. 2.

The concrete damaged plasticity model also assumes non-associated potential plastic flow. The flow potential G used for this model is the Drucker-Prager hyperbolic function, as shown in Eq. (1).

$$G = \sqrt{(\varepsilon\sigma_{t0}\tan\psi)^2 + \bar{q}^2} - \bar{p}\tan\psi \tag{1}$$

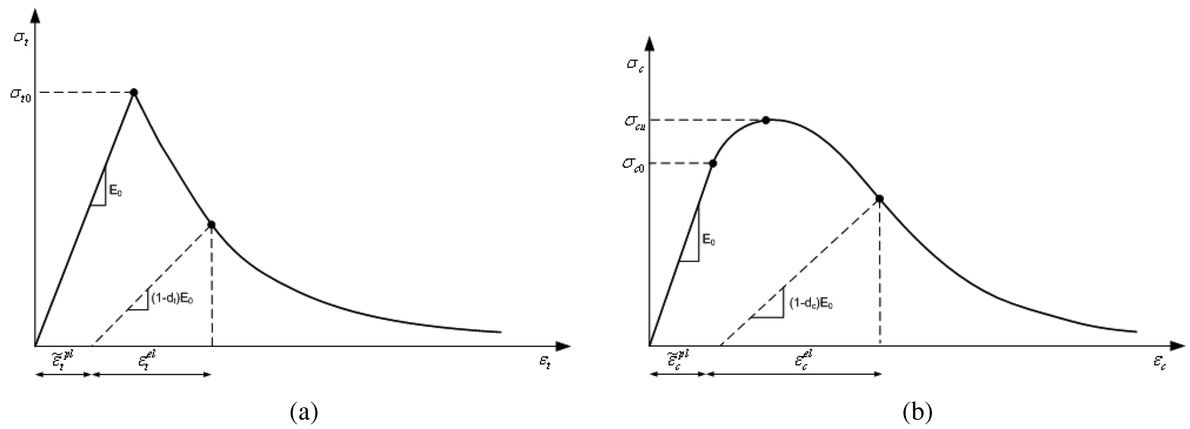
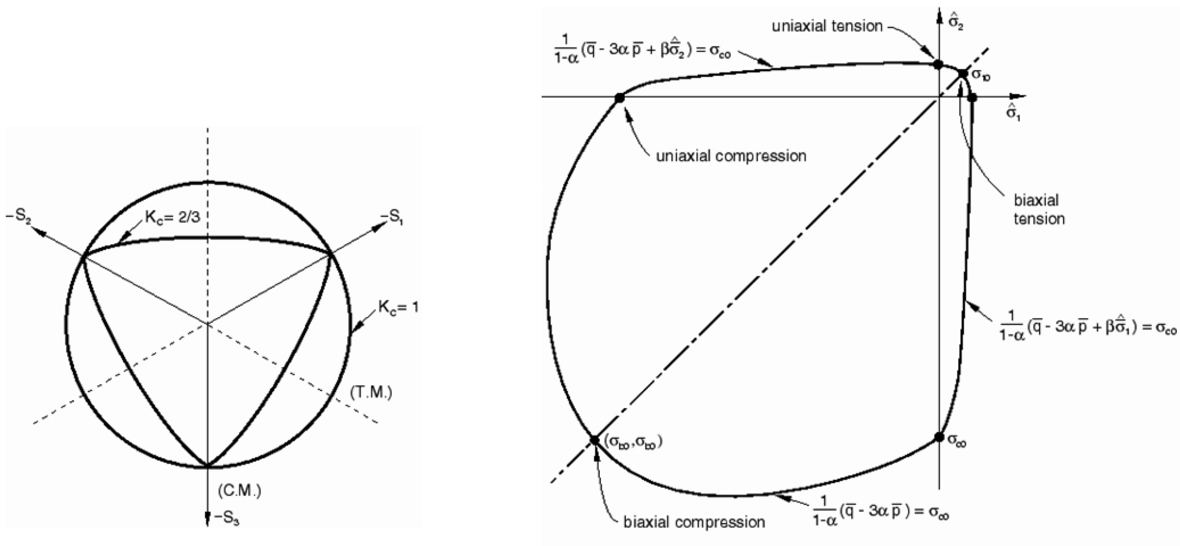


Fig. 2 Response of concrete subjected to uniaxial loading: (a) tension loading and (b) compression loading (ABAQUS 2004).



(a) (b)

Fig. 3 Yield surfaces in: (a) the deviatoric plane, corresponding to different values of and (b) the plane stress (ABAQUS 2004).

where ψ represents the dilatation angle measured in the $p-q$ plane at the high confining pressure, $\bar{p}$ represents hydrostatic pressure stress, and $\bar{q}$ represents Mises equivalent effective stress.

The concrete model makes use of the yield function of Lubliner *et al.* (1989), with the modifications proposed by Lee and Fenves (1998) to account for differences in strength evolution under tension and compression, as shown in Fig. 3. The evolution of the yield surface is controlled by the hardening variables, $\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$. In terms of effective stresses, the yield function takes the form of Eq. (2).

$$F = \frac{1}{1-\alpha}(\bar{q} - 3\alpha\bar{p} + \beta(\tilde{\varepsilon}^{pl})(\hat{\bar{\sigma}}_{\max}) - \gamma(-\hat{\bar{\sigma}}_{\max})) - \sigma_c(\tilde{\varepsilon}_c^{pl}) = 0 \tag{2}$$

with

$$\alpha = \frac{(\sigma_{b0}/\sigma_{c0}) - 1}{2(\sigma_{b0}/\sigma_{c0}) - 1};\ 0 \le \alpha \le 0.5 \tag{3}$$

$$\beta = \frac{\bar{\sigma}_c(\tilde{\varepsilon}_c^{pl})}{\bar{\sigma}_t(\tilde{\varepsilon}_t^{pl})}(1-\alpha) - (1+\alpha) \tag{4}$$

$$\gamma = \frac{3(1-K_c)}{2K_c - 1} \tag{5}$$

Here, $\hat{\bar{\sigma}}_{\max}$ represents the maximum principal effective stress, σ_{b0}/σ_{c0} represents the ratio of the initial equibiaxial compressive yield stress to the initial uniaxial compressive yield stress, K_C represents the ratio of the second stress invariant of the tensile meridian $q_{(TM)}$ to that of the compressive meridian $q_{(CM)}$ at an initial yield for any given value of the pressure invariant P, such that the maximum principal stress is negative, $(\hat{\bar{\sigma}}_{\max} < 0)$. Furthermore, it must satisfy the condition $0.5 < K_C \le 1.0$. Also, $\bar{\sigma}_t(\tilde{\varepsilon}_t^{pl})$ represents the effective tensile cohesion stress and $\bar{\sigma}_c(\tilde{\varepsilon}_c^{pl})$ represents the effective compressive cohesion stress.

5.1.2 Reinforcing steel bar

This study adopts the simple and computationally efficient model developed by Lowes (1999) to predict steel behavior. The model is developed on the basis of modern plasticity theory, with the one dimensional behavior of a steel reinforcement bar being representative of an elastic-plastic material. The one-dimensional constitutive relationship developed on the basis of plasticity theory has been defined by the following equation

$$f_s = E_s(\varepsilon_s - \varepsilon^P - \varepsilon^T) \tag{6}$$

where f_s represents the stress at steel, E_s represents the modulus of elasticity of steel, ε_s represents total strain, ε^P represents plastic strain, and ε^T represents thermal strain.

The yield and inelastic flow of steel at relatively low temperatures, where loading is relatively monotonic and creep effects are not important, can typically be described adequately using plasticity models. For the present study, these models adopt standard Mises yield surfaces with associated plastic flows to define isotropic yielding. Perfect plasticity and isotropic hardening conditions are both included in plasticity models. Certain finite element commercial software packages provide appropriate isotropic hardening models, which are useful in cases that involve gross plastic straining or in which straining at each point runs essentially in the same direction (in strain space) throughout the analysis.

5.2 3D modeling of reinforced concrete structural component

In the current finite element analysis, 3D models are developed for each reinforced concrete structural component, with 3D models used so that concrete stress-strain behavior can be better simulated and understood. In this study, concrete is modeled as a solid continuum element, with the reinforcing steel bar assumed to be a beam element (Darwin 1991). A perfect bond between concrete and steel bar is assumed.

ABAQUS provides a constraint technique, called "the embedded region constraint", to describe the reinforced concrete model (ABAQUS 2004). This constraint technique can be used to analyze an element or group of elements embedded in host elements. In this study, the reinforcing steel bar is assumed to be the embedded element and the concrete is assumed to be the host element. ABAQUS searches for geometric relationships between embedded and the host element nodes. If an embedded element node lies within a host element, the translational degrees of freedom at the node are eliminated and the node becomes an embedded node. Translational degrees of freedom of the embedded node are constrained to the interpolated values of the corresponding degrees of freedom of the host element. Embedded elements are allowed rotational degrees of freedom, which cannot be constrained by embedding. However, all host elements can only have translational degrees of freedom, and the number of translational degrees of freedom at a node on the embedded element must be identical to the number of translational degrees of freedom at a node on the host element.

The confinement effect from the transverse reinforcement may be considered for column components. This passive confinement is done to restrain lateral expansion of concrete, enabling higher compression stresses and strains. More importantly, higher strains sustained in the compression zone can delay failure. The confinement model by Mander *et al.* (1988) has been adopted for the column model.

Lan (1998) proposed that in order to develop the confined model, the concrete model in ABAQUS must be modified to account for the confinement effect. This is done because the model provided by ABAQUS fails to properly account for such an effect. In ABAQUS, the hardening rule is solely a function of plastic strains and not a function of hydrostatic stresses (ABAQUS 2004). This causes the model to underestimate confined concrete ductility. This is not a problem in cases involving unconfined concrete, as corresponding axial plastic strains under uniaxial compression are relatively smaller than those faced in confined concrete cases. Hence, to account for the confinement effect, the hardening rule must be dependent on both plastic strains and hydrostatic stresses. Also, to account for the modified hardening rule in ABAQUS, the provided model is designed to be dependent on hydrostatic stress. Several sets of stress-strain relationships associated with various confining pressures are used, instead of one set of uniaxial compression data, as shown

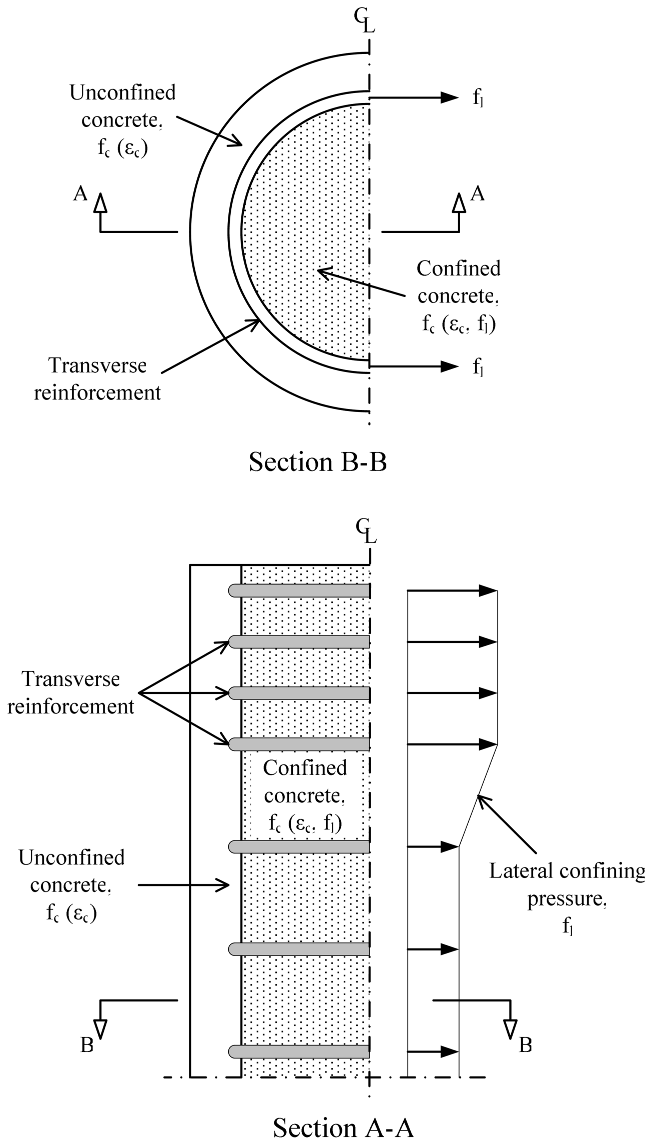


Fig. 4 Confined concrete associated with lateral confining pressure in the column component

in Fig. 4. These pressure dependent stress-strain relationships have been implemented in ABAQUS using so-called solution dependent field variables, as material properties can be made dependent on these field variables.

6. Translating SCM response to RCM behavior

A requirement for performing a realistic simulation of a structural system with the proposed multi-level modeling approach (as shown in Fig. 1) is that the employed analysis algorithm must be able to handle the transition between distinct scales. Namely, global behavior obtained from the macro-level model must be translated into the behavior of constituent components at the micro-level model. Also, the localized effects of a component obtained from the micro-level model must be related to the behavior of the macro-level model in the global system (D'Ambrisi and Fillippou 1999). Thus, mechanisms are required to extract displacement and stress values in the SCM and then appropriately link these quantities to known values in the corresponding RCM analysis.

Since the SCM has been modeled as a frame structural element and the RCM has been modeled as a continuum element, using beam theory as presented by Ghugal and Shimpi (2001) is a way extract the SCM response and then translate it into the RCM behavior. One of the beam theories known as the First-order Shear Deformation Theory (FSDT) or Timoshenko Beam Theory is based on kinematics. It was mentioned in the literature by Rebello *et al.* (1983). Eqs. (7) and (8) show the kinematic equations of the theory,

$$u(x,z) = u_0 + z\phi_y \tag{7}$$

$$w(x,z) = w(x) \tag{8}$$

where u_0, w_0 and ϕ_y represent unknown variables and ϕ_y represents the rotation of a cross section of the beam at the neutral layer.

Another beam theory applicable to this problem is the Parabolic Shear Deformation Theory (PSDT). The third order beam theory, which includes transverse shear strain and non classical (nonlinear) axial stress (as proposed by Krishna Murty, 1984) is adopted for this study. In this theory, the parabolic transverse shear stress distribution through the cross section of the beam can be obtained using constitutive relationships. The displacement field of the theory is shown in Eqs. (9) and (10),

$$u(x,z) = u_0 - z\frac{dw_b}{dx} + z\left[1-\frac{4}{3}\left(\frac{z}{h}\right)^2\right]\phi_y \tag{9}$$

$$w(x,z) = w_b(x) + w_s(x) \tag{10}$$

where u_0, w_b, w_s and ϕ_y represent unknown functions. The transverse deflection is considered as a sum of two partial deflections, i.e., the deflection due to bending (w_b) and deflection due to transverse shearing (w_s).

Fig. 5 shows the mechanism for extracting and translating from a single nodal displacement in the SCM into several nodal displacements in the corresponding plane in the RCM. This figure illustrates the mechanism in a Two-dimensional (2D) plane. Each single SCM node has three displacement components, namely the vertical displacement, horizontal displacement, and rotation. Those displacements should be extracted and translated into the corresponding plane of the RCM. As it is modeled as the continuum element, the nodal displacement of the RCM consists only of

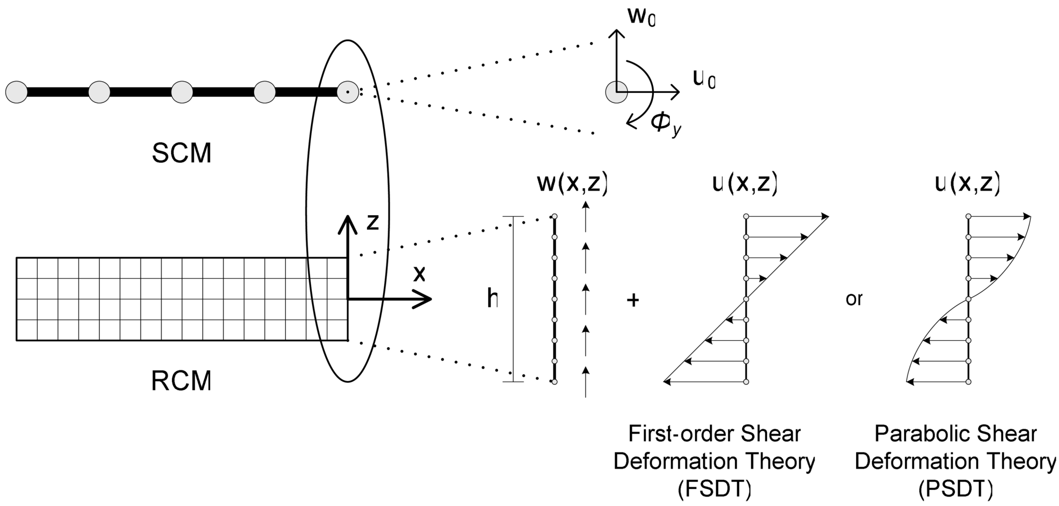


Fig. 5 The extracting and translating mechanism of nodal displacement from SCM to RCM

vertical and horizontal displacements. The vertical displacement of the SCM can be directly translated into several nodal displacements in the corresponding RCM plane as it is not a function of the *z*-axis. However, for horizontal displacement, which is function of the *z*-axis, the single nodal displacement of the SCM should be extracted using the FSDT or PSDT equation into several nodal displacements and translated into corresponding RCM plane. This 2D mechanism is the basis for developing the extracting and translating mechanism in the 3D space.

7. Verification of the proposed simulation method

A simple frame structure (Fig. 6(a)) is used to help analyze the accuracy of the proposed simulation method. This two-level model consists of a global frame model in line elements and a detailed component model (RCM) in plane elements. A downward concentrated load is applied to the middle span of the beam portion of the global frame model. Because the beam component represents the most critical structural part, it is further analyzed as the RCM using the proposed simulation method. Each nodal displacement in the beam portion in the global frame model assumed as SCM is extracted and translated into several nodal displacements in the corresponding plane of the RCM in order to yield a detailed response.

RCM analysis results obtained using the proposed simulation method are then compared to the assumed exact solution, obtained by analyzing a detailed portal frame model (Fig. 6(b)). The peak and average percentage of error of strain in the beam component are 0.1639% and 0.0083%, respectively. At this level of accuracy, the authors assess the proposed simulation method as appropriate to simulate the response of actual structural systems.

8. Application of the proposed method for realistic simulation of structural systems

A proposed multi-level hierarchical modeling and simulation method has been presented, and its feasibility has been preliminary checked using a simple example. In this section, the proposed

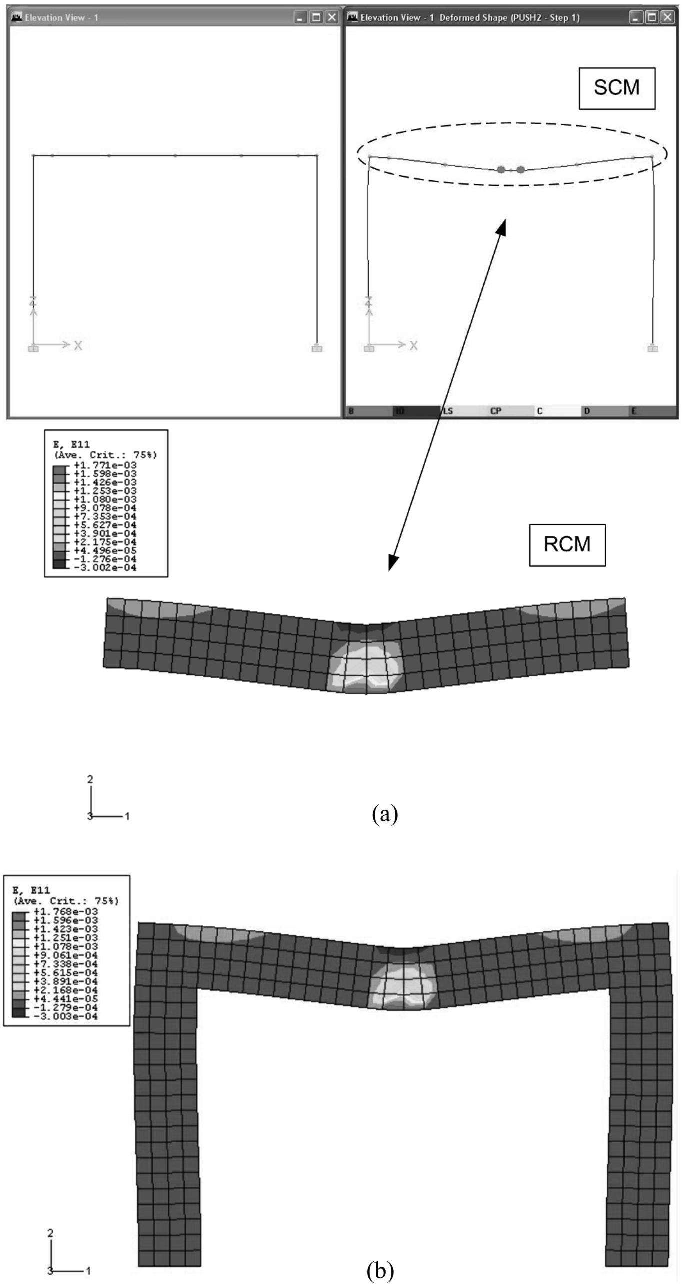


Fig. 6 The 2D portal frame model: (a) global frame model (SCM) and RCM simulated response of beam component using proposed method, (b) response of detailed portal frame model

simulation method is further investigated by applying it to simulate two actual structural systems. The first is a one-story simple reinforced concrete structure, built by National Center for Research on Earthquake Engineering (NCREE) in Taipei, Taiwan. The second is the Reuipu Elementary

Table 1 Material properties

	Simple Reinforced Concrete Structures		Reuipu Elementary School Building	
Concrete	Value		Value	
Elastic Modulus	19940 MPa		18263 MPa	
Poisson's Ratio	0.175		0.175	
Compressive Strength	18 MPa		15.1 MPa	
Compressive Ultimate Strain	0.001544		0.001414	
Tensile Strength	1.9 MPa		1.6 MPa	
Tensile Ultimate Strain	0.000095		0.000088	
Tension Residual Strength	0.1%		0.1%	
Steel	Transverse Reinforcement	Longitudinal Reinforcement	Transverse Reinforcement	Longitudinal Reinforcement
Elastic Modulus	210000 MPa	210000 MPa	210000 MPa	210000 MPa
Yield Strength	282.5 MPa	280.0 MPa	477.2 MPa	280.2 MPa
Strain for Strain Hardening	2.0%	2.1%	1.2%	1.9%
Hardening Modulus	6900 MPa	6900 MPa	6900 MPa	6900 MPa
Tensile Strength	429.6 MPa	374.5 MPa	678.1 MPa	425.5 MPa
Strain for Tensile Strength	10%	10%	9%	10%
Fracture Strain	18%	18%	15%	18%

School building, a two-story reinforced concrete structure, located in Taoyuan, Taiwan. Material properties for each structure, including concrete and steel rebar materials, are listed in Table 1, with numbers in bold-face type identifying the values approximated as part of the current study.

Structural system frame members such as beams and columns are each assumed to be beam elements. Joints between frame members are assumed to be rigid connections. Seismic lateral loading and pushover analysis using ETABS was performed for both structural systems.

The Two-level hierarchical models built for the one-story simple reinforced concrete structure and Reuipu Elementary School building are shown in Fig. 7(a) and (b), respectively. Both structures represent non-ductile reinforced concrete structures typical of construction in Taiwan. Column components represent a critical structural part. Column components of the structural global model were picked and analyzed using RCM. A finite element analysis conducted by ABAQUS was then used to simulate the behavior of RCM. A single nodal displacement in the structural global model (SCM) was extracted and translated into several nodal displacements on the corresponding plane in the RCM with regard to yielding the detailed responses of this component.

Simulation results were compared with experimental results, which were based on the in-situ monotonic pushover collapse test for both the simple reinforced concrete structure and Reuipu Elementary School building. The capacity curve for the structural global model obtained by pushover loading simulation was compared with that for the real structural system. Furthermore, the detailed response of the column component, analyzed as RCM, was compared with the picture observation of the experimental progress.

8.1 The one-story simple reinforced concrete structure

The design of the simple reinforced concrete structure is shown in Fig. 8. In order to be

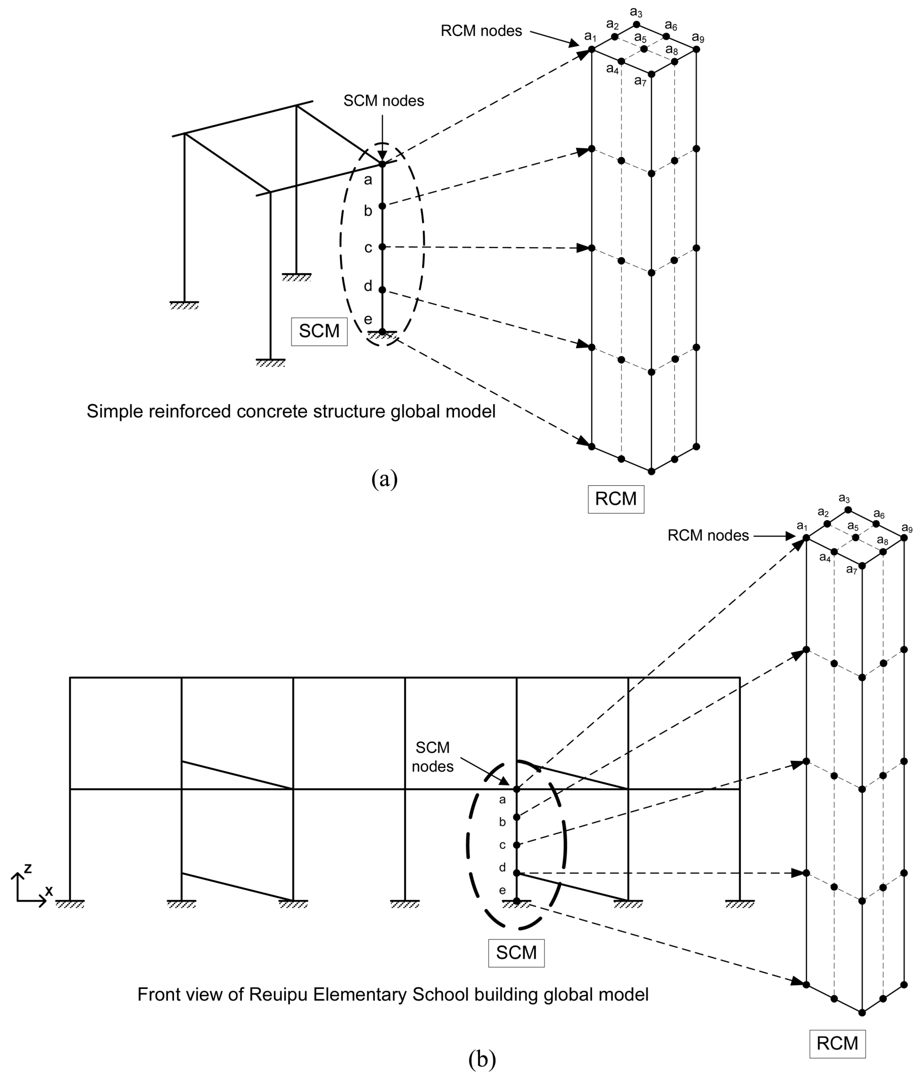


Fig. 7 Representations of two-level hierarchical models for (a) simple reinforced concrete structure and (b) Reuipu Elementary School building

representative of non-ductile reinforced concrete structures typical in Taiwan, the structure was designed as a strong-beam weak-column system.

Plastic hinge distributions in the first and last steps of pushover analysis are illustrated in Fig. 9. The first step shows that the plastic hinge occurred first at the bottom part of the column. The failure point of this structure also occurred at the bottom part of the column as shown in the last step. Note that the plastic hinge did not occur at all on beam components. Therefore, plastic hinge distribution in the pushover analysis shows clearly that the column-sway-collapse mechanism occurred due to lateral loading.

Fig. 10 compares the analytical capacity curve to the experimental result. During initial displacement, a similar pattern is shown for both. After the yield point is reached, the capacity curve of the analytical model shows a value that is about 2.4% lower than that for the experimental result. At the

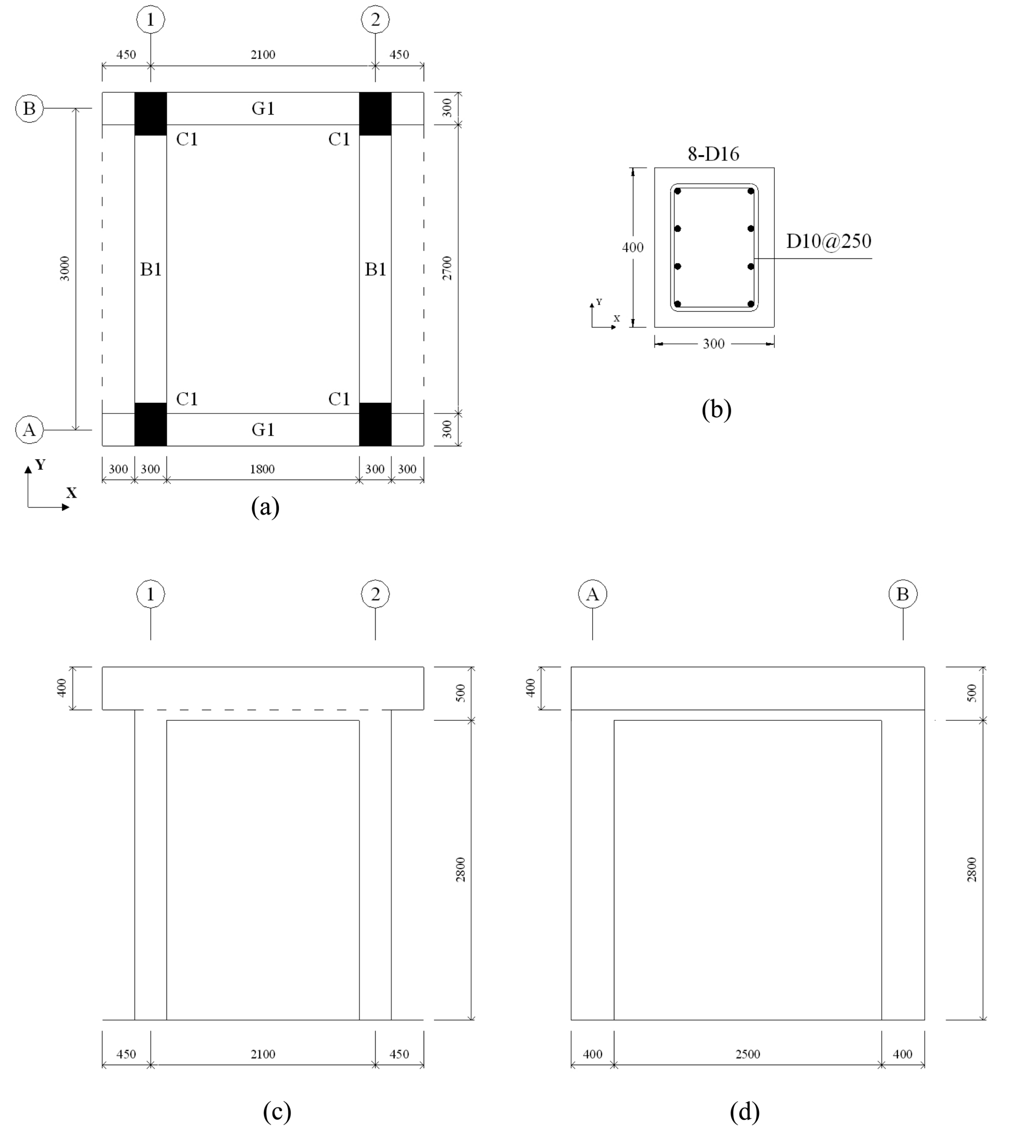


Fig. 8 Layout of simple reinforced concrete structure: (a) top view, (b) cross section of column C1, (c) front view and (d) side view

ultimate point, the analytical model also has a lower base shear and a lower displacement value than the experimental result. The experimental result shows the base shear and displacement to be 211.57 kN and 50.56 mm, respectively, at ultimate point. Meanwhile, the analytical model shows the base shear value to be 206.62 kN and the displacement to be 49.51 mm. Immediately following attainment of the ultimate point, the capacity curves of both the experimental result and analytical model drop significantly. At this stage, the analytical model still has enough residual strength to support the structure until it totally collapses, at a maximum displacement of 55.17 mm. In actual experimental results, the structure withstood a slightly higher displacement for the same amount of base shear, and the maximum displacement that was reached before the structure totally collapsed

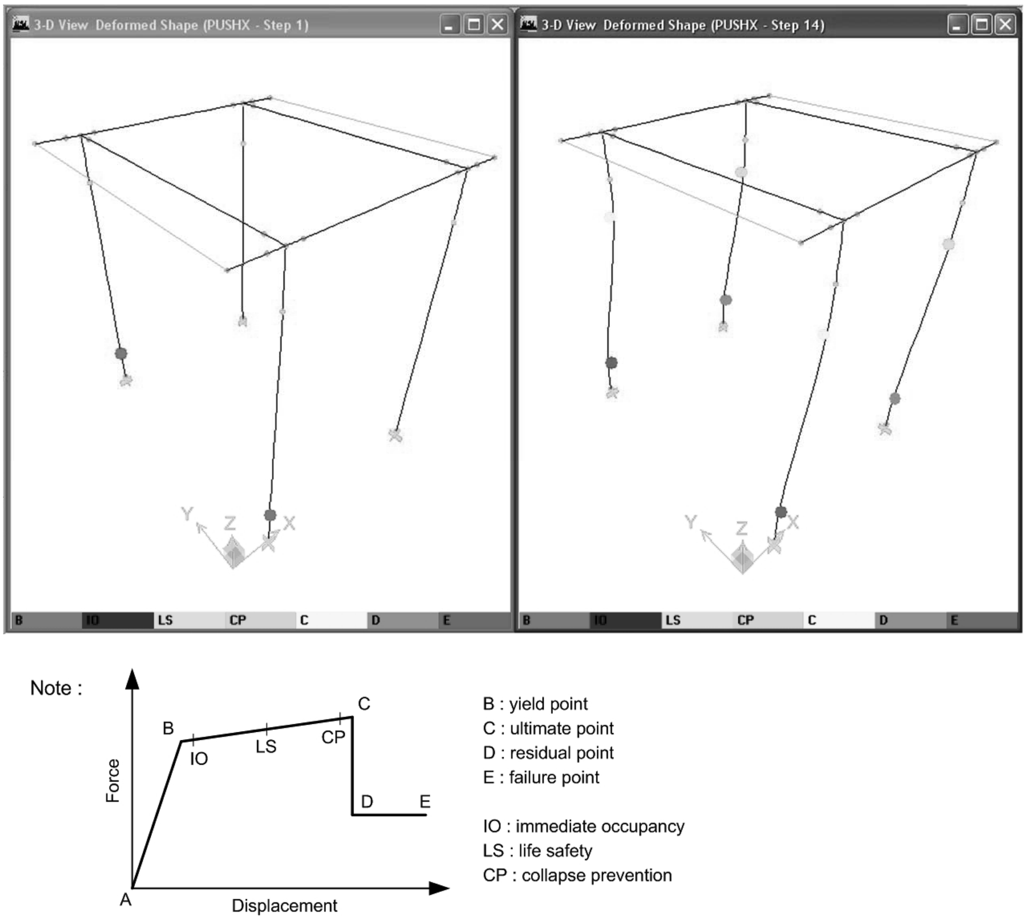


Fig. 9 Plastic hinge distribution on the first and last step of a pushover analysis of a simple reinforced concrete structure using ETABS

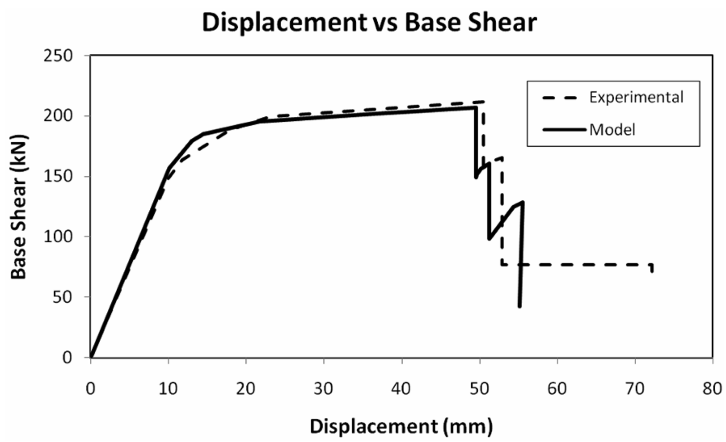


Fig. 10 Comparison of capacity curves for the analytical model and experimental results for a simple reinforced concrete structure

was 72.22 mm.

The column components simulation using RCM is shown in Fig. 11. The finite element model depicts the deformed shape and contour of the column vertical strain (E33) at a roof displacement of 34.85 mm. Column response and behavior were evaluated and compared with the observed experimental result (pictured). The deformation of this column begins at the bottom part of the

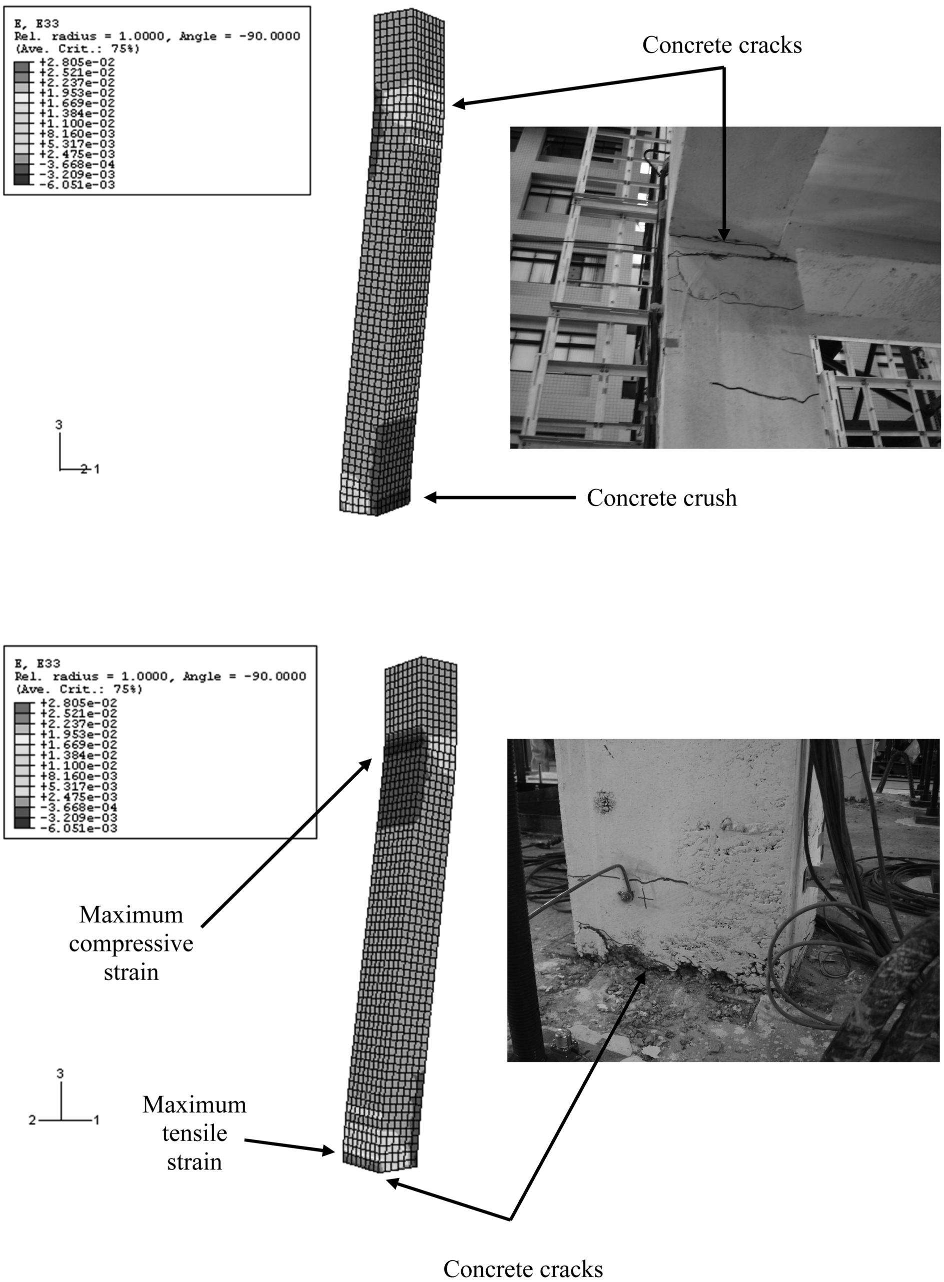


Fig. 11 Comparison of the RCM simulated response and photographs of an actual experiment for the column of a simple reinforced concrete structure

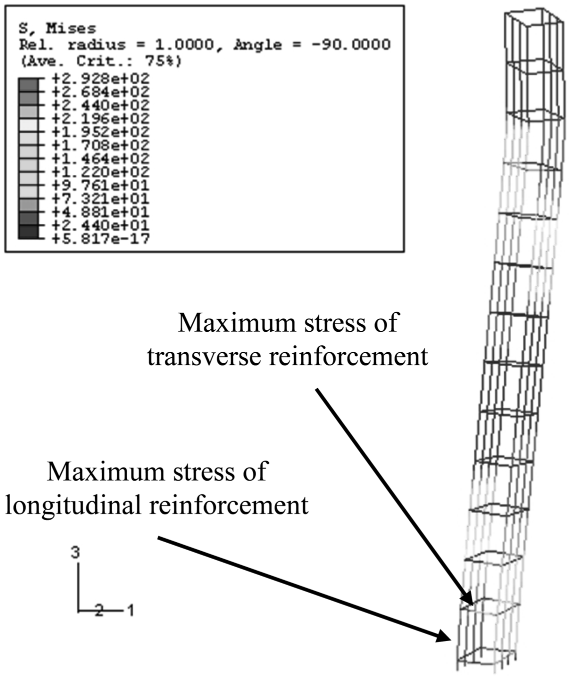


Fig. 12 The RCM simulated response of the steel rebar within a simple reinforced concrete column, as found in the experimental structure

column at the intersection of the beam-column joint. The column region below floor level was assumed to be rigid, and concrete strain varied between +0.0281 and -0.0061. Maximum tensile strain in the concrete occurred at the bottom part of the column. Meanwhile the maximum compressive strain in the concrete occurred on the same side, but toward the top of the column near the beam-column joint. In the confined concrete region, the strain varied between +0.027 and -0.0012.

The maximum stress of the longitudinal reinforcing steel bar in this condition was 292.8 MPa, which is within the strain hardening region (Fig. 12). The transverse reinforcements remained elastic, as the maximum stress was only 90.9 MPa.

8.2 The two-story reinforced concrete structure on the campus of reuipu elementary school

The layout of Reuipu Elementary School building is shown in Fig. 13. Column components have a very low ratio value of transverse reinforcement. In fact, this ratio value falls below standard minimum design requirements for seismic resistance, as presented in ACI-318 (2005). Also, rebar hoops within each column were spaced too far apart, exceeding the maximum spacing permitted to meet ductile design requirements. Therefore, this structure is considered to be a non-ductile reinforced concrete structure having a strong-beam weak-column design, and has been designated as potentially hazardous under seismic conditions.

The seismic lateral load was applied and evaluated. For typical school buildings in Taiwan, most damage occurs parallel to the building corridor. Therefore, we conducted a pushover analysis for

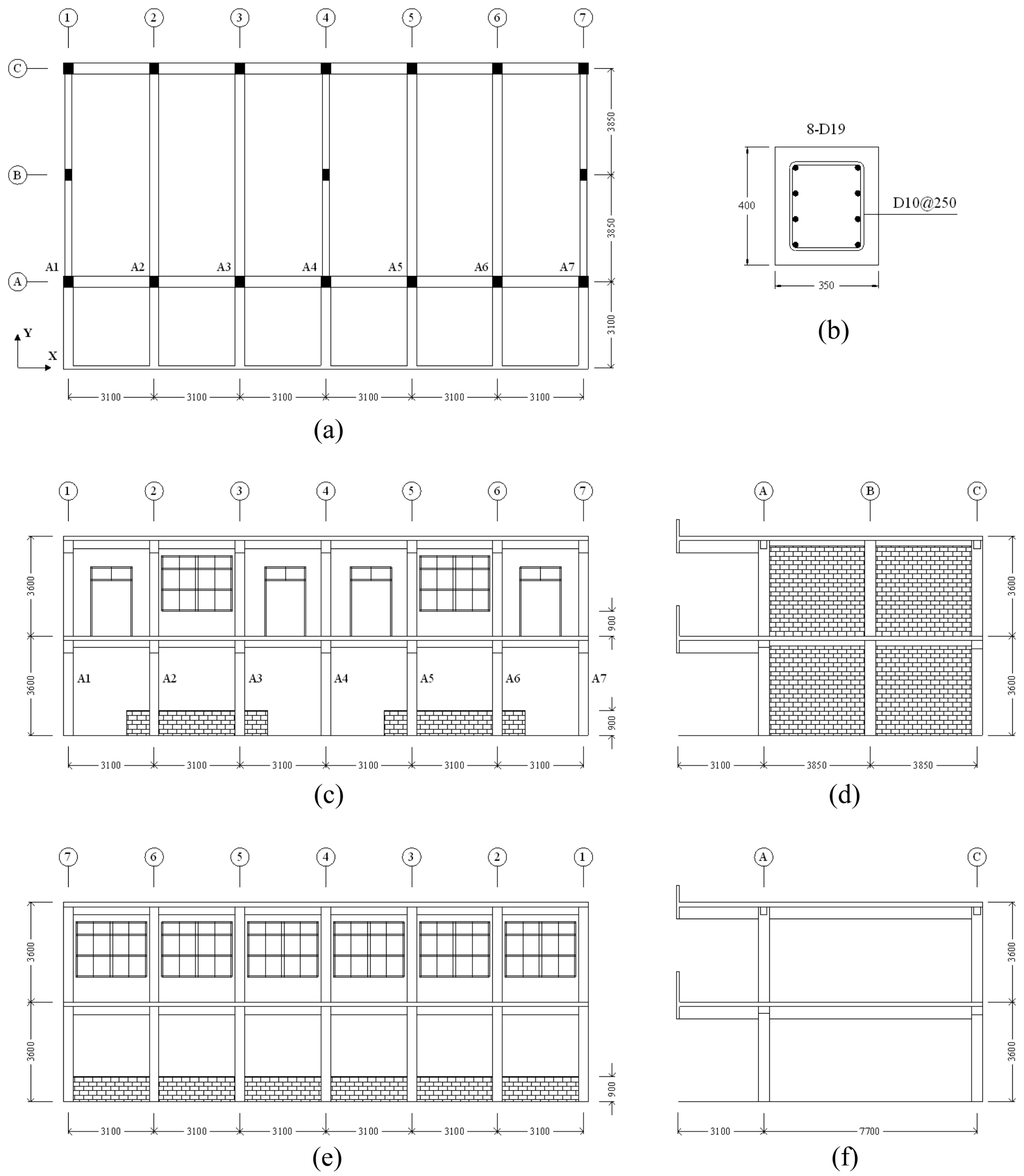


Fig. 13 Layout of Reuipu Elementary School building: (a) typical top view of the 1st and 2nd floor, (b) cross section of columns A1 through A7, (c) front view, (d) sectional view of portals 1, 4, and 7, (e) back view and (f) sectional view of portals 2, 3, 5, and 6.

that direction only.

The brick wall component was considered in this model. Based on the Chung *et al.*'s (2006) model and a simulation presented by Lin (2006), the brick wall under lateral load can be simulated as a compression brace that has a stiffness and strength equivalent to that of the brick wall.

Fig. 14 shows the plastic hinge distribution that occurred during the first and last steps of pushover analysis. In the first step, the plastic hinge effect first occurred in the beam component on the first floor. The last step revealed that, while many of the beam components exceeded the yield

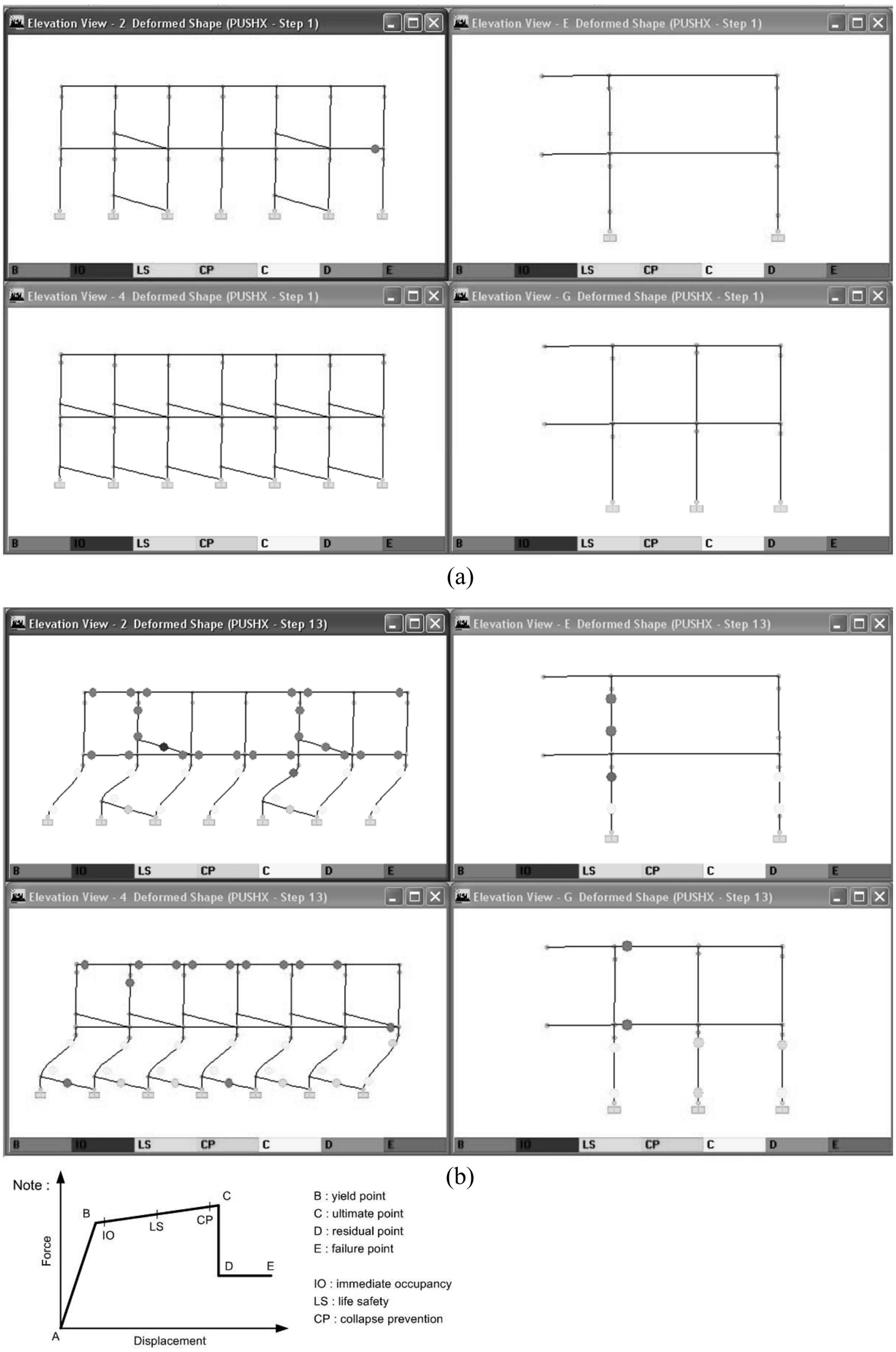


Fig. 14 Reuipu Elementary School building plastic hinge distribution of the pushover analysis using ETABS: (a) first step and (b) last step

point, none had reached the failure point. However, one of the column components had reached the failure point, which led to building collapse. The plastic hinge distribution of the pushover analysis clearly shows that this structure experienced a column sway (soft story) collapse mechanism due to lateral loading. This mechanism was a direct result of its strong-beam weak-column design. Horizontal components such as the beams and the floor slabs provided high strength and stiffness to resist lateral loading. However, columns lacked sufficient ductility to accommodate significant displacement. Though the added effects of strength and stiffness from the brick wall were considered, the soft-story sway mechanism could not be avoided.

Fig. 15 shows the analytical capacity curve compared with the experimental result. Within the elastic range, both displayed the same behavior. Also, the ultimate point of the capacity curve was approximately the same between the two. In the first half of the plastic range, the analytical capacity curve was around 3.3% higher than that in the experimental result. However, after the roof displacement of the building reached 109 mm, the base shear of the analytical model suddenly dropped from 968 kN to 278 kN. This result was different significantly from the experimental result which remained ductile.

The difference between capacity curves for the analytical model and experimental results for each structural global model was possible because the analytical model was based solely on calculation. As each successive iteration could not identify a converging solution, the iteration ceased, resulting in a lower capacity curve. In actual structures, many factors remain that can still influence failure, including non-structural components, which remain ductile and retain some of the loading.

With regard to building designs that use a reinforced concrete frame with brick wall components, while also considering the monotonic pushover load direction, column types are classified into two categories. The first is the ordinary column, i.e., a column that is unconstrained between beams and the second is the so-called captive column, i.e., a column constrained by a rigid septum, such as a brick wall. Monotonic pushover load is applied in the x-positive direction. According to the layout of Reuipu Elementary School building, the ordinary columns were columns A1, A4, and A7. Captive columns included columns A2 and A5. Columns A3 and A6 were defined as partially captive. Columns A2, A3, A5, and A6 were assumed to be captive because they were connected to a brick wall in the x-direction (i.e., the loading direction). Thus, the y-direction was not significant

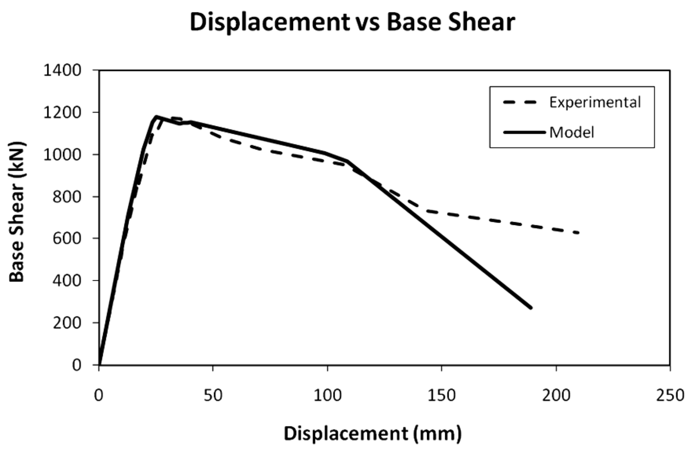


Fig. 15 Comparison of capacity curves for the analytical model and experimental results for Reuipu Elementary School building

in terms of captivity with respect to loading. Columns A1, A4, and A7 were classified as ordinary columns because they were not connected to a brick wall in the x-direction (even though they were connected to brick walls in the y-direction).

Fig. 16 shows the deformed shape and contour plots of vertical strain (E33) of a concrete first story ordinary column (A4) at the point in which school building story drift is 0.75% and roof displacement is 55.16 mm. Only the regions of the column within the beam-column joint are considered as rigid. Therefore, the strain region in question lay between beam-column joints. Column concrete strain varied between +0.0093 and -0.0041. Maximum concrete tensile strain

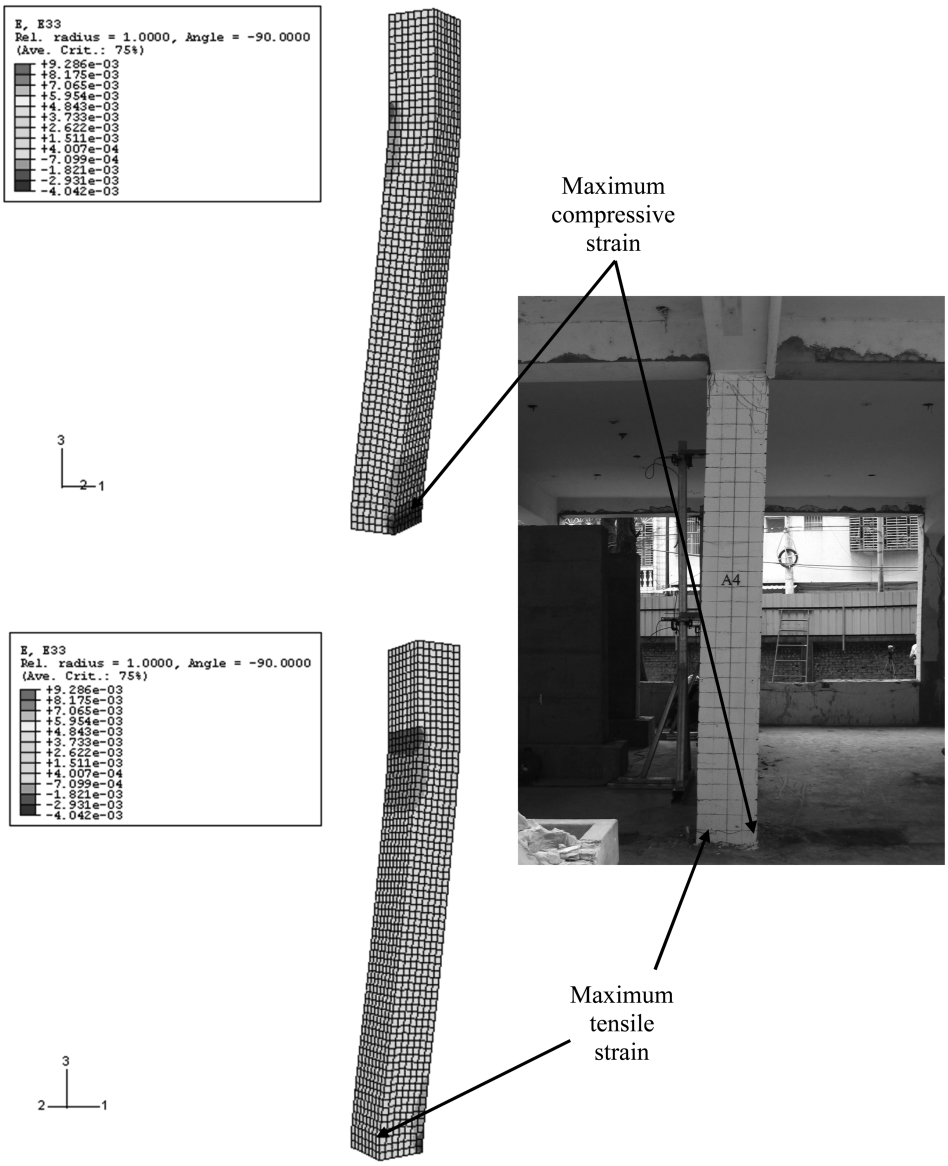


Fig. 16 Comparison of the RCM simulated response of an ordinary column and photograph of the actual Reuipu Elementary School building

occurred in the bottom part of the column where it is adjoins the floor beam. Maximum concrete compression stress also occurred in the bottom part of the column on the opposite side. In the confined concrete region, the strain varied between +0.0092 and -0.002.

Fig. 17 shows the deformed shape and contour plots of vertical strain (E33) of a concrete first story captive column (A5) at the point in which school building story drift is 0.75% and roof displacement is 55.16 mm. The captive column response and behavior was evaluated and compared with observed experimental results. In the captive column, the deformable region was shortened,

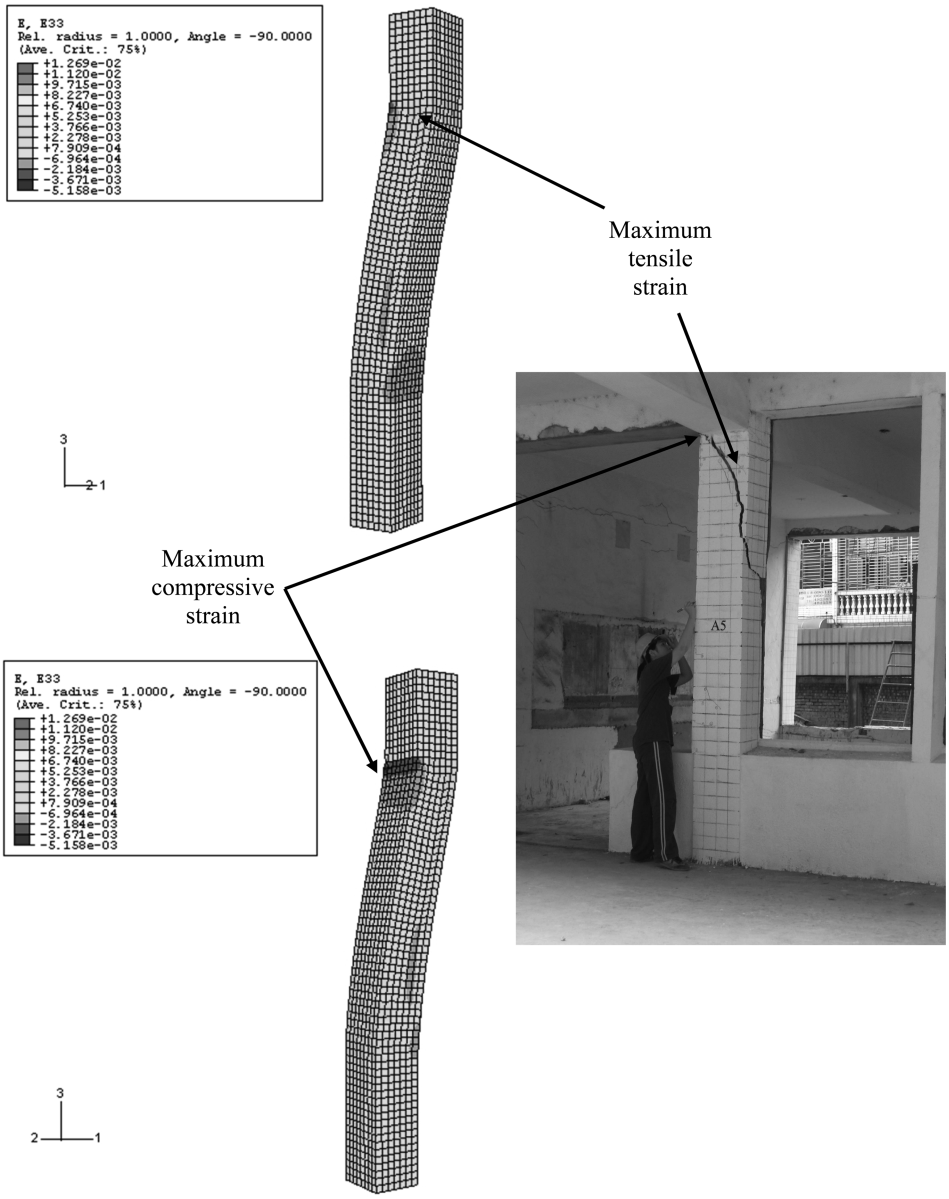


Fig. 17 Comparison of the RCM simulated response of a captive column and photograph of the actual Reuipu Elementary School building

ranging from the top of the brick wall to below the beam-column joint. The section of the column within the brick wall and the beam-column joint was assumed to be more or less rigid. Column concrete strain varied between +0.0127 and -0.0052. Maximum tensile strain occurred in the top half of the column (several centimeters below the beam-column joint) and formed a diagonal split, which determined the point of column failure. Maximum compression stress also occurred on the

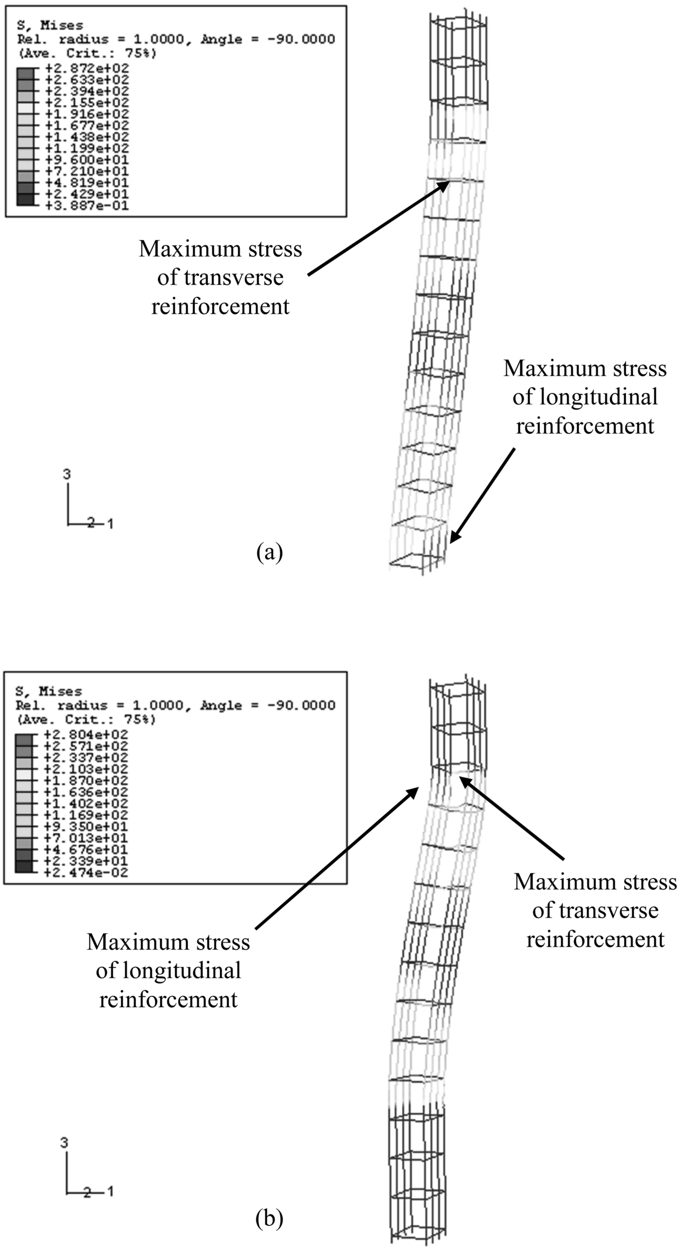


Fig. 18 RCM simulated response of a steel bar within (a) an ordinary column and (b) a captive column, as found in Reuipu Elementary School building

top side of the column, opposite from the maximum tensile strain. In the confined concrete region, the strain varied between +0.0116 and -0.0019. Moreover, collective results clearly show that it was the presence of the brick wall that caused structural collapse (Fig. 14).

A comparison of steel reinforcement bar stresses for an ordinary column (A4) and captive column (A5) when school building story drift is 0.75% and roof displacement is 55.16 mm is shown in Fig. 18. The ordinary column (A4) shows the maximum stress of the longitudinal reinforcement to be 287.2 MPa and the transverse reinforcement to be 117.4 MPa. For the captive column (A5) the maximum stress of the longitudinal reinforcing steel bar was 280.4 MPa, which exceeded the yield point. However, transverse reinforcements remained elastic, as the maximum stress was only 228.9 MPa.

The results of the analytical model for one-story simple reinforced concrete structure and Reuipu Elementary School building subjected to monotonic lateral loading accurately described the behavior of the structural system. The model for each structure predicted successfully the soft story sway mechanism as it was observed in the experimental test. The capacity curve of the model was also able to approximate safely the experimental result. The behavior of the analytical model indicates the SCM as appropriate for use in simulating a structural response to monotonic pushover loading.

The RCM model's evaluation of monotonic pushover loading accurately described structural component behavior. The extraction and translation of SCM nodal displacement into corresponding RCM boundary conditions linked successfully RCM response and behavior to the global model. The column RCM predicted the deformed shape and stress results as they were observed in the experimental test. Observed behavior agreed with the RCM of the ordinary column in showing that maximum damage would occur on the bottom part of the column. For the captive column, both the RCM and the observed behavior revealed that maximum damage would occur on the top side of the column. Evaluation of the column's finite element model indicates the column's RCM as appropriate for use in simulating the response of the concrete column subjected to monotonic pushover loading.

9. Conclusions

A proposed method for realistic simulation of structural systems by combining a simplified component model (SCM) and a rigorous component model (RCM) is investigated and verified in this paper. The SCM response on the global structural model is identified and assigned to appropriate nodal values in the corresponding RCM analysis. As a result, the detailed response of structural components can be obtained by a synchronous RCM referenced to a corresponding SCM residing in the global structural model. Thus, the refined RCM represents a mechanism able to determine the detailed behavior of inelastic frame members such as constituent beams, columns, and beam-column joints, as well as the overall behavior of concrete and rebar composite material.

Using nonlinear finite element analysis, the proposed method is applied to two realistic simulations: a simple reinforced concrete structure and a building on the campus of Reuipu Elementary School. Pushover analysis is performed for both structures, following which the column component of structures is taken and analyzed as SCM and RCM. The difference between simulation and experimental results for the simple reinforced concrete structure and Reuipu Elementary School building are about 2.4% and 3.3%, respectively. Results indicate the proposed method to be appropriate for use in simulating structural system behavior.

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