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# NONLINEAR 3D SOLID FINITE ELEMENT ANALYSIS OF RC SHEAR WALL SUBASSEMBLAGES UNDER LATERAL LOADS

Kyung Ran HWANG[1] and Han Seon LEE[2]

## SUMMARY

This paper presents results of nonlinear 3D solid finite element analyses of a 2-, 3-, 5-, and 10-story RC shear wall subassemblage under lateral loads, and effects of the number of stories and the existence of slabs and coupling beams on the stiffness, strength, ductility, and failure mode are investigated. The following conclusions are drawn from analytical results: (1) The higher the number of stories, the lower initial stiffness and ductility the model has. By contrast, strengths are similar regardless of the number of stories. The damage and failure modes of all the models appear to be the combined mode of flexural and shear failure. (2) The effect of the existence of slabs on the initial stiffness and the strength is almost negligible, but the ductility of models with slabs is generally higher than that without slabs. Cracks and damages of the wall are not generally concentrated at the lower portion of the wall, but distributed over the significant range of the whole wall height regardless of the existence of slabs. Finally, (3) coupling beams are effective in increasing the initial stiffness and strength of models, but they do not improve the ductility capacity of models. Overturning moment resistance contributed by tension-compression coupling forces at the base accounts for approximately 30~40% of the total overturning moment capacity. This ratio does not change significantly with regard to the number of stories.

***Keywords****: reinforced concrete; shear wall; nonlinear pushover analysis; 3D finite element analysis; coupling beam.*

## INTRODUCTION

More than 58% of the total number of housing units use apartment buildings as dwelling houses in Korea (KNSO 2010). These residential apartment buildings commonly consist of high-rise reinforced concrete (RC) wall structures. The style of these RC box-type structural systems such as shown in figure 1(a) is unique around the world and the seismic performance of these structures has been investigated with due interest, neither in Korea nor abroad. However, there are great deals of experimental and analytical studies to evaluate the seismic performance of RC wall structures. Wallace (2002, 2004) observed the behavior of RC structural walls with rectangular, T-shaped, and barbell-shaped cross sections through experimental and analytical studies, examined conventional codes, and suggested the new approach for the seismic design of structural wall. Also, Orakcal and Wallace (2006) adopted the MVLEM element, which was a macro wall model, to simulate the static test results of wall specimens with high accuracy, but they did not deal with the influence of slabs and openings. Meanwhile, Balkaya and Kalkan(2004) carried out experimental and analytical researches on RC box-type structures to evaluate the influence of slab-wall interaction and compare effects of 3D shell and 2D membrane element modeling to highlight the significance of 3D effect. Recently, Kalkan and Yuksel (2008) performed the nonlinear 3D solid finite element analysis of RC box-type structures, but they did not directly compare the influence of existence of slabs and coupling beams. Lee et al. (2011) performed earthquake simulation tests on a 1:5 scale 10-story RC residential building model, and this study revealed that the considerable crack and damage in slabs due to the flexural behavior and in walls caused by the membrane actions.

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# STATIC LOADING TEST OF PRECAST CES SHEAR WALLS IN FLEXURAL FAILURE

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## SUMMARY

The objective of this study is to develop the CES shear wall in Composite concrete encased steel (CES) structures composed of steel and fiber reinforced concrete, and Static loading tests were conducted on the two shear wall with a simplified anchoring method for connecting the CES frame to the FRC wall panel. The wall panel of one specimen was precasted. And specimens were designed to be flexural failure mode. This paper describes the summary and investigates the failure mode, ultimate strength and restoring force characteristics. In addition, an evaluation method of the ultimate strength of CES shear walls is discussed. As the result, the results showed that maximum shear force of precast CES shear walls were smaller than that of integral construction CES shear walls. Precast shear walls showed slip at the boundary of wall panel and CES frame at the small level of drift angle, it is thought that the slip deformation affect failure mode of precast shear wall. And it was found that the effect of anchorage of horizontal wall reinforcing bar on flexural strength of shear wall was small.

***Keywords****: CES shear wall; fiber reinforced concrete; structural experiment; Precast wall panel; anchorage method.*

## INTRODUCTION

Steel Reinforced Concrete (SRC) Structures developed in Japan have good structural performance for resisting lateral forces imposed by wind and earthquakes, and have been adopted for medium-rise, high-rise, and super high-rise buildings. However, the number of SRC buildings constructed has decreased since the 1990s. Although the decrease might be caused by the development of a new structural engineering system called the High-strength concrete structure or Concrete-Filled Steel Tube (CFT) structure, the main reason seems to be the construction problems that increase construction costs and lengthen construction schedules. Even so, it could be important that SRC structures provide better seismic performance in comparison with other structural systems. So, the authors aim to develop a structural system with as good seismic performance as SRC structures and good workability, and have conducted a continuing development study on composite Concrete Encased Steel (CES) structures composed of steel and fiber reinforced concrete (FRC) as shown in Figure 1. In the experimental study on CES columns, CES beam-column joints, and a two-bay two-story CES frame, it was confirmed that the CES structural system showed stable restoring force characteristics and good seismic performance.

On the other hand, a shear wall, which is a main earthquake resistant member, is also effective at increasing the stiffness and strength of the CES structural system. However, it will be difficult to arrange wall reinforcing bars in the CES structure, which contains encased steel in beams and columns. Recently, there have been some researches on methods of joining a frame and wall panel in SRC shear walls. It is likewise an important problem to improve the workability of a frame and wall panel joint in the CES structure.

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# INTERACTION BETWEEN OVER-TURNING MOMENT AND SHEAR CAPACITY OF SEISMIC LIGHT GAGE BEARING WALL

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## SUMMARY

Light gage structure, where the plate thickness is less than 2.3mm, is limited up to three story buildings in Japan. This type of structure is used mainly on residential and commercial buildings. Seismic force is one of the most important design factor; light gage bearing wall is usually used for the horizontal force resisting component. Light gage bearing wall is composed with light gage frame and structural laminated plywood or gypsum board. Usually, over-turning moment due to lateral force is negligible in low rise building; however, it is questionable to adapt the same theory in middle rise building. To clarify the effect of over-turning moment to the bearing wall on light gage structure, full scale testing was conducted in this study. From the full scale testing, following results were found. (1) Over-turning moment was mainly resisted by the couple forces of the stud. (2) Horizontal force was mainly resisted by screws which are connecting plywood or gypsum board to the light gage frame. (3) Effect to over-turning moment to the shear capacity was observed when the magnitude of the moment exceeded 50% of its capacity.

***Keywords**: light gage structure; bearing wall; middle rise building; over-turning moment; structural test; seismic performance.*

## INTRODUCTION

Light gage structure, where the plate thickness is less than 2.3mm, is limited up to three story buildings in Japan (BSJ 2007). This type of structure is used mainly on residential and commercial buildings. Due to limited construction space, there is a demand to increase the story height limit in this system. Seismic force is one of the most important design factor; therefore, light gage bearing wall is usually used for the horizontal force resisting component. Light gage bearing wall is composed of light gage frame with structural laminated plywood or gypsum board. When the light gage structure is designed, bearing wall is treated as cantilever column (JISF 2002). This means that the anchor bolt in the first floor will be subjected in a high axial force. In low rise building this effect is limited and it was not a significant concern. However, if the story height will be higher, the over-turning moment subjected to the bearing wall will be larger. Therefore, it is very important to clarify the effect of over-turning moment to the capacity of the bearing wall. The Building Standard limited the story of this system, the research which is considering over-turning moment with horizontal force is limited (Sugimoto et al. 2008).

The purpose of this study is to clarify the interaction between over-turning moment and shear capacity of the light gage bearing wall. For this purpose, new testing set-up, which can apply over-turning moment and horizontal force simultaneously, was developed. Total seven full scale specimens were prepared and several loading protocol was applied in the testing.

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# STUDY ON DEFORMATION CAPACITY OF R/C BEARING WALLS WITH RECTANGULAR CROSS-SECTION BASED ON EXPERIMENTAL DATABASE

*(This paper was submitted to 15th WCEE, 24 to 28 September 2012, Lisbon, Portugal.)*

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## SUMMARY

An experimental database of about 119 R/C bearing wall specimens with rectangular cross-section was made. The accuracy of ultimate flexural and shear capacity estimation methods commonly used in Japan and U.S. was discussed. Based on the database, the statistical analysis was also conducted to assess the influence of the key parameters provided by the codes in Japan and U.S. such as the detail of the boundary elements, the ratio of ultimate flexural capacity to ultimate shear capacity, etc. on the ultimate deformation capacity of the R/C walls.

***Keywords***: *R/C bearing wall, Experimental database, Ultimate deformation, Ultimate capacity.*

## INTRODUCTION

R/C bearing walls have been used as structural members in many countries located at seismic zone because of their high stiffness and capacity. Bearing walls set in a moment resisting frame (walls with boundary columns) have been commonly used in Japan. *AIJ Standard for Structural Calculation of Reinforced Concrete Structures* (Architectural Institute of Japan 2010) was revised in 2010 and permits the use of bearing walls without boundary columns if the requirements are complied. However, these walls are at risk for brittle failure such as compression failure or buckling at wall boundary observed at the 2010 Chile Earthquake as shown in Fig. 1 (Tani et al. 2011). The research about multi-story bearing walls without boundary columns as structural core walls of high-rise buildings has been conducted recently in Japan and there is some amount of experimental data. In this research, the experimental data of bearing walls without boundary columns in the past literatures was collected. This paper discusses the relationships between deformation capacity and experimental parameters.

**Figure 1.** Damage of multi-story bearing wall without boundary columns at the 2010 Chile Earthquake

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# NEW SHAPE OF LCVA WITH MULTI CELLS REDUCING WIND INDUCED RESPONSES OF TALL BUILDINGS

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## SUMMARY

LCVA is in general installed on the top of a building for reducing wind-induced response by tuning its natural frequency to building's one. This study deals with the experiment of new LCVA with multi cells whose vertical tubes are divided into several square columns for easily changing natural frequency. A real 64-story building is chosen as control target model. Proposed LCVA is designed according to SDOF model of the building. Experimental building model is manufactured with steel mass, spring, and linear motion guide to describe SDOF model. Vertical tube of a designed LCVA has 18 cells to have wide range of natural frequency. Experiment shows that LCVA has natural frequency band from 0.65Hz to 0.81Hz. Shaking table test is carried out to verify control performance of LCVA installed on the building model. Test results indicate that the LCVA has not only wide band of natural frequency by simply opening or closing cells but also attenuates building response by tuning LCVA frequency.

***Keywords**: liquid column vibration absorber; natural frequency; shaking table test; control force.*

## INTRODUCTION

Previous study deals with the design of robust tuned liquid column damper (TLCD) with adjustable natural frequency[Kim etc., 2011, 2012]. In general, natural frequency of TLCD is assumed to be same as that of a building for its response control. After the building is constructed, its natural frequency is different from what we expect in design phase. Since the shape of TLCD is determined in the design phase of a building, the natural frequency is fixed before the building is constructed. Proposed robust TLCD has the same general form as conventional TLCD. The conventional TLCD can control only liquid height in order to tune its natural frequency to that of the building after construction. New robust TLCD can control not only liquid height but also the area of vertical column with lots of small columns, so that it can adjust its natural frequency with change of TLCD size. The vertical columns of the TLCD are divided into several individual cell type small column. The liquid cannot move by air pressure in airtight cell. The natural frequency can be adjusted by controlling area of vertical columns with changing the number of airtight cells. Shaking table test is performed to investigate the TLCD for adjustable natural frequency by changing the number of airtight cells. Shaking table test shows that natural frequency of TLCD is changeable by adjusting the number of cells as well as control force.

This study addresses control performance verification by shaking table test. Target real 64 story building is chosen and scaled down as SDOF model. According to scale down rule, LCVA is designed. Experimental building model is manufactured with steel mass, spring, and linear motion guide to describe SDOF model. Vertical tube of a designed LCVA has 18 cells to have wide range of natural frequency. Experiment shows that LCVA has natural frequency band from 0.65Hz to 0.81Hz. Shaking table test is carried out to verify control performance of LCVA installed on the building model. Test results indicate that the LCVA has not only wide band of natural frequency by simply opening or closing cells but also attenuates building response by tuning LCVA frequency.

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# SEISMIC PERFORMANCE EVALUATION OF A SCHOOL BUILDING RETROFITTED WITH ULTRA-HIGH STRENGTH STEEL

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## SUMMARY

The 1999 Chi-Chi earthquake tested the seismic resistant capacity of the buildings in Taiwan. Over 600 low to mid-rise reinforced concrete buildings collapsed, including many street buildings and school buildings. In the study, a retrofit system, which combines a ultra-high strength steel column and low-yield point steel plates, was proposed to enhance the seismic performance. In the previous test, the ultra-high strength steel column had been proved that it was able to perform two times larger flexure strength and elastic deformation capacity than that of the conventional steel column. In the study, a series of numerical simulations were conducted to examine the performance of the proposed retrofit system on a school building. The results indicated that the system significantly reduced the drift concentration factor, which was defined as the ratio of the maximum story drift angle to the roof drift angle, to 1.28 from 1.8 subjected to the El Centro ground motion. The maximum drift angle was reduced to 0.9% from 1.8%.

***Keywords**: reinforced concrete, deformation concentration, ultra-high strength steel, low-yield point steel, retrofit.*

## INTRODUCTION

The 1999 Chi-Chi earthquake tested the seismic resistant capacity of the buildings in Taiwan. Over 600 low to mid-rise reinforced concrete buildings collapsed including street buildings and school buildings. The common failure patterns, such as failure in the longitudinal direction due to the luck of walls, short-column damages due to constraint by windows walls, and strong-beam-weak-column failure behavior were observed in the earthquake. The damages caused significantly large story deformation concentration and led to many collapses (Tsai et al. 2000, Architecture and Building Research Institute, ABRI, 1999). To reduce the story deformation concentration, controlled rocking systems have been proposed and proved to be effective in preventing the damage or large deformation concentrate at certain locations. By installing damping devices in the system, the energy dissipation capacity of the structure can be significantly increased and the seismic response can be reduced (Wada et al.2009,Deierlein et al. 2009).

The H-SA700 steel is a newly developed ultra-high strength steel with a specified minimum yield strength of 700 MPa. This steel is produced by an advanced plate manufacturing process called thermo-mechanical control process (Yoshida et al. 2009), which produces relatively small amount of $CO_2$, saves rare alloying elements and significantly reduces energy consumption compared to conventional manufacturing processes. Taking advantage of its high strength, Lin et al. (2011) proposed a weld-free column system using the H-SA 700 steel. The bending test of the column indicated that the column yielded at story drift angle of 4%. The flexure strength and the elastic deformation capacity both reached two times large as that of the conventional steel column.

In the study, a new rocking system was proposed using the developed ultra-high strength steel column for the retrofit purpose. The column was placed in the weak direction of the existing building. The low-yield point steel

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# Innovative Construction Technologies for Sustainable Development

**Raymond Wang[1], Samuel Yen-Liang Yin[2]**

## SUMMARY

This paper presents some recent innovations on the design and construction of precast reinforced concrete structures including the multi-spiral shear reinforcement, automatic precast concrete production technology, and application of precast waffle slab system for high-tech factories. The innovative multi-spiral shear reinforcement provides both technical and cost advantages over the conventional column stirrups. It can greatly enhances concrete confinement, improves ductility and seismic resistance, reduces the labor costs by more than 45%, reduces the amount of steel shear reinforcement by 30 to 50%, reduces the carbon imprint, shortens the construction time by 50% and is highly cost-effective for precast construction. The development of automatic precast concrete production technology provides good quality and efficiency for construction of concrete structures. Only five days are needed for a typical building story by using the automatic precast concrete production technology. The precast waffle slab system can be installed with very high precision (1.5 cm in 400 m) and reduce construction time by up to 50%. The precast waffle slabs can also be used as the formwork to support the construction loadings. The construction time and accidents can be significantly reduced.

***Keywords**: SEEBUS; precast construction, multi-spiral shear reinforcement, precast waffle slab*

## INTRODUCTION

Reinforced concrete is one of the most widely used construction material for buildings in the Asian region and in the world. In Taiwan, about 84% of the buildings are reinforced concrete structures and more than 95% of these buildings are constructed by cast-in-place concrete construction method. However, precast concrete construction technology has been successfully applied for the building construction in recent years.

The current precast building construction market in Taiwan is dominated by a "composite construction method," which integrates the installation of precast components with the cast-in-place concrete (Yin, 2008). The main feature is that the manufacturing of the building components is done in the factory and then the components are transported to a construction site for complete fabrication. One advantage of the precast construction is the reduction of the possibility of unstable qualities induced by weather, poor construction resources, and so on. In addition, the precast method can also save the construction resources and shorten the construction time. Moreover, it may decrease the risk of the on-situ environmental impacts, occupational accidents, and worker health.

In 1995, RUENTEX GROUP introduced several advanced precast technologies from Europe and Japan into Taiwan. Nowadays RUENTEX GROUP develops more innovative precast technologies, such as one-bar hoop, multi-spiral shear reinforcement, precast waffle slab systems for high-tech factories, and several others. Its application has been expanded into shopping malls, high-tech factories, office buildings, and residential buildings. These technologies can provide both technical and cost advantages. The following sections briefly introduce some of the invented technologies. It is believe that the continuous innovations on construction automation and precast technologies will result in rapid progress of sustainable development of construction industries.

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# FLEXURAL BEHAVIOR OF PRESTRESSED CONCRETE BEAMS USING STEEL-FIBER REINFORCED CONCRETE

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## SUMMARY

Steel-fiber reinforced concrete (SFRC) shows pseudo-strain-hardening behavior caused by uniform distribution of multiple cracks under tensile stress. SFRC is expected to enhance the tensile properties such as strength and stiffness of the resulting composite material. Pretensioned members have been used to control crack width and deflection under service load. Prestressing force applied on them is generally smaller than the one on post-tensioned members. However, the construction cost of pretensioned members is lower than the one of post-tensioned members because they do not need anchorage devices. Prestressed concrete members are considered less ductile than ordinary reinforced concrete members. In this study flexural performance to be enhanced by pretension technology and SFRC is discussed. Cyclic loading tests were conducted on pretensioned beams using steel-fiber reinforced concrete, where the main parameter was the volumetric ratios of fibers: 0.0, 0.3 and 0.5%. The tests showed that the maximum flexural strength and the initial cracking load of the beams with steel-fiber increased at most 16.4% over the beams without steel-fiber.

***Keywords**: prestressed concrete; pretension; steel-fiber reinforced concrete; flexural strength; crack width*

## INTRODUCTION

In recent years, Fiber Reinforced Concrete (hereafter referred to FRC) material has been developed and studied for application to structural members. A property of this material is Pseudo-Strain- Hardening behavior (hereafter referred to PSH behavior) caused by the distribution of multiple fine cracks under tensile stress (Kunieda et al. 2006, Suwada et al. 2006, 2007, Fisher et al. 2003). Fibers have been used to enhance tensile characteristics of concrete by surpressing crack growth and improving mechanical behavior (Bilal S. 2001). Concrete with fibers is characterized by its fiber content. The fiber content is the weight of fibers per unit volume in concrete; it is the product of the volume fraction $V_f$ (volume of fibers per unit volume of concrete) and the specific gravity of the fibers. It is still uncertain how the tensile characteristics of FRC affect the flexural resistance mechanism of structural elements (Suwada 2006). Various analytical and empirical methods have been proposed to predict the flexural strength of the composite material reinforced with fibers (Swamy 1982, Henager 1976). Of all the fibers currently in use to reinforce cement matrices, steel-fibers are the only fibers that can be used for carrying long-term load (Swamy et al. 1982, Padmarajaiah et al. 2001).

Prestressed concrete requires the concrete to attain high compressive strength at an early age. In addition to its higher compressive strength, high-strength concrete possesses an increased tensile strength, and reduced shrinkage and creep strains than normal-strength concrete. High-strength concrete has been found, however, to be more brittle when compared to normal-strength concrete. Inclusion of fibers is one way to alleviate the problem of brittleness in high-strength concrete. Pretensioned members have been used to control crack width

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# SEISMIC PERFORMANCE OF PRESTRESSED CONCRETE BEAMS -EFFECT OF BOND STRENGTH AND MILD STEEL RATIO-

Jaeman LEE[1] and Minehiro NISHIYAMA[2]

## SUMMARY

The objective of this study is to analytically investigate the structural behavior of unbonded prestressed concrete beams. Analyses were conducted using a software package (FINAL). Load-displacement curves of an unbonded beam analytically obtained based on the constitutive laws of the materials well simulated the experimental results reported in the past. Two parameters were chosen for a parametric study; bond strength for PT bar and mild steel ratio. The analysis results can be summarized as follows; the smaller the bond strength was, the smaller flexural capacities of prestressed concrete beams were obtained. The bond strength, $\tau_{max}$, did not significantly affect hysteretic energy dissipation capacity represented by equivalent viscous damping factor, $h_{eq}$. $h_{eq}$ was affected by the amount of mild steel, $\rho_r$. Yielding of non-prestressed mild steel dissipated most of hysteretic energy. In addition, the larger $\rho_r$ was, the larger residual deformation was observed after the mild steel yielded. The larger $\rho_r$ was, the larger prestressing force was necessary to effectively close residual cracks. Based on the analysis results, the bond strength of PT tendon affected the flexural strength while the mild steel ratio, $\rho_r$, had a significant effect on hysteretic energy dissipation capacity, residual deformation and residual crack width.

***Keywords****: unbond, prestress, FEM, bond strength, energy dissipation, residual deformation, residual prestress*

## INTRODUCTION

Post-tensioned precast concrete structures are generally constructed with grouted tendon. The grouting is of importance for preventing tendons from corroding and providing bond between tendon and concrete. Unbonded tendon is coated with a corrosion inhibitor such as grease. Prestressed concrete (PC) structures with unbonded tendon systems have been constructed to save work and time for grouting.

The significant development of a new technology using a self-centering function of unbonded PT tendon systems has been established as part of PRESSS project. Unbonded tendon systems once developed to save grouting work help construct a high-performance building for earthquake resistance.

Because of no bond between PT tendon and concrete, the structural behavior of members with unbonded tendon is different from the one of members with bonded tendon.[1] Because plane section assumption cannot be applied to unbonded tendon, the flexural strength of prestressed concrete members using unbonded tendon is considered to be 10 - 20% smaller than when bonded tendon is used. However, there are not enough analytical or experimental investigations for the structural behavior of prestressed concrete members with unbonded tendons.

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# CONSIDERATIONS OF BOND-SLIP FOR PERFORMANCE-BASED DESIGN OF RC BEAM-COLUMN CONNECTIONS

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## SUMMARY

Under repeated cyclic loading, the earthquake resistance of reinforced concrete beam-column connections is degraded by the bond-slip and yield penetration of beam flexural bars at the beam-column joint. In the first part of the present study, existing cyclic test results of beam-column connections were investigated to evaluate the energy dissipation capacity. The result showed that the energy dissipation capacity of the test specimens correlated well with the bond resistance at the joint, which is affected by the concrete strength, and the embedment length, yield strength, and diameter of the beam re-bars. On the basis of the result, the energy dissipation capacity of the beam-column connections was defined as a function of bond resistance. Using the energy function, a simplified hysteresis model for beam-column connections was developed. In the second part, two strengthening methods for the beam-column joint, 45° bent bars and 90° hooked bars, were studied in order to restrain bond-slip by ensuring a longer development length of beam flexural bars. Cyclic loading tests were performed for five full-scale cruciform beam-column connections with low $h_c/d_b$ ratios (column depth-to-beam rebar diameter ratio = 18.1 or 14.2). The test results showed that even with the low $h_c/d_b$ values, the yield penetration and bond-slip of the beam re-bars were substantially decreased by using the strengthening methods. This result indicates that the effective embedment length of beam re-bars can be used for the design of the beam-column connections with strengthening bars.

***Keywords**: Beam-column connection; Energy dissipation; Earthquake design; Reinforced concrete.*

## INTRODUCTION

At the plastic hinge of the beam-column connections, the tensile plastic strain of the flexural re-bars continues to increase under repeated cyclic loading (Paulay 1996; Eom and Park 2010). Due to the tensile plastic strain, the compressive yield stress of the re-bars develops early under the reversed loading even before the flexural cracks in the compression zone are completely closed. As a result, the beam-column joint is subjected to increased bond stress. Thus, in the beam-column connections having small $h_c/d_b$ ratios, significant bond-slip occurs under cyclic loading. In the present study, the bond resistance parameters that are closely related to the hysteretic energy dissipation were investigated by using existing test results. Then, the load-deformation relationships of beam-column connections were defined with the design parameters such that the area enclosing a cyclic curve is the same as the energy dissipation capacity.

In the second part of the present study, strengthening methods for beam-column joints were studied, focusing on restraining bond-slip of beam re-bars at the joint. Full-scale beam-column connections were tested under cyclic

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# ANCHORAGE BEHAVIOR OF HEADED BARS IN BEAM-COLUMN JOINT

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## SUMMARY

In this study, headed bars are adopted to replace traditional standard hooks for avoiding reinforcement crowding in the beam-column joints. Seven full size experiments were conduct to investigate the influence of headed bars on beam-column joints. Not only to understand the anchorage behavior of headed bars in beam-column joints, but to comprehend the effect on flexure moment of beams. The issue on whether the performance of beam-column joints with different arrangements of head bars satisfy the seismic requirement is also discussed. For interior beam-column joints with two types of arrangement, i.e., butt and splices, the strength and ductility of the beam-column joints are found to be satisfactory. Additionally, for butt arrangement with enough anchorage length, the yielding of reinforcement and crush of concrete at beam end are obvious during incremental loading vertically. For the exterior beam-column joints, this study recommends the seismic resistant anchorage length of headed bars should be the ACI 318-08 provision multiplied by 1.23 times on condition that the clear spacing of headed bars is $2.2d_b$, the beam-column joint belongs to the low shear ratio and strong column-weak beam pattern.

***Keywords***: *Reinforced concrete; Beam-column joint; Headed bar; Anchorage length*

## INTRODUCTION

The connection point of column component and beam component (beam-column joint), is one of the structures bearing the most complex forces, especially for the beam-column joints of side columns, corner columns, top columns, discontinuous beam or column components. The traditional engineering method for the seismic design results in more congested reinforcement configuration in joints since the main reinforcement end adopts the hook for anchorage and all reinforced hooks are anchored to the center of joints, which easily causes poor concrete pouring due to difficult construction. Furthermore, it leads to the doubt about insufficient strength in beam-column joints. Traditionally, the anchoring of the reinforcement ends mostly adopts the reinforced hook. The specification ACI 318-08 (2008) suggests that the anchorage length of the hook should be greater than that of headed bar. If headed bars can be used to replace the reinforced hooks for anchorage, not only the embedded length can be shortened, but also the bars congestion issue in beam-column joints can be resolved. Additionally, it can improve the construction quality of beam-column joints. When headed bars are used in joints, the main reinforcement in beams will be tensed under the cyclic forces. Therefore, when headed bars are adopted for the main reinforcement of beams, the anchorage length will directly affects the behavior of the plastic hinge of beams. Additionally, it will further affect the seismic performance of a bending-resistant structure. If the bar is mechanically anchored, it can be made at the factory previously instead of the on-site manufacturing; in other words, it can guarantee both the accuracy of material size and quality, and then improve the production rate and reduce the cost in mass production. Additionally, it can also realize quality control in the construction.

The results of the previous study (Lin et al. 2010) on applying headed bars to the anchorage of beam-column joints show that, when specimens with headed bars are arranged with the appropriate anchorage length and the clear spacing of $4d_b$ and $2.2d_b$ in headed bars, both of them can develop the plastic hinge in the beam ends, and the bending moment strength can also reach 1.2 times of the nominal bending moment of beam $M_{nb}$. The results

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# FLEXURAL STRENGTH OF BEAM-COLUMN JOINT IN REINFORCED CONCRETE BUILDING WITH SOFT-FIRST STORY

**Susumu TAKAHASHI[1], Halim Sefatullah[2], Toshikatsu ICHINOSE[3], Masaomi Teshigawara[4], and Hiroshi Fukuyama[5]**

## SUMMARY

In this research, RC buildings with soft-first story are considered to discuss the flexural strength of the beam-column joint. Columns in the soft-first story are usually designed to be large section compared with those of upper stories because this story is required to have large capacity for resisting earthquakes. To verify the flexural strength of this kind of beam-column joint, we tested two types of specimens: 1) the first story column is extended toward outside, and 2) the first story column is extended toward inside. Each specimen consists of first story column, second story column, second story wall panel, and boundary beam. For each type of specimen, additional specimens reinforced by diagonal steel reinforcement in the joint or haunch including diagonal reinforcement were constructed. These specimens are used to discuss what is effective to increase the flexural strength of the joint. Test results show that the performances of these two types of joints are different from each other. Considering observed failure modes, a method to evaluate the flexural strength of the joint is proposed. This method is simple enough to be used in structural design and adaptable to both types of beam-column joints.

***Keywords:*** *reinforced concrete; beam-column joint; soft-first story; flexural strength; structural design; failure mode.*

## INTRODUCTION

In Japan, RC buildings with soft-first story for parking lot as shown in Fig. 1 are in high demand. The soft-first story is required to have large strength and ductility because this story will deform much more than upper stories during earthquakes. To satisfy this requirement, the sections of the columns in the soft-first story are often larger than those of upper stories. On the other hand, the failures of the beam-column joints were observed as shown in Fig. 2. It causes the story collapse and the first story column does not show its capacity.

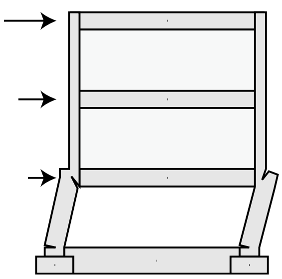

**Fig. 1 Building with soft-first story**

**Fig. 2 Failure of beam-column joint**

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# EXPERIMENTAL STUDY ON STRUCTURAL BEHAVIOR OF CONCRETE-FILLED SQUARE TUBULAR COLUMN TO H-BEAM CONNECTIONS WITHOUT DIAPHRAGM

Masaru TERAOKA[1] and Satoshi SASAKI[2]

## SUMMARY

Structural behaviors of concrete-filled square tubular steel column to steel H-beam connections without diaphragm have been studied by beam-and-column sub-assemblage tests. The sub-assemblages exhibited ductile restoring force characteristics for seismic loading. The yield and ultimate strength of the connections were approximately estimated by the analysis based on yield line theory.

**Keywords**: *concrete-filled square tubular steel column ; steel H-beam ; connection without diaphragm ; sub-assemblage structural test ; performance evaluation for seismic loading ; yield line theory*

## INTRODUCTION

High-rise building structures consisting of concrete filled square tubular columns and steel H-beams without diaphragms in joints, were experimentally investigated to obtain basic information useful for design of beam-to-column connections.

## EXPERIMENTAL PROGRAM

Figure 1 shows the configuration and dimensions of specimens. The test parameters are thickness ($_ct_f$) and material properties of column plates. Both square tubular column and steel H-beam were built up by $CO_2$ gas-shielded arc welding. Column's corner seam(a) was welded with *partial penetration weld* and column wall-to-beam flange was welded with full-penetration weld. The mechanical properties of the materials are shown in Table 1. Anti-symmetric incremental cyclic reversal loading were applied at both ends of beam keeping axial load (N) on column as shown in Fig.1.

Table1 Mechanical Properties of Materials

(a)Steel

| Plates | t (mm) | $\sigma_y$ (N/mm$^2$) | $\sigma_u$ (N/mm$^2$) | e (%) |
|---|---|---|---|---|
| PL-32 | 31.54 | 350 | 525 | 32.1 |
| PL-28 | 27.51 | 312 | 520 | 30.1 |
| PL-25* | 25.22 | 432 | 571 | 25.1 |
| PL-16 | 15.51 | 319 | 520 | 28.1 |
| PL-9 | 8.6 | 358 | 534 | 26.7 |
| Weld | SM490A | 378 | 501 | 37.1 |
| Metal | SM570Q | 510 | 611 | 29.5 |

t : Actual Thickness : $\sigma_y$ : Yield Point : $\sigma_u$ : Tensile Stremgth
e : Elongation : *HT590,all others SM490A

(b)Concrete(Normal-weight)

| $\sigma_B$ (N/mm$^2$) | $\varepsilon_B$ (%) | $\sigma_{st}$ (N/mm$^2$) |
|---|---|---|
| 34.3 | 0.198 | 3.25 |

$\sigma_B$ : Compressive Strength : $\varepsilon_B$ : Strain at $\sigma_B$
$\sigma_{st}$ : Split Tear Strength

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# ANALYTICAL INVESTIGATION OF SEISMIC PERFORMANCE OF WUF-W CONNECTIONS TO INCLINED COLUMNS

Yong-Wan KIM[1], Taejin KIM[2], Dae-Eon KANG[3], and Jong-Ho KIM[4]

## SUMMARY

Unlike the conventional steel moment frames, made up of horizontal beams and vertical columns, the complex-shaped frame having inclined columns has a different flow mechanism of forces because of its geometrical characteristics. Especially, at the beam-to-inclined column connection, the most distinguished differences in the structural behavior such as local buckling and fractures are expected. Also, those expected failure modes are crucial in the special steel moment resisting frames (SMRF), so it is worth looking into the behavior of WUF-W connection which is prequalified by AISC(AISC, 2010). In 2008, Kim conducted a comparative experimental study on the WUF-B and WUF-W connections using SM and SHN steels (Kim et al. 2008). Therefore, this paper introduces the result of the test done by Kim in order to address the superior seismic performance of WUF-W connection. Regarding the analytical investigation of the behavior of steel inclined column connections, the finite element method was used and the analysis tool was calibrated by Kim's experiment data. For the analysis, drift angle, beam local buckling and fracture potential were examined. Finally, this paper provides the comparison between the normal beam-column connection and the inclined column connection in terms of the seismic performance.

***Keywords**: SEEBUS; complex-shaped, moment-resisting frame, inclined column, finite element method, SHN, WUF-W*

## INTRODUCTION

Since it is a popular trend all over the world that the recently designed buildings have complex and irregular features, a demand of studies on the behavior of twisted, tilted and tapered buildings has been arisen. In architect's and engineer's point of view, to form such irregularities in the structures, it is necessary to disassemble the orthogonality in the frame so nowadays the inclined columns are used frequently. However, comparing to the conventional structures, the researches on the behavior of beam-column connection having an inclined column in the region of high seismicity are insufficient.

In the region that has a high potential of large earthquakes, steel special moment frame is used to achieve the ductility demand of the structure. After Northridge earthquake in 1994, a lot of researches on the steel moment connections have been conducted by SAC Joint Venture (El-Tawil et al. 1998).Lu proved that the moment connection behavior is affected by the geometry of weld access holes, toughness of weld metal, and panel zone deformation control (Lu et al. 2000). Also, El-Tawil analytically assessed the strength and ductility of welded unreinforced flange-bolted web(WUF-B) connection, applying stress and strain indices(El-Tawil et al. 1998).Based on the numbers of researches including these, Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications provides the standard design criteria of moment frames (AISC, 2010). Proceeding from such background researches, Kim(Kim et al. 2008) proved the superior performance of welded unreinforced flange-welded web(WUF-W) connection, which is a suggested connection detail for SMRF in Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, by experiments using SM and SHN steels.

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# LOCAL BUCKLING AND INELASTIC BEHAVIOUR OF I-SHAPED BEAMS FABRICATED FROM HIGH STRENGTH STEEL

Cheol-Ho LEE[1], Kyu-Hong HAN[2], Dae-Kyung KIM[3], and Chang-Hee PARK[4], Chia-Ming UANG[5]

## SUMMARY

Flexural tests on full-scale I-shaped beams, built up from high-strength steels (HSB800 and HSA800) with a nominal tensile strength of 800 MPa, was carried out to study the effect of flange slenderness on flexural strength and rotation capacity. The primary objective was to investigate the appropriateness of extrapolating current stability criteria (originally developed for ordinary steel) to high-strength steel. The performance of high-strength steel specimens was very satisfactory from the strength, but not from the rotation capacity, perspective. The inferior rotation capacity of high-strength steel beams was shown to be directly attributable to the absence of a distinct yield plateau and the high yield ratio of the material. When a higher rotation capacity is required as in plastic design, the testing clearly showed that high-strength steel beams were vulnerable to brittle fracture when full-height transverse stiffeners were welded to the tension flange in the plastic hinge region. Residual stress measurements reconfirmed that the magnitude of the residual stress is almost independent of the yield stress of the base metal.

***Keywords****: high strength steel; local buckling; width-thickness ratio; rotation capacity; tensile fracture; full-scale test.*

## INTRODUCTION

Constructional steels with a yield stress higher than 450 MPa (65 ksi) are often called as high-strength steel. Flexural tests of steel beams made of high-strength steel with a yield stress of 690 MPa or greater (ASTM A514) date back as early as the late 1960s (McDermott 1969). The A514 steel has not been popular in civil engineering applications because of the difficulties in weldability, high cost, and insufficient deformation capacity in the form of shapes when used for beams and girders (Galambos et al. 1997; Bjorhovde 2004). However, because of technological advances in steel making nowadays, high-strength steels can be economically produced by the thermo-mechanical control process (TMCP), and can provide good weldability and notch toughness. The benefits of high-strength steels combined with economical steel making have stimulated a great interest in developing high-strength steels for use in building and bridge applications in Korea and other countries.

Recently two types of high-strength steels, HSB800 and HSA800, with a nominal tensile strength of 800 MPa are under development in Korea for bridge and building applications, respectively. Table 1 summarizes the target material specifications for both steels. HSA800 has a tighter control on material properties as it specifies an upper limit on the yield ratio (0.85) as well as tensile strengths. Carbon equivalent to ensure weldability is comparable.

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# EXPERIMENTAL BEHAVIOR OF IN-PLANE BUCKLING BRACES

Cheng-Chih CHEN[1] and Wei-Chien TANG[2]

## SUMMARY

In order to achieve in-plane buckling of the braces while subjected to compression, and to study the hysteretic behavior and axial strength of the braces, the brace sections at both ends are reduced by cutting the flanges to form plastic hinges at both ends. Finite element analysis was conducted to elucidate the effects of the design parameters on the brace behavior. On the basis of the numerical results, four specimens were designed and tested to validate the hysteretic behavior of the braces. The test results demonstrated that all specimens attained typical hysteretic behavior, achieving 4 to 5% rad of interstory drift angle, buckling in-plane, and rupture at the brace mid-length. As verified from the test results, the braces developed some strain hardening when subjected to tension. Moreover, the plastic flexural moment developed in the reduced section at both ends of the braces resulted in higher compressive forces than the buckling strength calculated based on pin-ended compression member. In summary, the braces designed by reducing section at both ends can achieve in-plane buckling and possess typical inelastic behavior.

***Keywords**: special concentrically brace frame; buckle in-plane; gusset plate; reduced section.*

## INTRODUCTION

Specially concentrically braced frames (SCBF), consisting of diagonal braces, are one of the effective earthquake-resisting frames used in high seismic areas. The braces are located between two work points which are intersection points of two centerlines of the beam and the column. The braces used in the SCBF are usually designed via a gusset plate to join the beam and the column, and the braces are expected to provide significant inelastic deformation capacity through yielding in tension and buckling in compression. The behavior of concentrically braced frames have been extensively studied (Whitmore 1952; Ghanaat 1980; Astaneh-Asl and Goel 1985; Black et al. 1980; Popov et al. 1976; Thornton 1984; Tremblay 2001; Tremblay et al. 1996). Various experimental studies have been carried out to investigate the cyclic behavior of the braces having sections such as pipe, rectangular tube, angle, and channel. However, experimental research for braces having H-shaped cross section is very few (Lee and Bruneau 2005; Chen and Lin 2011) although they are also used in practice. Through the use of the gusset plate, the braces generally deform significantly out-of-plane when the braces buckle. The out-of-plane movement may cause failure of non-structural elements such as wall partitions and cladding. Astaneh-Asl and Goel (1984) studied the cyclic behavior of double-angle buckled in-plane. Lien (2009) and Ao (2010) used a knife plate between the brace and the gusset plate to attain the brace buckled in-plane by developing rotation of the knife plate. This research was conducted experimentally to design a brace connection to achieve in-plane buckling and, further, to explore the cyclic behavior and strength of the braces, having H-shaped cross section, designed to have reduced section by cutting the flange at both ends of the brace.

## EXPERIMENTAL PROGRAM

### Test specimens

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# INTERNAL FLANGE STIFFENED MOMENT CONNECTIONS WITH LOW DAMAGE CAPABILITY UNDER SEISMIC LOADING

Chung-Che CHOU[1], Sheng-Wei LO[2], and Gin-Show LIOU[3]

## SUMMARY

This study presents the seismic performance of steel moment connections using internal flange stiffeners (IFSs) welded at the face of the wide-flange column and inner side of the beam flange. The objective is to develop a steel moment connection to achieve a good seismic performance with low damage capability so the repair cost after a large earthquake is low. The connection can sustain subsequent large seismic loading up to a required drift level without failure. Four large-scale moment connections were tested to validate the cyclic performance. One connection which represented a welded-unreinforced flange-bolted web connection failed before finishing cyclic tests at a drift of 4%. Three IFS moment connections showed excellent performance and low damage in experiencing the AISC seismic load twice up to the target drift of 4%, without strength reduction. The specimens were modeled using the computer program ABAQUS to further verify the effectiveness of the IFS in transferring beam moment to the column.

***Keywords***: *IFS moment connection; low damage; cyclic test; finite element analysis.*

## INTRODUCTION

The widespread damage of welded steel moment connections after the 1994 Northridge earthquake and 1995 Hyogoken-Nanbu (Kobe) earthquake initiated extensive research aimed at improving connection seismic performance. Many traditional steel moment connections, which were fabricated following pre-Northridge construction practices with low notch toughness E70T-4 electrode, show minimal plastic deformation (e.g. 1% drift) before weld fracture at the beam-to-column interface (Lu et al. 2000, Nakashima et al. 2000, Chi et al. 2006, Kim et al. 2008). In addition to using a high notch toughness electrode for connection welds, strengthening or reducing the beam end section are needed for modern moment connections to reach required seismic performance at the target drift of 4% (Engelhardt et al. 1996, Uang et al. 2000, Yu et al. 2000, Chou and Uang 2002, 2007, Kim et al. 2002, Lee 2002). High damage in the beam in expected seismic loading leads to heavy cost for repairing.

Adding a pair of full-depth side plates or separate internal flange stiffeners (IFSs) between the column face and beam flange inner side has been demonstrated as an alternative to achieve good seismic performance of moment connections (Chou and Jao 2010, Chou et al. 2010). Test results showed that the connection experiences very minor beam local buckling (or low damage) during the first AISC cyclic loading test in excess of a 4% drift. The connection requires minor repair and has the capability to sustain the AISC loading again to a drift of 4% without failure, similar to a self-centering moment connection, brace, or column with repeatable seismic performance (Chou et al. 2006, Chou and Chen 2011, 2012, Chou et al. 2012). However, previous studies were focused only on a connection with a steel built-up box column and a wide flange beam. The use of a wide-flange

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# RESPONSE MODIFICATION FACTOR FOR LIGHTWEIGHT STEEL PANEL-MODULAR STRUCTURES*

Sung-Gul HONG[1], Eo-Jin LEE[2]

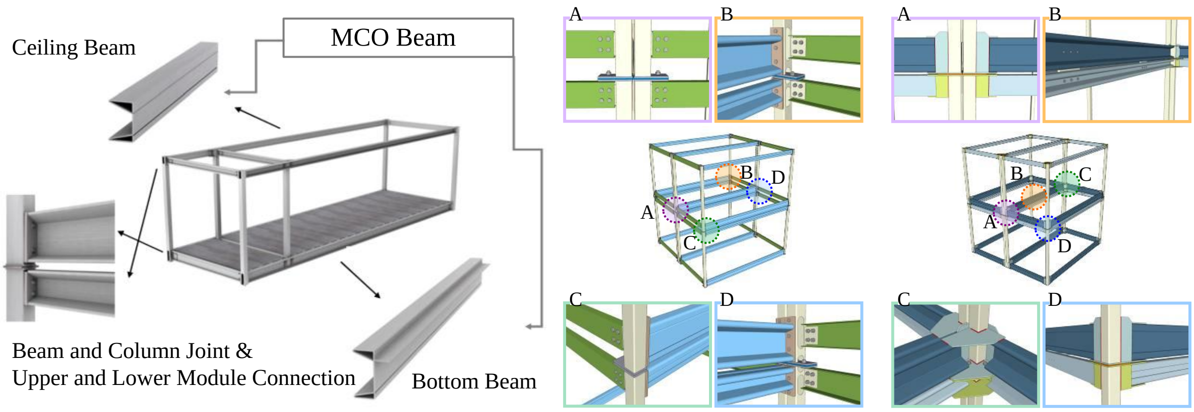
## SUMMARY

This study proposes how to estimate a response modification factor for a lightweight steel panel-modular system which has not been clarified in current building codes. As a component of the response modification factor, an over-strength factor and a ductility factor were drawn from the nonlinear static analysis curves of systems modelled on the basis of performance tests. The final response modification factor is then computed by modifying the previous response modification factor with a MDOF (Multi-Degree-Of-Freedom) base shear modification factor considering the MDOF dynamic behaviours. The results of computations of the structures designed with a dual- frame system, ranging from two-story to five-story structures, produce a value of 4 estimated as the final response modification factor for a seismic design. A value of 5 is considered as the upper limit of the number of stories.

***Keywords****: Response Modification Factor, Lightweight Steel Panel, Modular Structure, Dynamic Analysis*

## INTRODUCTION

In recent years, the efficiency of modular structures has been appraised in terms of the ability to reduce the construction time of the building sites. This is true for projects such as schools, army barracks, and refugee camps. The modular structure applied to this study was composed of MCO beams (Modular Construction Optimized Beams) manufactured by a roll-forming method in order to improve the workability and reduce the cost. From a full-scale test of modular bottom slabs with MCO beams, it was proved that premature local buckling and torsional deformation due to their use of thin plates and their sectional asymmetry could be prevented. For the joints of each MCO beam-column, a bracket connection with rectangular plates welded at the sides of the beam and column was considered to improve the poor capacity of the semi-rigid connections of the end-plate with a minimum number of bolts.




(a) MCO beams (b) End-plate connection (c) Bracket connection

**Figure 1**. Modular frames with MCO beams and types of connection

Although the joints of beam-column and upper-lower modules using MCO beams are welded, the modular structures of frames have insufficient lateral resistance and are expected to have side-sway failure mechanism

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# EVALUATION OF FIELD MEASURED DAMPING RATIO OF TALL CONCRETE BUILDINGS IN KOREA

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**SUMMARY**

It is important to access the damping ratio accurately in the wind design of tall buildings, but the damping ratio is not specified in the Korean Building Codes and thereby the values provided in foreign codes are adopted somewhat arbitrarily by engineers. The significant portion of concrete tall buildings in Korea has the shear wall structure system, and residential buildings have larger gravity load than buildings in other countries due to the use of floor heating system. Therefore, the damping ratio provided in foreign codes may not be directly applied to design of tall buildings in Korea. In this study, the damping ratio of actually built concrete buildings are measured and compared to those provided in foreign codes. It was found that damping ratio of tall concrete buildings decreases as the height of the buildings increases and the measured values are significantly smaller than 2.0% of the ASCE code.

***Keywords***: *Damping ratio, Natural frequency, Natural period, Ambient vibration measurement, System identification*

## INTRODUCTION

The interests in tall buildings have been gained more worldwide recently, and examples of tall buildings can be easily found in Korea (Fig. 1). The tall buildings in Korea are mostly reinforced concrete (RC) buildings and they are also mostly residential buildings unlike those in other countries. Only one of twelve projects in Fig. 1 is office building while the others are all residential buildings. Further, they are all RC buildings including two steel-framed reinforced concrete (SRC) buildings.

It is very important in the design of tall buildings not only to consider earthquake loads but also to check serviceability and safety limit under wind loads. Unlike seismic design, however, the specific value of damping ratio for wind design is not provided in Korea Building Code, KBC 2009 (2009). Therefore, engineers have to choose proper damping values somewhat arbitrarily based on their experiences and the values provided in foreign design codes. The improperly large damping values may underestimate the design wind loads. Further, residential buildings with concrete shear wall system are more popular in Korea than foreign countries, and the heavier gravitational load due to floor heating system in residential buildings results in bigger column or shear wall size than those of foreign countries. Consequently the damping ratio provided in foreign codes may not be directly applied to design of tall buildings in Korea due to these different conditions.

In this paper, the damping ratio of actually built concrete buildings in Korea are measured and compared to those provided in foreign codes. The damping ratio formulae are categorized into 1) single value damping ratio and 2) frequency and amplitude dependent damping ratio, and each formula is compared to the measured damping values for different heights.

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# CO-SHAPE OSCILLATORS FOR THE SEISMIC ANALYSIS OF INELASTIC ASYMMETRIC-PLAN BUILDINGS

Jui-Liang LIN[1], Keh-Chyuan TSAI[2], and Ming-Chieh CHUANG[3]

## SUMMARY

This study proposes a novel oscillator, referred to as the co-shape oscillator (COSO), which can be effectively used in the nonlinear response history analysis (NRHA) of multistory asymmetric-plan buildings. Each COSO, consisting of a rigid mass block connected with two spring-dashpot sets, represents a pair of vibration modes of the original multistory asymmetrical building. When compared with the common single-degree-of-freedom (SDOF) oscillator, the COSO with translation-rotation coupled motion is more suitable to represent the motions of asymmetrical buildings. The issue of reflecting the non-proportionality between the modal translation and the modal rotation existing in the SDOF modal oscillator does not exist in the COSO. The COSO further considers the nonlinear coupling within the associated two vibration modes of the inelastic multistory building. Substituting the governing COSOs for the finite element model of the multistory building substantially improves the efficiency of performing NRHA.

***Keywords***: *asymmetric-plan building; oscillator; spring-dashpot set; modal coordinate; nonlinear response history analysis.*

## INTRODUCTION

The inelastic dynamic analysis, i.e., the nonlinear response history analysis (NRHA), is widely recognized as a more reliable approach for assessing structural seismic performance when compared with the inelastic static analysis approach. Nevertheless, the inelastic static analysis approach is seemingly more popular than NRHA in engineering practice due to its computational simplicity and efficiency. Krawinkler *et al*. (2011) clearly pointed out that simplicity in modeling is at the core of facilitating the use of the NRHA. Han and Chopra (2006) proposed the simplified approach of performing incremental dynamic analysis, which suggested analyzing the first single-degree-of-freedom (SDOF) modal oscillator, instead of analyzing the finite element model of the multistory building. This again highlights that a simplified and effective inelastic structural model is necessary for promoting the NRHA in engineering practice.

Lin and Tsai (2012) proposed the effective one-story building (EOSB) as a vehicle for characterizing the supplemental damping in the non-proportionally damped elastic multistory asymmetrical buildings. For a multistory one-way asymmetrical building, the associated EOSB is constructed from the two-degree-of-freedom (2DOF) modal properties (Lin and Tsai 2007, 2009) of the first translational-dominant and the first rotational-dominant vibration modes of the multistory asymmetrical building. By establishing the relationships between the seismic responses of the multistory asymmetrical building and those of the associated EOSB, Lin and Tsai (2012) satisfactorily applied the elastic response spectra constructed from EOSBs to estimate the peak responses of elastic non-proportionally damped multistory asymmetrical buildings.

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# EARLY STAGE SEISMIC DAMAGE ASSESMENT OF REINFORCED CONCRETE WALLS CONSIDERING CRACK CONDITIONS

Junya YOSHIMURA[1], Susumu KONO[2], Masanobu SAKASHITA[1], Minehiro NISHIYAMA[1], Takuya NAGAE[3], Taizou MATSUMORI[3], Tomohisa MUKAI[4], Masanori TANI[4], Hiroshi FUKUYAMA[4] , and Isao AMANO[5]

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## SUMMARY

Two 40% scale rectangular section wall specimens with different amount of confining reinforcement were tested to study the damage process under static cyclic loading. Crack width was recorded with electronic crack gages and crack condition was quantified at different load levels in terms of number and width of cracks in order to study the correlation between crack condition and damage level. Based on the experimental results, an existing numerical model to predict crack condition was evaluated. Then static test results were compared to crack conditions observed for structural walls of 2010 real-scale four-story reinforced concrete buildings dynamically tested on the E-Defense shaking table in Japan.

***Keywords****: Damage assessment; Crack evaluation; RC structural walls; Real scale dynamic test*

## INTRODUCTION

Performance based design criteria have been slowly spreading for reinforced concrete structures since large-scale earthquakes, such as the 1994 Northridge EQ and the 1995 Kobe EQ, hit major urban cities in 1990s. Researchers made efforts to determine characteristic points such as cracking, yielding, peak, and ultimate points and it has been becoming possible to predict these points with good accuracy. Based on these characteristic points, performance criteria may be determined for different limit states; serviceability, reparability, safety and collapse prevention limit states. However, it is still difficult to accurately simulate damage level which determines retrofit schemes and cost. For concrete members, number, width and length of cracks are important factors to determine early stage seismic damage. However, quantification of cracks is not easy especially for two dimensional members like structural walls since a large amount of meandering cracks emerge. It is even more difficult to utilize crack conditions to describe the damage level.

Two 40% scale structural wall specimens were tested statically by varying amount of confining reinforcement to study the seismic performance under static cyclic loading. Damage states at different loading stages were recorded with displacement gages and high resolution digital cameras. Number and width of cracks were quantified at different load levels. Based on the experimental results, an existing numerical model to predict crack conditions was used to simulate the test results. Then, static test results were compared to crack conditions observed for structural walls of the 2010 real scale four-story reinforced concrete building dynamically tested on the E-Defense shaking table.

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# BOND-STRENGTHING HOOKS FOR RC MEMBERS WITH HIGH STRENGTH SHEAR-REINFORCING SPIRALS

Kil-Hee Kim[1], Young-Seek Kim[2], Min-Jun Kim[2], Dong-Hwan Kim[3] and Sang-Woo Kim[4]

## SUMMARY

The shear strength of reinforced concrete (RC) beams is generally known to be affected by the strength and quantity of shear reinforcements. However, the bond strength of RC members is greatly affected by the amount of shear reinforcements, whereas it is insensitive to the yield strength of the shear reinforcing bars. Therefore, using high-strength shear reinforcements can leads the bond failure of RC beams. In this paper, an experimental study on the bond behavior of RC beams with various steel ratios and yield strengths of shear reinforcements was conducted under cyclic loading to investigate the effective application of high-strength stirrups. The experimental results showed that the specimens with the same transverse steel ratio exhibited similar bond behavior, regardless of the yield strength of shear reinforcements. The test results also confirmed that confining the longitudinal steel bars by the proposed U-shape unclosed reinforcements greatly improved the bond performance of RC beams.

***Keywords**: RC beams; bond; shear; high-strength steel bar; U-shape unclosed reinforcement.*

## INTRODUCTION

As the high-strength concrete develops[1,2], the need to enhance the strength of reinforcing bars, a main component of reinforced concrete (RC) structures, is steadily increasing due to raw material shortages and rising costs. Furthermore, in the case of RC structures heavily affected by lateral load, the closely spaced shear reinforcements can lead to such problems as reduced construction efficiency and economy. To resolve these problems, there has been research on the application of the high-strength reinforcements.[3] However, studies on the high-strength shear reinforcing bars are relatively scarce compared to those on the high-strength flexural reinforcement due to the complex mechanism of shear failure. In particular, research on the effect of high-strength reinforcements for the bond performance of the RC beams is virtually rare.

An effective way to prevent the bond failure of RC members is to increase the amount of shear reinforcement; in general, this is done by reducing the spacing of transverse reinforcements or applying sub-ties to confine the inner longitudinal reinforcements. However, unfortunately, such methods can lead to overcrowded reinforcement and degraded construction efficiency. Therefore, in this study, U-shape unclosed reinforcements are proposed to enhance the bond strength of the RC beams with high-strength shear reinforcements. Furthermore, the applicability of the proposed method is verified by experimentally evaluating the bond strength of RC beams subjected to cyclic loading.

## EXPERIMENTAL PROGRAM

### Test specimens

The mix design strength of the concrete used in this study was 30 MPa. The maximum size of the coarse aggregate was 20 mm, and the slump of the concrete was 150 mm. To obtain the physical properties of the

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# AXIAL ELONGATION OF REINFORCED CONCRETE MEMBERS

Jung-Yoon Lee[1] and Jongwook Park[2],

## SUMMARY

The longitudinal axial strain in the plastic hinge regions of RC columns has a significant influence on the behavior of RC structures subjected to reversed cyclic loading. After reinforcement yields in tension because of reversed inelastic cyclic loading, the flexural cracks remain open, increasing the elongation of the plastic hinge. This strain affects energy dissipation in the hysteretic response by causing the sliding along interconnecting wide flexural cracks. In addition, the strain also influences on the deformability of RC structures. This paper investigated the effects of the axial force on the elongation of a RC member by using a sectional analysis of RC members. In order to compare to the analytical results, RC members were tested. The analytical and experimental test results of the longitudinal axial strain of RC members subjected to various axial loads indicated that the longitudinal axial strain of columns was quite different to that of beams because of axial force. The axial force ratio reduced the longitudinal axial strain of RC members.

***Keywords****: RC members; plastic hinge; axial elongation; axial force ratio*

## INTRODUCTION

The load carrying capacity and energy dissipation behavior of columns in post yield range of deformation has a significant role to prevent a total collapse of reinforced concrete (RC) buildings. The longitudinal axial strain in the plastic hinge regions of reinforced concrete columns influences on the behavior of RC structures subjected to reversed cyclic loading. This strain affects energy dissipation in the hysteretic response by causing the sliding along interconnecting wide flexural cracks. In addition, the strain also reduces the effective compressive strength of cracked concrete of the RC members dominated by shear action.

Special care is needed when a plastic hinge forms in column at the base of the structure subjected to seismic loading because of its relatively short-span-to-depth ratio which is normally less than 3 [1, 2]. After flexural yielding, a plastic hinge develops at the bottom of the column, followed by yielding of the lateral reinforcement or crushing of the diagonal compressive concrete struts in the plastic hinge regions, which led to a sudden failure of the column. The deformability of this column is relatively small when compared to the deformability of column with a larger span-to-depth-ratio.

The longitudinal axial strain in the plastic hinge regions of RC columns has a significant influence on the behavior of RC structures subjected to reversed cyclic loading [1, 2]. After reinforcement yields in tension because of reversed inelastic cyclic loading, the flexural cracks remain open, increasing the elongation of the plastic hinge. This strain affects energy dissipation in the hysteretic response by causing the sliding along interconnecting wide flexural cracks. In addition, the strain also influences on the deformability of RC structures. Although there are some studies regarding the longitudinal axial strain of RC beams subjected to reversed cyclic loading, limited studies of the RC columns regarding the axial strain are available. Fenwick et al. [1] indicated that when the RC members undergo cyclic displacements corresponding to the structural displacement ductility of four to six, the elongation of each hinge is about two to four percent of the beam depth. Lee and Watanabe [2] tested twelve RC beams subjected to reversed cyclic loading. They proposed a model to predict the axial strain in the plastic hinge regions of RC beams. The longitudinal axial strain in the plastic hinge regions of the RC beams rapidly increases after flexural yielding. Furthermore, the longitudinal axial strain of RC beams subjected to

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# PLASTIC HINGE LENGTH OF CORRODED REINFORCED CONCRETE BEAMS

Yu-Chen OU[1], and Nguyen-Dang NGUYEN[2],

## SUMMARY

Existing plastic hinge length formulations for reinforced concrete members do not consider the effect of steel reinforcement corrosion. The objective of this study is to investigate the effect of reinforcement corrosion on the plastic hinge length of reinforced concrete beams. A non-linear finite element analysis method was developed in this research capable of analyzing the mechanical behavior of corroded reinforced concrete beams. The finite element method considers the effects of corrosion by incorporating (1) softening of concrete in compression due to corrosion cracks; (2) reduction of steel area due to uniform corrosion; (3) decrease of the steel yield and ultimate strengths, and ultimate strain due to pitting corrosion; and (4) modification of the bond constitutive law between corroded steel bars and concrete. The method was verified by published experimental results. A parametric study was then carried out to examine the influences of concrete compressive strength, longitudinal tension reinforcement ratio, shear span, and corrosion level on the plastic hinge length of reinforced concrete beams. The plastic hinge length was determined by using the tip displacement obtained from the beam finite element model and the yield and ultimate curvatures derived from sectional analysis of the beam. Bond reduction was considered by modifying the steel stress-strain relationship in the sectional analysis. Results of the study showed that, the plastic hinge length does not have a significant correlation with concrete compressive strength and longitudinal tension reinforcement ratio, but positively correlates with the shear span for both uncorroded and corroded beams. Moreover, the plastic hinge length decreases as the increase of the corrosion level. A theoretical proof was developed to explain this phenomenon. Furthermore, a simplified expression was proposed to incorporate reinforcement corrosion into existing plastic hinge length models.

***Keywords***: *Reinforced concrete beams; Corrosion; Finite element analysis; Plastic hinge length.*

## INTRODUCTION

Corrosion of reinforcing steel, induced by chloride penetration or carbonation attack, is the most common deterioration problem faced by reinforced concrete structures. Carbonation attack tends to produce uniform reduction of cross-sectional area (uniform corrosion). On the other hand, chloride penetration tends to cause uneven reduction of cross-sectional area (pitting corrosion). Both types of corrosion decrease the load-carrying capacity of the reinforcement while the latter type of corrosion also reduces the ductility of the reinforcement as a result of stress concentration at pitting locations. Expansion of corrosion products causes internal pressure inside the concrete which when exceeds the tensile strength of concrete, leads to cracking and spalling of cover concrete. This reduces the bond capacity between reinforcement and surrounding concrete and the load-carrying capacity of the member. Figure 1 shows examples of steel reinforcement corrosion in two different reinforced concrete buildings in Taiwan. Examination of the corroded steel reinforcement from the buildings revealed that the corrosion weight loss is more than 50%.

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# RESPONSE OF OLD-TYPE RC COLUMNS UNDER SEISMIC LOADING

Bing LI[1] and Thanh Phuong PHAM[2]

## SUMMARY

This paper presents an experimental investigation carried out on nine old-type concrete columns reinforced with plain longitudinal reinforcing bars subjected to seismic loadings. The variables of interest are axial loads, aspect ratio and cross-sectional shapes. The performance of test specimens was analyzed in-depth in terms of crack patterns, hysteretic response, initial stiffness, shear strength and drift capacity. The test results were compared with existing model for evaluation of existing structures. It is shown that ASCE/SEI 41-06 overestimates initial stiffness, shear strength but substantially underestimates drift capacities of columns plain bars. A correction factor is recommended for shear strength evaluation of RC columns with plain longitudinal reinforcement.

***Keywords***: *reinforced concrete; column; axial failure, plain bar, seismic loading.*

## INTRODUCTION

There has been increasing emphasis in many countries on seismic assessment of existing structures designed to pre-1970s seismic codes when plain reinforcing bars were commonly used to reinforce concrete members. Earthquake observations showed that numerous buildings suffered severe seismic damage attributable to the use of plain longitudinal reinforcement in columns. Strong effort has been devoted to further understand the seismic behavior of such RC columns. This knowledge represents an essential background for structural evaluation of existing structures, but refers generally to concrete constructions reinforced with deformed bars.

There are fewer studies on the response of columns reinforced with plain round bars compared to those on columns built with deformed bars. Verderame *at el.* (2009) conducted a test of twelve columns to assess the sources of deformation related to poor bond performances of smooth bars. The test results showed that the main difference in the deformation mechanism between columns with smooth bars and deformed bars was the extensive yielding of plain bars in column-foundation interfaces. Yalcin *et al.* (2008) reported the experiment results of two columns reinforced with plain bars in comparison with columns retrofitted with carbon fiber reinforced polymer (CFRP). Ozcan *et al.* (2008) tested five column specimens with plain bars focusing on the effectiveness of CFRP wrapping retrofit technique. Arani *et al.* (2010) conducted a test of four columns reinforced with plain bars and also found out that slip (fixed-end rotation) contribution was the major source of deformation. Effectiveness of strengthening methods on behavior of concrete columns with smooth bars was investigated by Bousias *et al.* (2007) through CFRP wrapping and RC jacketing methods. The reference specimens (un-retrofitted specimens) had low deformation capacity and energy absorption capacity which was independent from the overlapping splice length. Among all of the studies reviewed, none of the tests under double curvature bending was conducted so that one of important aspects which seldom occur in cantilever tests tends to be ignored especially splitting bond failure. There were no studies systematically investigating the

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# EFFECTS OF ADDED VISCOUS DAMPING ON SEISMIC RESPONSE OF MASONRY-INFILLED FRAMES

Park, Ji-Hun[1]

## SUMMARY

Response reduction effects of added viscous damping on masonry-infilled frames are investigated in this paper. Simplified models composed of an inelastic SDOF system and a linear dashpot are adopted for nonlinear analysis. Effects of period lengthening due to stiffness and strength degradation are taken into account. Nonlinear time history analysis results are compared with responses predicted by conventional damping correction factors and corresponding yield strength reduction factors for SDOF systems without added damping. Errors from conventional damping correction factor are unacceptably high to be used for the purpose of structural design, and alternative methods need to be developed.

***Keywords:*** *masonry-infilled RC frame; damping system; nonlinear time history analysis; SDOF system*

## INTRODUCTION

Many reinforced concrete buildings, of which lateral force resisting systems are the moment frames, have masonry infill walls due to architectural requirements. The masonry infill walls have significant effects on the structural behavior of those RC moment frames by adding stiffness and strength, which degrade in a brittle manner due to the material characteristics of the masonry infill and out-of-plane action caused by inertial forces (ASCE. 2000, ATC. 1997, Paulay T et al. 1992). Although the masonry infill walls fail, the remaining RC frames should be able to resist the seismic demands. If the capacity of the remaining RC frames is insufficient, damping systems can be applied in order to reduce the seismic demands. However, in the design of the damping systems for masonry-infilled RC frames, designers need to take into account the increase and degradation of stiffness and strength, which affect the added damping ratio significantly by shifting the effective stiffness.

In this study, the seismic response of the masonry-infilled RC frames with damping systems are investigated through extensive nonlinear time history analyses of simplified models. The masonry-infilled RC frames are modeled with nonlinear SDOF systems composed of multiple springs that represent hysteretic characteristics of the masonry infill walls and RC frames, respectively. The damping systems are modeled by adding a linear dashpot to the nonlinear SDOF systems for the masonry-infilled RC frames. Nonlinear time history analyses of the SDOF systems with and without damping systems are conducted for a set of simulated ground motions, and compared to investigate response reduction efficiency of the damping systems for the masonry-infilled RC frames. In addition, influence of various attributes of the SDOF systems on seismic response reduction efficiency is investigated as well. Using the same response data, the applicability of the conventional damping correction factors to the masonry-infilled RC frames with damping systems is evaluated by interpolating ductility demands of the SDOF systems without damping systems for reduced seismic demands.

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# ANALYTICAL CRACKING RESISTANCE AND INITIAL STIFFNESS OF CONFINED MASONRY WING-WALLS

Yi-Hsuan TU[1], Hsing-Yu YEH[2], and Tsung-Hua CHUANG[3]

## SUMMARY

An analytical model for the seismic behavior of confined masonry (CM) wing-walls is proposed in this paper. CM wing-walls are the panel segments between openings and RC tie-columns. Based on experimental results of seven full-scale CM wing-walls, the model describes a bilinear relationship of the load-displacement envelope curve. This paper focuses on the elastic section defined by the cracking load and initial stiffness in the envelope curve and presents the rationale in setting up the analytical factors.
The cracking load is estimated with the diagonal tension strength according to FEMA 356. The initial stiffness is estimated by considering CM wing-walls as diagonal compression struts. The compression strut analogy indicates that CM walls are subjected to additional axial compression induced by the lateral loading. Since the axial stress plays an important part in the diagonal tension strength estimation, this paper proposes a method to derive the additional axial force from the lateral load. The distribution of lateral force between the masonry wall and tie-column is also examined, so the contribution of tie-columns can be included in the strength estimation.
The proposed model has been validated with the experimental data. The model shows a conservative estimation for both cracking load and displacement, but the comparison with experimental load-displacement curves indicates a good fit.

***Keywords****: confined masonry; seismic behavior; lateral strength; load-displacement relationship.*

## INTRODUCTION

Confined masonry (CM) consists of masonry wall panels and cast-in-place reinforced concrete (RC) confining elements. The confining elements, including tie-columns and tie-beams, are built after the masonry panels. Another difference from RC frame with masonry in-fills is that the vertical edges of the panels adjacent to tie columns are usually toothed. After the tie columns are cast, the toothed edges become shear keys and integrate the masonry panels and tie columns into a composite member. This confining effect improves both in-plane and out-of-plane performance of the masonry panels. It makes CM a relatively economical and efficient choice for low-rise buildings in countries and regions with high seismic risk.

CM buildings are widely used in South Europe, Latin America, and Asia (EERI & IAEE, 2012). The confining elements usually have smaller cross-sectional dimensions than the beams and columns in a RC frame building (Brzev, 2007). The Seismic Design Guide for Low-Rise Confined Masonry Buildings (Meli & Brzev, 2011) suggests a minimum tie-column/beam size of 150mm $\times$ t with minimum 4-#3 reinforcing bars, where t denotes the wall thickness. The CM construction has also been used in Taiwan for decades. However, the difference between CM and RC buildings was not well known until recently. The local engineers tend to use CM construction way for masonry panels in a RC frame building. In this case, the column and beam sections have larger dimensions. Sometimes the lateral resistance contribution of masonry panels is neglected in the design for

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# ANALYTICAL MODEL FOR STRENGTH AND STIFFNESS PREDICTION OF BRICK MASONRY INFILL

Maidiawati[1], Yasushi SANADA[2], Thandar OO[3]

## SUMMARY

An analytical model for strength and stiffness prediction of brick masonry infill is proposed. Contribution of masonry infill in a frame structure is analytically evaluated by replacing the masonry panel by an equivalent compression strut representing a masonry infill by a distributed compression transferred diagonally between infill/frame interfaces. The infill/frame contact length can be determined by solving two equations, i.e., static equilibriums related to the compression balance at infill/frame interface and lateral displacement compatibility. Consequently, the equivalent strut width is presented as a function of infill/frame contact length. The analytical model was validated and verified by comparing to experimental results obtained from a series of structural tests. Experimental specimens included reinforced concrete (RC) bare frame and infilled frame which represented a typical R/C building with brick masonry elements in Indonesia. According to the test results on the bare frame and the infilled frames, the infill contribution was extracted to compare with the promising results by the proposed equation. Good agreements were observed between the experimental and analytical results on lateral strength, lateral stiffness, and ductility.

***Keywords**: SEEBUS; reinforced concrete frame, brick masonry wall, infill/frame contact length, strut width, infill contribution*

## INTRODUCTION

Brick masonry infill used widely in reinforced concrete buildings, particularly in developing countries with high seismicity. However, this infill is not usually considered in current design practice, assuming it to be a nonstructural element. It has been obvious from several past studies that a masonry infill has significant effect on strength and stiffness. Therefore, they should not be neglected in the analysis and design of a structure against lateral load. Analytical and experimental past studies of the authors also showed that a masonry infill contributes significantly to both strength and stiffness of this kind of structure (Maidiawati et al. 2008 and 2011). Contribution of masonry infill in a frame structure is analytically evaluated by replacing the masonry panel by an equivalent compression strut made of the same material of the infill. In last few decades, several studies have been carried out to define the effective width of equivalent diagonal strut for prediction the strength and stiffness of masonry infill as reported by Holmes (1961), Smith and Carter (1996), Mainstone (1971), Pauley and Priestley (1992), El-Dakhakhni (2004) and P.G Asteris (2008). Most also focused on infill/frame contact lengths when discussing interactions between the infill and its surrounding frame. This study proposes an alternative method for determining infill/frame contact length with a simplified equation.

In this study, a masonry infill is replaced by a diagonal compression strut, which represents a distributed compression transferred diagonally between infill/frame interfaces. The infill/frame contact length can be determined by solving two equations, i.e., static equilibriums related to the compression balance at infill/frame

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# DAMAGE ASSESSMENT OF SCALED-DOWN TWO-STORY REINFORCED CONCRETE SCHOOL BUILDING MODELS

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## SUMMARY

This study presents a damage assessment scheme for seismically-excited torsion-coupled nonlinear buildings based on the normalized relative displacement vibration shape (NRDVS). In order to investigate the applicability of proposed NRDVS, the relative displacements are obtained from a real-time structural response integrator (RTSRI) by integrating the relative accelerations simulated from multiple-degree-of-freedom nonlinear models. As a result, the normalized inter-story drift ratio (NIDR) calculated by differencing the normalized displacements of the two successive floors of NRDVS and dividing by the story height can reveal the damaged floors as well as the location of damaged building members. Furthermore, the proposed damage assessment scheme is validated by observing the experimental measurements of shaking table tests of three scaled-down two-story reinforced concrete school building models conducted at National Center for Research on Earthquake Engineering (NCREE), Taipei, Taiwan.

***Keywords***: *system identification; structural damage assessment; normalized relative-displacement vibration shape; normalized inter-story drift ratio; reinforced concrete school building.*

## INTRODUCTION

Damage indices that indicated the location of damage have been used basically can be classified into vibration response-based and structural parameter-based indices. The vibration response-based indices used the structural response measurements in a single earthquake event and from that the damage-related physical parameters, such as peak acceleration, peak velocity, etc. can be calculated (Park and Ang 1985; Elenas and Meskouris 2001). The structural parameter-based indices estimated the change of structural parameters based on a set of independent earthquake events: before and after the damage occurrence. To apply the structural parameter-based indices, the structural parameters are required to remain time-invariant during those events. Observing the change of modal frequency to detect damage is popular in structural health monitoring systems because damage is always related to a reduction of stiffness as well as modal frequency. However, damage in different locations may lead to different frequency changes in various modes. Therefore, it remains difficult to determine the damage location by simply investigating the changes of modal frequencies. According to the modal-superposition assumption of linear system, the mode shape is a good indicator among location-related parameters. Many researchers have attempted to adopt mode shape-based indices. All above indices are based on the important assumption of keeping linear time-invariant before and after the occurrence of damage (Lin et al. 2006). When structural elements are damaged subjected to a severe earthquake, the overall capacity of stiffness may be significantly reduced. Hence, the structural damage can be reflected by the changes of parametric values of the damaged element. Frequency-domain algorithms involve averaging temporal information (Brincker et al. 2001), thus discarding most of their details. For the identification of structural parameters' variations due to damages, time domain analyses are used extensively. Over the past decades, a few time domain methods have been transferred

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# SENSITIVITY STUDIES FOR SEISMIC PERFORMANCE ASSESSMENT OF SAFETY-RELATED NUCLEAR STRUCTURES

Yin-Nan HUANG[1]

## SUMMARY

Sensitivity studies are used to identify the impact of the following factors on the seismic vulnerability of nuclear power plants (NPPs), namely: 1) the description of fragility curves for primary and secondary components of NPPs, and 2) the number of response simulations required for performance assessment. The studies include a series of intensity-based performance assessments of a sample NPP. The studies illustrate the utility of the response-based fragility curves and the inclusion of the correlation in the responses of NPP components directly in the risk computation. The analysis results indicate that the use of mean fragility curves for NPP components and a demand-parameter matrix of 1000+ row vectors can provide an unbiased estimate of the mean probability of unacceptable performance for the NPP with high confidence.

***Keywords****: nuclear power plant; performance assessment; fragility curve; seismic response; spread of plasticity.*

## INTRODUCTION

Huang et al. (2010, 2011a) proposed a five-step methodology for the probabilistic seismic performance assessment of nuclear power plants (NPPs). The methodology computes the annual frequency of unacceptable performance using a seismic hazard curve and a series of *intensity*-based assessments.

Step 1 of the methodology proposed by Huang et al. (2010, 2011a) performs plant system analysis to determine accident sequences that could contribute to the target unacceptable performance and develops component fragility curves as a function of a structural response parameter. Step 2 develops the seismic hazard curve(s) for the NPP site and selects and scales ground motions for each intensity level. Step 3 identifies the distributions and correlation of all structural response parameters of Step 1 using nonlinear response-history analysis at each intensity level. Step 4 uses Monte-Carlo-based procedures to generate a significant number of response data that are statistically consistent with those of Step 3 and to assess the possible distribution of damage to structural components and secondary components and systems of the NPP for each set of simulations. Step 5 computes the probabilities of unacceptable performance at each intensity level and the annual frequency of unacceptable performance of the NPP subjected the seismic hazard of Step 2.

Two sets of sensitivity studies were conducted using the methodology proposed by Huang et al. (2010, 2011a, 2011b). Results are summarized here to answer the following two questions:

1. What is the impact on the annual frequency of unacceptable performance of using a mean or median fragility curve instead of a family of fragility curves for NPP components?
2. How many sets of response simulations are required to compute a reliable estimate of the seismic vulnerability of a NPP?

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