THEORETICAL MODELING OF PRESTRESSED CONCRETE MEMBERS SUBJECTED TO SHEAR CONSIDERING BOND STRENGTH OF PT TENDONS

Masanori TANI[1], and Minehiro NISHIYAMA[2]

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SUMMARY

A set of design equations based on the strut and truss mechanisms analogous to a strut-and-tie model is proposed in this paper, which can predict the shear strength of prestressed concrete members. Yielding of longitudinal mild steel reinforcement and bond strength of PT tendon are taken into account. Test results from 119 prestressed concrete beams and columns under anti-symmetrical loading found in the past literature are compared with the results evaluated by the proposed design equations. The comparison reveals they give a better and more accurate estimation of shear strength than the current design equations of Architectural Institute of Japan.

***Keywords**: prestressed concrete, ultimate shear strength, truss mechanism, strut mechanism, bond strength.*

INTRODUCTION

The standard for design and construction of prestressed concrete structures published by Architectural Institute of Japan (AIJ 1998) specifies a couple of design equations for ultimate shear strength of prestressed concrete members. One is empirical and based on a summation of contributions from concrete, transverse reinforcement, and compressive force in concrete due to prestressing force and/or axial load as shown in Eq. (1). The other is based on the strut and truss mechanisms analogous to a strut-and-tie model and is shown in Eq. (2).

$$V_u = \{\alpha(f_s + 0.1\sigma'_g) + 0.5\sigma_{wy}(p_w - 0.002)\} b \cdot j \tag{1}$$

where

$$f_s = 7.5 + 1.5/100 f'_c \ (f_s \le 1.65\text{MPa}), \quad \alpha = \frac{4}{L/(2 \cdot d_r) + 1} \quad (1 \le \alpha \le 2)$$

$$V_u = bj_t p_w \sigma_{wy} + \frac{bD}{2}(\nu f'_c - 2 p_w \sigma_{wy}) \tan\theta \tag{2}$$

where

$$\tan\theta = \sqrt{(L/D)^2 + 1} - L/D, \quad \nu = \alpha L_r \left(1 + \frac{\sigma_g'}{f'_c}\right) \quad (0.65 \le \nu \le 1),$$

$$\alpha = \sqrt{60/F_c} \quad (\alpha \le 1), \quad L_r = \frac{L}{4D} \quad (L_r \le 1)$$

where V_u = ultimate shear strength, f_s = concrete shear strength, σ'_g = average compressive stress (=$(N + P_e)/bD$), N = applied axial load, P_e = effective prestressing force, σ_{wy} = yield strength of transverse reinforcement ($\le$295 MPa), p_w = transverse reinforcement ratio ($0.002 \le p_w \le 0.012$), b = width of the section, j = lever-arm, d_r = effective depth of tensile reinforcing steel bars, j_t = distance between the tensile and compressive longitudinal reinforcement, f'_c = concrete compressive strength, ν = reduction factor of concrete compressive strength, L =

[1] *Assistant Professor, Graduate school of Architecture, Kobe University, Japan, e-mail: mtani@port.kobe-u.ac.jp*
[2] *Professor, Graduate School of Architecture and Architectural Systems, Kyoto University, Japan, e-mail: mn@archi.kyoto-u.ac.jp*

DEFORMATION PERFORMANCE OF POST-TENSIONED PRECAST CONCRETE COLUMNS FAILING IN FLEXURAL SHEAR

Jaeman LEE[1] and Minehiro NISHIYAMA[2]

SUMMARY

Deformation performance of post-tensioned precast concrete column subjected to reversed cyclic loading simulating earthquake load and failing in flexural shear was investigated. Deformation of the column consists of flexural deformation, R_f, and shear deformation, R_s. Flexural deformation, R_f was further divided into flexural deformation in plastic hinge region, $R_{f.ph}$, and the deformation due to pullout of post-tensioning bar (PT bar, hereafter), $R_{f.pu}$ from beam-column joints or foundations. To evaluate the ultimate deformation, the simplified method based on the flexural shear behavior of post tensioned precast concrete columns was proposed. For investigation of their flexural shear behavior, three half-scale post-tensioned precast concrete columns with high strength concrete in exceedance of 100 MPa and with shear reinforcement in exceedance of 500 MPa were tested. For verification of the proposed method, experimental data from this study and Ref. 2 were used.

Keywords: *post-tension; precast; ultimate deformation; ductile; flexural shear; pullout of reinforcement*

INTRODUCTION

Current seismic design philosophies favor the design of ductile structural systems able to undergo inelastic reversed cycles while sustaining their capacity. Precast concrete structures assembled by post-tensioning can be an alternative to the ductile structural system. The structural system can control damage and enhance the structural deformation performance by concentrating the deformation at beam-column joint interfaces.

To design ductile structural systems using post-tensioning, it is necessary to prevent structural members from brittle failure such as shear failure. If shear strength of the members is larger than flexural strength, they should fail in flexure, theoretically.

However, from experimental research on flexural or shear behavior of post-tensioned precast concrete columns in the past, it was observed that even though specimens were designed to fail in flexure such as crushing of concrete or yielding of flexural reinforcement, some of them subjected to reversed cyclic loading failed in shear as deformation increased after flexural yielding (shear failure after flexural yielding, hereafter).[1,2]

According to a theoretical study on the ultimate deformation capacity of reinforced concrete (RC) beams, capacity reduction after yielding of flexural reinforcing bars is closely related to shear capacity of concrete in the flexural compression zone of the member section. When tensile force of the reinforcing bars is maintained as the yield strength after their yielding, shear capacity of the concrete compression zone decreases because the depth of the compression zone decreases to satisfy the force-equilibrium in the cross section.[3] In post-tensioned precast concrete columns, however, it was observed that the capacity reduced as PT bars did not yield and their stress continuously increased[2]. Furthermore, although a prediction method for ultimate deformation capacity of RC columns was proposed by Mostafei[4] based on the MCFT (Modified Compression Field Theory), it may not be applied for the one of precast concrete columns where large deformation due to pullout of PT bar occurs.

In this study, to evaluate the ultimate deformation of post-tensioned precast concrete columns, a simplified method considering pullout of PT bar as well as flexural and shear deformation in the column is proposed. For investigation of flexural shear behavior of post-tensioned precast concrete columns failing in shear after flexural yielding, static loading tests on three half-scale post-tensioned precast concrete columns are conducted. The

[1] *Ph.D Candidate, Graduate School of Architectural Eng., Kyoto University, Japan, e-mail: jaeman.lee@ax2.ecs.kyoto-u.ac.jp*
[2] *Professor, Graduate School of Architectural Eng., Kyoto University, Japan, e-mail: mn@archi.kyoto-u.ac.jp*

SEISMIC PERFORMANCE OF LARGE-SCALE REINFORCED CONCRETE BEAMS WITH CORRODED STEEL REINFORCEMENT

Yu-Chen OU[1], Li-Lan TSAI[2], and Hou-Heng CHEN[3]

SUMMARY

Nine large-scale beam specimens were constructed. Of which, one was used as the control, while the other eight ones were divided into four sets. Each set had two specimens and was subjected to accelerated corrosion using an imposed current for the same time interval. Following the corrosion, a specimen in each set was tested using cyclic loading to examine the seismic performance, while the other one was demolished to examine the extent of corrosion. Cyclic loading results indicated that with an increasing corrosion level, the ultimate drift capacity, ductility, plastic rotation capacity, and energy dissipation of the beams initially increased and later decreased. The failure mode switched from flexural failure, largely owing to buckling of the longitudinal reinforcement to flexural-shear failure, which is due mainly to fracturing of the transverse reinforcement. Corrosion increased shear deformation and the spread of plasticity of the plastic hinge region.

Keywords: *Reinforced concrete beams, corrosion, cyclic loading.*

INTRODUCTION

Chloride attack and/or carbonation of concrete makes a reinforced concrete structure prone to corrosion of steel reinforcement after an in- service period of a certain years. Corrosion expands the steel reinforcement, leading to spalling of cover concrete and bond damage. Corrosion also reduces the area of steel reinforcement and creates pitting, subsequently decreasing the load-carrying capability and ductility of a reinforced concrete member. As an island in the Pacific Ocean, Taiwan has its population centers close to the coastline. Consequently, civil engineering structures are prone to corrosion of steel reinforcement due to humid weather and wind-born salts. Figure 1 shows a photograph of corroded beams and slabs in a reinforced concrete building in Taiwan. Significant rust staining was found in the plastic hinge region of one of the beams. Given the location of Taiwan in an earthquake-prone region, corrosion of steel reinforcement in the plastic hinge region can seriously damage the seismic capacity of the beam, ultimately endangering the seismic safety of buildings.

[1] *Assistant Professor, Department of Construction Engineering, National Taiwan University of Science and Technology, Taiwan, e-mail: yuchenou@mail.ntust.edu.tw*

[2] *Master student, Department of Construction Engineering, National Taiwan University of Science and Technology, Taiwan, e-mail: hank740909@hotmail.com*

[3] *Master student, Department of Construction Engineering, National Taiwan University of Science and Technology, Taiwan, e-mail: hktwinshk@msn.com*

Punching Shear Strength and Deformation Capacity of Fiber Reinforced Concrete Slab-Column Connections

Min-Yuan CHENG[1]

SUMMARY

The use of discontinuous concrete fiber reinforcement as an alternative to headed shear studs for increasing punching shear strength and ductility of slab-column connections subjected to earthquake motions was investigated. Three experimental stages were conducted in this research. First, ten slabs with different fiber reinforced concretes and steel reinforcement ratios were tested under monotonically increased concentrated load. In the second research stage, two approximately 1/2-scale slab-column subassemblies reinforced with the materials selected from the first testing phase were tested under combined gravity load and uni-axial lateral displacement reversals. In the last research phase, three approximately full-scale slab-column specimens were tested under combined gravity load and bi-axial lateral displacement reversals. Based on experimental results, it is concluded that the use of fiber reinforced concrete with either regular strength or high-strength hooked steel fibers in a 1.5% volume fraction is effective in increasing punching shear resistance and deformation capacity of slab-column connections subjected to combined gravity load and lateral displacement reversals. The limited test results also indicate that design criteria for headed shear studs in the 2008 ACI Building Code may not be conservative.

Keywords: *fiber reinforced concrete; headed studs; slab-column;, punching shear; drift capacity; hooked steel fiber;, bi-axial.*

INTRODUCTION

Reinforced-concrete framed structures that feature slabs supported directly by columns, without the use of beams or girders, are referred to as slab-column framed systems (flat slabs). This type of system offers economical advantages and larger open spaces with reduced story heights compared to framed systems with beams. Therefore, for low and medium-rise buildings, located in regions of low or no seismicity and when gravity loads are not particularly heavy, reinforced concrete slab-column frames represent an attractive alternative to building designers and owners. Also, when combined with special moment-resisting frames or structural walls, slab-column frames may be used in moderate and high seismic regions. However, punching shear failure around slab-column connection is a major concern in the design of slab-column framed systems, particularly when located in regions of moderate to high seismicity because of the combination of gravity- and earthquake-induced shear stresses and deformations. Punching shear failures occur suddenly, without warning and therefore, they need to be prevented. Further, poor shear design of slabs may lead to a progressive collapse when lower floors fail to support the impact loading initiated from collapsed floors above.

In this research, the potential of using fiber reinforced concrete as an alternative for increasing punching shear strength and deformation capacity of slab-column connections is evaluated. Total fifteen slab-column subassemblies were tested in three experimental phases that feature different research objectives. In the first experimental phase, two fiber reinforced concrete materials that offered the best performance for slab-column subassemblies under monotonically increased gravity-type load were selected. In the second experimental phase,

[1] *Assistant Professor, National Taiwan University of Science and Technology, Taiwan, email: minyuancheng@mail.ntust.edu.tw*

FINITE ELEMENT MODELING OF CYCLIC OUT-OF-PLANE BEHAVIOR OF MASONRY WALLS RETROFITTED BY INSERTING INCLINED STEEL PINS

Kshitij C. SHRESTHA[1], Takuya NAGAE[2], and Yoshikazu ARAKI[3]

SUMMARY

The paper is focused on finite element modeling of the out-of-plane behavior of retrofitted masonry walls when subjected to cyclic loading. The retrofit technique involves inserting inclined steel pins in the plane perpendicular to the wall face, already practiced in Japan in several historical masonry structures. This paper presents a finite element model for the masonry walls, where continuum elements represent brick units and interface elements represent brick unit/mortar slip interface, and truss elements represent the reinforcing bars. The proposed finite element methodology is validated by the comparisons with experimental results. The paper also proposes a simplified finite element model where the reinforcing bars are represented by an equivalent vertical bar to stabilize the nonlinear analysis and reduce computational burden. A study to evaluate the sensitivity of numerical results to the modeling parameters has been done to demonstrate not only the stability of the proposed modeling strategy but also the robustness of the retrofitting technique.

Keywords: *masonry; seismic retrofitting; steel pin; out-of-plane cyclic loading; finite element*

INTRODUCTION

Unreinforced masonry, commonly termed as URM, is one of the oldest construction practices performed worldwide. URM walls have been largely found to be prone to failure during high or even moderate intensity earthquake playing significant role in hazard to life safety. Moreover, with the advent of time, historical masonry structures undergo constant structural decay and damage. Vulnerability of historical structures under earthquake excitations has been seen in very recent Italy earthquake accountings (Modena et al. 2010) as well. Thereby, there is a strong need for improving the performance of existing URM structures.

Past research works (Karantoni et al. 1992, ElGawady et al. 2004, Abrams et al. 2007, Ehsani et al. 1999, Willis et al. 2010) done in the field of developing techniques for improving the seismic performance of URM walls involve retrofitting mainly involving: (1) attachment of reinforcing bars, (2) surface treatment, (3) grout injection, (4) post-tensioning and (5) reinforced core technique. The first two techniques mentioned above may change the appearance of URM constructions significantly and may cease their aesthetic value, problematic especially in historical masonry constructions. And the rest though do not cause significant changes to the appearance of the wall, nevertheless require removal of roof and changes to existing foundation which is troublesome. Difficulties associated with preservation of historical masonry buildings, durability of strengthening materials and also restriction on the parts of the building to be damaged make choice of retrofitting technique more challenging.

To overcome above difficulties, in Japan, a fairly effective retrofitting technique, where stainless pins are inserted perpendicular to the plane of the wall, has been applied to several historic brick buildings in practice. The strength of this technique is ease of construction, wherein removal of roof and changes to foundation are unnecessary. This contributes in lower construction cost and shorter construction period. An experimental study on determining effectiveness of such technique has been done by Takiyama et al. (2008) where steel pins are

[1] *PhD Student, Department of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail:se.kshitij@archi.kyoto-u.ac.jp*
[2] *Researcher, Hyogo Earthquake Engineering Research Center, National Research Institute of Earthquake Science and Disaster Prevention, Japan, e-mail:nagae@bosai.go.jp*
[3] *Assoc. Professor, Department of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail:araki@archi.kyoto-u.ac.jp*

EXPERIMENTAL STUDY ON SEPARATING REINFORCED CONCRETE INFILL WALLS FROM STEEL MOMENT FRAMES BY SLITS

Hung-Jen LEE[1], Ruey-Shyang JU[2], and Cheng-Cheng CHEN[3]

SUMMARY

This paper investigate the structural behavior of one-bay by one-story steel moment frames with or without reinforced concrete infill walls subjected to reversed cyclic lateral loading. There were four steel moment frame specimens including one bare frame, one with ordinary RC infill wall, and two with separation slits between RC infill wall and columns. Hysteretic behavior including stiffness, strength degradation, ductility, and energy dissipation capacity were presented and discussed. The test results indicate that adding separation slits at the infill wall boundary can significantly improve the drift capacity and obtain a more sustainable lateral resistance. Based on measured properties and test results, the non-linear hinge parameters of steel frames and RC infill walls were established by iteration. Following same member properties, pushover analysis of several three-bays by ten-story frame models were conducted to evaluate the effect of the separation slits. It was found that the slits could significantly separate the RC infill wall contribution and eliminate the soft-story effect, when the RC infill walls did not continue from top to the ground floors.

***Keywords**: infill wall; steel moment frame; cyclic loading; slit.*

INTRODUCTION

In multi-story moment resisting frame buildings, infill walls are usually made of masonry, reinforced concrete (RC), drywall, or lightweight panel filled with mortar. Apart from bearing walls or shear walls, which are carefully analyzed by engineers, most infill walls are functioned as partition components and regarded as non-structural elements in the moment resisting frame, that is, neither a vertical force resisting element nor a lateral force resisting element. This simplification seems quite reasonable whenever the surrounding frame members are relatively strong with respect to the infill walls. However, from studying the previous investigations of destructive earthquake disasters conducted by Watanabe (1997) and Tsai et al. (2000), the detrimental influence of infill walls may be underestimated in practice.

Prior studies conducted by Canbay et al. (2003) and Lee et al. (2008) showed that infill masonry or RC wall could significantly increase the lateral stiffness and strength of RC moment frames. On the other hand, the behavior of stiff walls infilled moment frame may turn out to be relatively brittle as compared to the identical bare moment frame. The behavior change is due to the significant degradation in stiffness and strength of stiff wall beyond its peak strength. Lee et al. (2008) tested code-compliant ductile RC moment resisting frames with non-structural RC infill wall and showed that a ductile RC moment frame may result in a brittle shear failure as the extensive lateral force released from damaged RC infill wall in a very short process.

There have been numerous experimental studies on steel moment frames with infill masonry or RC walls; for example, Makino M. (1985) conducted a series of cyclic lateral loading tests of one-third scale portal steel

[1] *Presenter, Associate Professor, National Yunlin University of Science and Technology (YunTech), Taiwan, e-mail: leehj@yuntech.edu.tw*
[2] *PhD Candidate, National Yunlin University of Science and Technology, Taiwan (ROC)*
[3] *Professor, National Taiwan University of Science and Technology, Taiwan (ROC)*

Evaluation of Seismic Performance of RC L-shaped Core-walls for High-rise Buildings

Masanobu SAKASHITA[1], Kazuma INADA[2], Kouhei SAKAMOTO[3], Susumu KONO[4] and Hisataka SATO[5]

ABSTRACT

This paper addresses the seismic performance of core-walls with L-shaped cross sections based on an experiment and numerical analysis on two reinforced concrete specimens with 13% scale. The specimens had large amount of vertical reinforcement in the wall panels and the boundary columns, and the concrete compressive strength of 80MPa so that such core-walls could be a major seismic component in the high-rise building. The test variable was the intensity of axial load. The maximum axial load was 35% of the axial load capacity for L45C specimen, and was 50% for L45D specimen. Damage of boundary columns was severe and indicated the importance of good confinement. Although a simple calculation method and a fiber-element model based on the plane-sections-remain-plane assumption predicted the maximum lateral load with good accuracy, the lateral load-drift angle relations could not be simulated well. In order to improve accuracy of the analysis, it is necessary to evaluate the effect of the plane-sections-remain-plane assumption, pulling out of longitudinal reinforcement from the bottom stub, shear deformation and constitutive model of concrete.

***Keywords**: Core-wall, Confinement, Highly compressed zone, L-shaped cross section*

INTRODUCTION

Maffei and Yuen[1] addressed the advantage of core-wall buildings in their article as follows. "*In recent years, a new type of tall buildings is being constructed. They use reinforced concrete core-walls without supplemental moment resisting frames in the seismic force resisting system. RC core-walls offer advantages of open and flexible architecture. By eliminating the need for moment resisting frames, smaller framing members or flat slabs can be used for the building floors, and the framing depth of floors can be reduced. In a core-wall building, resistance to seismic forces is provided by a reinforced concrete core that surrounds the elevator banks. A core-wall building eventually realizes lower costs and faster construction."* Adebar et al.[2] summarized their tests on concrete shearwalls with height-to-length ratios greater than 2.0. The cross section of those specimens were mainly rectangular, flanged, and barbell. The total vertical reinforcement, as ratio of gross area of the wall cross section, was 2.62% at maximum and the maximum axial compression level was 20%.

In Japan, a large seismic capacity demand requires larger core-walls with heavy amount of vertical reinforcement and higher concrete strength. Since the late 1990's, seismic performance of large scale core-walls have been studied extensively in Japan[3 and 4]. Konishi et al.[5] reported that the confinement of concrete at the corner of the section increased deformation capacity. They simulated the load-displacement relation with good accuracy with a fiber element model with the plane-sections-remain-plane assumption. Maruta[6] proposed a method to determine the amount of shear reinforcement at the boundary column region to confine concrete based on Konishi's work. However, Nakachi[7] reported that the plane-sections-remain-plane assumption cannot be applied to a core-wall after the formation of flexure-shear cracks occurred. Recently, additional experiments

[1] *Assistant Professor, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan, sakashita@archi.kyoto-u.ac.jp*
[2] *Structural Engineer, West Japan Railway Company, Japan*
[3] *Graduate Student, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan*
[4] *Associate Professor, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan, kono@ archi.kyoto-u.ac.jp*
[5] *Structural Engineer, Asanuma Technical Research Institute, Japan*

Rocking Capacity of Unreinforced Masonry Walls

Sung-Gul Hong[1] and Sung-Hoon Kang[2]

SUMMARY

This paper presents an experimental research on the in-plane seismic behavior of the rocking-critical unreinforced masonry walls. The rocking capacity of masonry walls are discussed in terms of strength, stiffness and energy dissipation with the experimental results. In addition, the practical formulae for strength for rocking and lateral stiffness are proposed based on a theoretical approach. In particular, a design chart to estimate the ultimate rocking strength of walls is proposed along with strength formula. The proposed formulae and design chart are compared with the experimental results showing good agreement between them.

Keywords: Unreinforced Masonry Wall; Rocking Failure; Shear Ratio; Ultimate Strength Model.

INTRODUCTION

It has been well known that unreinforced masonry walls are susceptible to earthquake loading. Major damages in unreinforced masonry structures due to moderate to major earthquake have shown that selection of rehabilitation techniques has to be done based on identification of failure modes and their corresponding capacity. Even though majority of low-to medium rise buildings have been well-constructed by concrete and steel design practices and skills, some of low-rise housing structures and schools in Korea which have been constructed by unreinforced masonry structures are weak against possible earthquake. To rehabilitate and strengthening such unreinforced masonry structures, realistic strength and stiffness estimation is necessary for retrofit design.

The governing failure modes include diagonal cracking, toe crushing, and sliding failures depending aspect ratios of walls and compression force. One of weakest masonry components is pier between openings found in school buildings. The governing failure mode of such piers is rocking mode similar to toe crushing failure mode. The rocking failure experiences opening at top and bottom at ultimate. This study focuses on rocking behavior of wall components. The experimental program for this study varies the aspect ratio of walls as a main variable. Based on the experimental results and theoretical approach, the strength formula and the stiffness are proposed.

EXPERIMENTAL PROGRAM

Materials Test

In order to apply the mechanical properties of unit masonry to wall components, the materials tests were conducted according to Korean Standard (KS) as well as American Society of Testing Materials (ASTM). Fig. 1 shows the following tests: compressive strength of mortar, concrete brick and masonry prism, and diagonal tension (shear) strength of masonry wall components.

[1] Professor, Seoul National University, Korea, E-mail:sglhong@snu.ac.kr
[2] Graduate student, Seoul National University, Korea, E-mail:ngudpe@snu.ac.kr

EVALUATING PROCEDURE OF OPTIMAL SEISMIC RETROFIT LEVEL FOR A LOW-RISE RC BUILDING

Chien-Kuo Chiu[1]

SUMMARY

In this study, an estimating procedure that can be used to set the optimal seismic level in the seismic retrofit design for a low-rise RC building is proposed. Along with damage-control, the cost of maintenance over the remaining service life is also considered in the estimating procedure. However, when a seismic retrofit level defined by the ground acceleration corresponding to the ultimate deformation capacity of the single degree-of-freedom (SDOF) system for a selected building is specified, several seismic retrofit methods can be used to achieve the specified seismic retrofit level. Since their maintenance costs are different to each other, the average value of the annual costs of maintenance for a specified seismic retrofit level was adopted. Under the condition that the average value of the annual reliability indices of a seismic retrofit level for a specified damage state needs to be equal or over the allowable annual reliability index, the seismic retrofit level corresponding to the minimal value is regarded as the optimal level. Finally, a case study was used to discuss the application of the estimating procedure for the optimal seismic retrofit level proposed here. In addition, the requested combinations of the upgrading rates in yielding acceleration and the ultimate cycle ductility capacity of the SDOF system are also suggested to the designer of the retrofit for the selected building.

***Keywords**: seismic reliability, maintenance, retrofit, reinforced concrete*

INTRODUCTION

In general, the seismic performance of a RC building can be evaluated based on the capacity spectrum method proposed by ATC-40; then, by means of upgrading the structural strength or ductility, or a combination of both, the retrofitting seismic performance could be upgraded to meet or even exceed the code-required level. However, the cost of maintenance and damage-control in the remainder of its service life are not considered in the seismic retrofit design, except for the safety performance. In other words, an evaluation system that can be used to find an optimal seismic retrofit strategy for an RC building based on damage-control and cost of maintenance over the remaining service life is essential.

The performance-based seismic design method has already been developed for a few years. Several models or procedures have been proposed to define and quantify multiple performances of a structure, such as safety, serviceability, economy and reparability. Furthermore, life-cycle assessments have also been included in many proposed models of performance-based seismic design for structures in recent times (Takahashi et al.). Although Padgett et al. (Padgett et al. 2010) has proposed the risk-based seismic life-cycle cost-benefit analysis method for bridge retrofit assessment, the concept of performance-based design is seldom mentioned in seismic retrofit work for existing RC buildings. In this study, an estimating procedure that can be used to set the optimal seismic level in the seismic retrofit design for a low-rise RC building is proposed. Along with damage-control, the cost of maintenance over the remaining service life is also considered in the estimating procedure. However, when a seismic retrofit level defined by the ground acceleration corresponding to the ultimate deformation capacity of the single degree-of-freedom (SDOF) system for a selected building is specified, several seismic retrofit methods can be used to achieve the specified seismic retrofit level. Since their maintenance costs are different to each

[1] *Assistant Professor, National Taiwan University of Science and Technology, Taiwan, e-mail: ckchiu@mail.ntust.edu.tw*

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STATIC LOADING TEST OF CES SHEAR WALLS WITH DIFFERENT ANCHORAGE CONDITION OF WALL REINFORCEMENT

Tomoya MATSUI[1], Suguru SUZUKI[2] and Hiroshi KURAMOTO [3]

SUMMARY

Composite Concrete Encased Steel (CES) structures composed of steel and fiber reinforced concrete (FRC) have been continuously studied by the authors. In this study, Static loading test of CES shear wall whose anchorage method of CES frame and FRC wall panel is simplified was carried out, and basic structural performance, strength, deformability and failure mechanism, of the CES shear wall are investigate. One experimental variable is anchorage condition, Shear walls with and without anchorage of longitudinal wall reinforcement were prepared. Another experimental variable is failure mode of shear failure and flexural failure type. In this paper, structural performance of CES shear wall obtained from the tests are described. The results show that the effect of anchorage condition of wall longitudinal reinforcement on shear strength and flexural strength of CES shear wall is small. The deformability of CES shear wall slightly improves by omitting anchorage of wall longitudinal reinforcement. It is because that damage area of concrete of wall panel is reduced by occurring slip under the beam after reaching maximum shear force due to without anchorage of wall reinforcement.

***Keywords**: Composite structure; CES framed shear wall; Static loading test; Fiber reinforced concrete.*

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 two-bay two-story CES frame, it was confirmed that CES structural system showed stable restoring force characteristics and good seismic performance.

On the other hand, shear wall which is main earthquake-resistant member is effective in increasing stiffness and

[1] *Assistant Professor, Dept. of Architectural and Civil Engineering, Toyohashi University of Technology, Toyohashi, Japan e-mail: matsui@ace.tut.ac.jp*
[2] *Graduate School of Eng., Toyohashi University of Technology, Toyohashi, Japan, e-mail: suzuki.suguru@rc.ace.tut.ac.jp*
[3] *Professor, Dept. of Architectural Engineering, Osaka University, Suita, Japan, e-mail: kuramoto@arch.eng.osaka-u.ac.jp*

CONTINUOUS COLUMN EFFECTS ON SEISMIC RESPONSE AND STABILITY OF U.S. STEEL MOMENT-FRAME STRUCTURES

Hiroyuki TAGAWA[1], Gregory MACRAE[2], and Laura LOWES[3]

SUMMARY

Typical steel moment-frame structures in the United States comprise a few seismic frames and many gravity frames, which include "continuous columns" that are pin-connected to beams. These continuous columns, which are often ignored in seismic design, can improve the seismic performance of the structure. This study investigates the effects of continuous columns on the structural stability and earthquake response of building frames. Data show that continuous columns within the building reduce both the impact of P-Δ effects and the concentration of yielding. Relationships between continuous column stiffness ratio and maximum interstory drift, maximum drift concentration factor are presented. It is shown that for realistic structures, the columns in the seismic frames are generally sufficient to prevent large drift concentrations due to unstable response, and the inclusion of gravity continuous column stiffness in the numerical model tends to decrease both the predicted drift concentration and the maximum story drift demand.

Keywords: *Continuous Column Effects; Steel Moment Resisting Frame; Seismic Response; Drift Concentration; Dynamic Stability; Gravity Columns*

INTRODUCTION

Tall steel moment-resisting frame structures may be subject to significant P-Δ effects as they move laterally during earthquake excitation. Analyses of both SDOF oscillators (e.g. Jennings and Husid, 1968, Miranda and Akkar, 2003) and tall frames (e.g. Uetani and Tagawa, 1998, Tagawa, 2005) have shown that if the tangent stiffness of a structure becomes negative, the potential for unstable dynamic response resulting in large displacements increases significantly. In multi-story frames the large displacements generally occur over a few stories resulting in a high concentration of drift.

Continuous columns, in lateral- and gravity-load resisting frames as shown in Figure 1(a), have been found to significantly reduce the potential for large drifts in braced frames (e.g. MacRae, 1990, Tremblay and Stiemer, 1994). Using simple kinematic considerations, Tremblay and Stiemer (1994) proposed design guidelines for the required minimum stiffness of the continuous columns to achieve desirable response. Gupta and Krawinkler (2000) carried out static pushover and dynamic earthquake analyses of 3-, 9-, and 20-story steel moment-resisting frame structures including and excluding gravity columns and found that the gravity columns generally reduced interstory drift in moment frames too. Recently, MacRae, Kimura, and Roeder (2004), using data from pushover and dynamic analyses, developed equations relating continuous column stiffness and the magnitude of drift concentrations within a brace frame. Here, increasing the column stiffness caused a desirable decrease in demand.

[1] *Researcher, Ph.D., General Building Research Corporation, Japan, e-mail: tagawa@gbrc.or.jp*
[2] *Associate Professor, Department of Civil Engineering, University of Canterbury, NZ, e-mail: gregory.macrae@canterbury.ac.nz*
[3] *Associate Professor, Department of Civil Engineering, University of Washington, USA, e-mail: lowes@u.washignton.edu*

EVALUATION OF BLAST-INDUCED PROGRESSIVE COLLAPSE RESISTANCE OF STEEL MOMENT FRAMES

Kyungkoo LEE[1], Seonwoong KIM[2], and Lan CHUNG[3]

SUMMARY

The recent progressive collapse analysis method of structures assumes the threat-independent removal of vertical load-carrying elements. In reality, a blast-induced column-missing event will produce the damage on adjacent structural elements and the rapid dynamic response of the structures. In this study, the strain rate effects on the dynamic collapse behavior of blast-damaged steel moment frames are investigated by performing the blast-induced sequential progressive collapse analysis of a three-storey steel moment frame.

Keywords: *steel moment frames; blast loading; progressive collapse analysis; strain rate effect.*

INTRODUCTION

Interest in the blast resistance design of buildings has been grown since the blast explosions usually result in severe damage to buildings and casualties. The building damage may be divided into the failure of structural elements (so-called direct air-blast effects), and progressive collapse of building (so-called consequential effects). Direct air-blast effects are that the blast load induces the localized failure of structural elements such as exterior walls, columns, beams, and floor systems. Consequential effects are that the localized damage propagates from element to element, and then leads to partial or total progressive building collapse.

A common progressive collapse analysis method of structures is the alternate path method, in which a structure is analyzed to ensure the structural integrity after the removal of vertical load-carrying elements regardless of threat (AISC 2004). This analysis method typically assumes that a single column at the first story of a building is missing and the residual capacity of structural system to bridge over the damaged area is evaluated to prevent the entire or partial collapse of the building. The alternate path method for the progressive collapse analysis of structures is preferred in the design guidelines issued by the General Services Administration (GSA 2003) and the Department of Defense in the United States (DoD 2005).

The evaluation of the ductility of structural systems is very important in order to mitigate the potential of the progressive collapse of buildings. Ductile steel moment frames may be a good structural system to resist the progressive collapse of buildings (AISC 2004). Blast resistant design of steel moment frames generally provides sufficient toughness of elements and structural system capable of limiting the possibility of building collapse (Hamburger and Whittaker 2003).

Experimental and analytical studies on the blast and post-blast collapse response of steel frame moment connection assemblies have been conducted recently to assess the implication of connection behavior on progressive collapse (Karns et al. 2006). Many studies have indicated that inelastic dynamic analysis should be conducted to represent the progressive collapse behavior considering the beam plastic rotation and the interaction effects of axial force and moment in the beams (Marjanishvili and Agnew 2006; Lee et al. 2010)

[1] *Assistant Professor, Department of Architectural Engineering, Dankook University, Korea, e-mail: kklee@dankook.ac.kr*
[2] *Post-doctoral Researcher, Department of Architectural Engineering, Dankook University, Korea, e-mail: corea13@snu.ac.kr*
[3] *Professor, Department of Architectural Engineering, Dankook University, Korea, e-mail: lanchung@dku.edu*

FRAME AND BRACE ACTIONS IN CORNER GUSSET PLATE CONNECTIONS OF STEEL BUCKLING-RESTRAINED BRACED FRAMES

Chung-Che CHOU[1] and Jia-Hau LIU[2]

SUMMARY

This work presents test and finite element analysis results of the steel buckling-restrained braced frame (BRBF). The objective is to evaluate the effects of the frame action and brace action on the corner gusset plate and provide a method that considers both actions in design. The BRBF exhibits excellent performance up to an interstory drift of 2% with a maximum axial strain 1.7% in the BRB. Without utilizing free-edge stiffeners, the corner gusset plate buckles at much lower strength than that based on AISC specification (2005). Idealizing the corner gusset plate as a strut, an equivalent strut model can be used to determine forces caused by the frame action. Considering stress distributions and force components from the frame and brace actins, limiting maximum stresses in the gusset tips at a design drift is used as an additional design parameter for sizing the corner gusset plate and free-edge stiffener.

Keywords: buckling-restrained braced frame, Corner gusset plate, Frame action, Brace action

INTRODUCTION

Buckling-restrained braced frames (BRBFs) have been increasingly used for lateral load resistance in addition to the traditional moment-resisting frames (Chou and Jao 2010, Chou et al. 2010). Gusset plates commonly used in conventionally braced frames are adopted in BRBFs to connect a BRB to the beam and column. To ensure stable energy dissipation of the BRB, the AISC seismic design provisions (2005) require that axial capacity of a gusset plate exceed ultimate load of a BRB. The Whitmore's width concept (1952) and the formula for column buckling are adopted in AISC specification (2005) for evaluating tension and compression capacities of the corner gusset plate based on the work by Bjorhovde and Chakrabarti (1985) and Gross (1990). The provisions also require that gusset plate instability be considered because recent studies demonstrated out-of-plane buckling or fractures of the gusset plate prior to reaching the ultimate compressive capacity of a BRB (Aiken et al. 2002, Tsai et al. 2008, Chou and Chen 2008). Aiken et al. (2002) and Kasai et al. (2009) also pointed out that when a diagonal BRB is used in the frame structure, a corner gusset plate is subjected to not only the brace action but also the frame action so the gusset-to-column tip or gusset-to-beam tip fractures while the BRB is in compression.

When the BRB is in tension, the angle of the beam-to-column connection becomes smaller than the original angle, producing compression forces to the corner gusset edges (Figure 1(a)). When the BRB is in compression, the angle of the beam-to-column connection is larger than the original angle, producing tension forces to the gusset edges (Figure 1(b)). By adopting the equivalent strut concept (Lee and Uang 2000, Lee 2002), Kaneko et al. (2008) proposed that forces in the corner gusset plate be considered from the frame action and brace action, respectively. It is not clear how the equivalent strut force is function of the beam and column deformation and how the magnitude of the equivalent strut force is compared to the brace force. Moreover, the frame action in the corner gusset plate is not considered in the specification, and no guideline is provided for sizing stiffeners along edges of the corner gusset plate.

[1]*Associate Professor, Department of Civil Engineering, National Taiwan University, Taipei, Taiwan, e-mail: cechou@ntu.edu.tw*
Associate Research Fellow, National Center for Research on Earthquake Engineering, Taipei, Taiwan
[2]*Graduate Student Researcher, Department of Civil Engineering, National Taiwan University, Taipei, Taiwan*

PERFORMANCE ASSESSMENT OF BUCKLING RESTRAINED BRACES

Heui-Yung CHANG[1]

SUMMARY

Buckling restrained braced frames (BRBFs) have superior performance in reparability and costs, when compared to moment resisting frames (MRFs). Since that, the use of BRBFs has rapidly grown in seismically active zones, such as Taiwan, Japan and North America. In the company paper, the global drift demand and capacity of a 6-story BRBF have been evaluated. That confirms the confidence with which a BRBF might achieve the performance expected of a new MRF. This paper evaluates the seismic performance of the BRBF from a local perspective. In detail, the capacity and demand of the BRBs are assessed by test data and response analysis. Then, the confidence levels of the BRBs are estimated for immediate occupancy, life safety and collapse prevention. All that helps gain a better understanding about the global and local seismic performance of BRBFs.

Keywords: *confidence level; performance evaluation; local perspective; buckling restrained brace.*

INTRODUCTION

Toward the development of performance-based engineering, there has been a necessity of assessing structural performance on a reliability basis. On the other hand, buckling restrained braced frames (BRBFs) have superior performance in reparability and costs, and the use of BRBFs has rapidly grown in seismically active zones, such as Taiwan, Japan and North America. In the company paper, the reliability frame work used for moment resisting frames (MRFs) was extended and employed to evaluate the seismic performance of BRBFs (Chang 2009). The evaluation confirmed the confidence with which a BRBF might achieve the seismic performance expected of a new MRF.

This paper evaluates the performance of the BRBF from a local perspective. In detail, the capacity and demand of the BRBs are assessed by test data and response analysis. Then, the confidence levels of the BRBs are estimated for immediate occupancy (I.O.), life safety (L.S.) and collapse prevention (C.P.). FEMA 351, for example, recommends design solutions to provide a 50% level of confidence that the building satisfy desired performance at a local level. In the presented work, the 50% value has been used to judge whether the design of the BRBs needs improvement and modifications. The evaluation also helps gain a better understanding about the global and local seismic performance of BRBFs.

BRB DEFORMATION CAPACITY ESTIMATES AND UNCERTAINTY

Performance objectives

Table 1 gives an example illustrating the structural performance levels and damage of braced steel frames. As can be seen, BRB deformations at first yield, buckle and fracture are important to justify the performance levels of the braced steel frames.

[1] *Assistant Professor, National University of Kaohsiung, Taiwan, e-mail: hychang@nuk.edu.tw*

EFFICIENT ANALYTICAL MODEL FOR DYNAMIC ANALYSIS OF TALL BUILDINGS

Dong-Guen LEE[1], Yong-Koo Park[2], and Hyun-Su Kim[3]

SUMMARY

In this study, an efficient analytical model for dynamic analysis of tall buildings with shear wall-frame structural system has been proposed. Shear wall-frame structural system usually consists of core wall showing flexural behavior and frame presenting shear behavior. Therefore, the deformed shape of shear wall-frame structural system is shown by combination of flexural mode and shear mode. These characteristics should be considered when an efficient analytical model is developed. To this end, the effect of shear wall and frame on dynamic behavior of a tall building with a dual system has been separately investigated. In this study, the structural characteristics of a separated individual shear wall model and the frame model without shear wall has been evaluated. In order to consider the effect of shear wall in the frame model without shear wall, rigid body was used instead of the shear wall. Each equivalent model for the separated shear wall part and frame part has been independently developed and then two equivalent models were combined to make an efficient analytical model for tall buildings with shear wall-frame structural system. In order to verify the efficiency and accuracy of the proposed method, time history analyses of tall buildings with shear wall-frame system were performed. Based on analytical results, it has been confirmed that the proposed method can provide results with accuracy requiring significantly reduced computational time and memory.

***Keywords**: tall buildings, efficient analytical model, dual system, wall core, dynamic analysis*

INTRODUCTION

Recently, building structures become larger and higher to satisfy the social and economical needs. Since high-rise building structures subjected to seismic loads usually undertake significant shear forces due to big structural masses and height, effective seismic resistance structural system is essential. Recently, shear wall-frame structural system such as dual system having R.C. core is widely used as a seismic resistance system of tall buildings. Since seismic resistance and wind resistance of high-rise buildings should be investigated to verify the structural safety and serviceability, dynamic analyses with wind and earthquake excitations are required. If dynamic behavior of high-rise buildings is not good enough, additional vibration control devices can be used. In order to effectively apply vibration control devices to tall buildings, repetitive structural analyses should be performed. In this process, time history analyses are necessarily carried out to accurately predict the dynamic behavior of structures. When time history analyses of tall shear wall-frame buildings are performed with conventional finite element model, significant computational time and efforts may be required. To solve this problem, efficient analytical model that can present dynamic characteristics of high-rise buildings with significantly reduced computational time has been proposed. Cantilever model (Model C) is widely used for this purpose because it can exactly present the first mode dynamic characteristics of high-rise buildings. However, since the cantilever model ignores the effect of beam stiffness and shows bending behavior, it is not good enough to present mode shapes and characteristics of higher modes of real high-rise building structures. Based on this problem, proposed the equivalent analytical model (Model M) that can consider mode shapes of high-rise

[1] *Professor, Department of Architectural Engineering, Sungkyunkwan University, Korea, e-mail: dglee@skku.ac.kr*
[2] *Graduate Student, Department of Architectural Engineering, Sungkyunkwan University, Korea, e-mail: yk-0127@hanmail.net*
[3] *Assistant Professor, Division of Architecture, Sunmoon University, Korea, e-mail: hskim72@sunmoon.ac.kr*

DAMAGE ASSESSMENT OF REAL BUILDING BASED ON ON-LINE RECURSIVE LEAST-SQUARES METHOD

Shih-Yu CHU[1], Shih-Chieh LO[2]

SUMMARY

The identification of structural damage is an important objective of health monitoring for civil infrastructures. In order to develop a real-time structural damage assessment system, the on-line recursive least-squares (RLS) identification technique is developed and applied in this study. The RLS method is based on the framework of adaptive filters; the observations are obtained sequentially in real-time. It is desirable to perform the identification tasks recursively to reduce computation time and to be able to observe the variations of parameters on-line. In this study, we discuss the applicability of proposed on-line RLS identification technique to evaluate the time-varying dynamic properties of the Civil and Environmental Engineering Department (CEED) building at National Chung Hsing University (NCHU) in Taiwan. The proposed on-line RLS identification technique is evaluated on the basis of the 1999 Chi-Chi earthquake main shock and its numerous after-shocks. Successive earthquake inputs and their corresponding output signals are selected and the global structural properties exhibited during each earthquake event are identified. Furthermore, the poles of relative story transfer functions between sensors at different stories are computed to identify the location of the damage. By observing the variations of the identified time-varying modal properties of the CEED building, its global and local damage behavior due to failure of components can be revealed.

Keywords: *system identification; recursive least-squares method; structural health monitoring; damage assessment; relative story transfer function.*

INTRODUCTION

The identification of structural damage is an important objective of health monitoring for civil infrastructure. The main goal of structural health monitoring is to provide the ability to assess damage in real-time or immediately after a catastrophic earthquake. An on-line recursive least-squares (RLS) identification technique is applied in this study to identify the time-varying dynamic parameters of structures subjected to earthquake loadings. Computation of the classical least-squares (LS) method can be arranged recursively so that the estimated parameters at the previous step can be used to predict the responses at the current time. The one-step-ahead predicted error between the estimated response and the measured response is calculated using the on-line RLS method. The dynamic properties of the system can also be identified.

[1] *Associate Professor, Department of Civil Engineering, National Cheng Kung University, Taiwan, e-mail: sychu@mail.ncku.edu.tw*
[2] *Ph.D., Department of Civil Engineering, National Cheng Kung University, Taiwan.*

TESTS ON SEISMICALLY DAMAGED REINFORCED CONCRETE BEAM-COLUMN JOINTS REPAIRED USING FIBER-REINFORCED

Bing LI[1]

SUMMARY

To prevent the casualties which can result from the potential collapse of earthquake damaged structures, it is important that structure can be rehabilitated as soon as possible. This paper proposes a rapid rehabilitation scheme for repairing moderately damaged reinforced concrete (RC) beam-wide column joints. Four non-seismically detailed interior beam-wide column joints were used as control specimens. All of these four sub-assemblages were subjected to similar cyclic lateral displacement so as to provide the equivalent of severe earthquake damage. The damaged control specimens were then repaired by filling their cracks with epoxy and externally bonding them with Carbon fiber reinforced polymer (CFRP) sheets and Glass fiber reinforced polymer (GFRP) sheets. These repaired specimens were then re-tested and their performance compared to that of the control specimens. This paper demonstrates that repair of damaged RC beam-wide column joints using FRP is able to restore the performance of damaged RC joints with relative ease, suggesting that the repair of beam-column joints is a cost effective alternative to complete demolition and replacement.

***Keywords**: beam-column joints; repair; rehabilitation; seismic, fiber reinforced polymers*

INTRODUCTION

Premature failure of reinforced concrete (RC) beam-column joints may lead to large lateral deformations and consequent collapse. Inadequate detailing of beam-column joints may result in bursting failures, shear failures, and anchorage failures, possibly contributing to the soft storey effect, particularly if such failures occur within the columns. As a critical element in structural design, beam-column joints play a pivotal role in resisting earthquake loading. However, the BS 8110 code used in Singapore does not specify any provision for the seismic design or detailing of RC structures. There is insufficient transverse reinforcement, discontinuous beam bottom reinforcement or other non-ductile detailing (Li *et al*. 2002). Therefore, the structures in these regions of low to moderate seismicity have to rely on its inherent ductility to respond to seismic excitations, making them vulnerable to damage and collapse in the case of an earthquake.

Current practices generally see the demolition of the damaged structure in the instance of damage, which is highly resource inefficient. Rehabilitation on the other hand provides a much more economical alternative. However, the lack of understanding of the performance and effectiveness of the repairs has seen structure rehabilitation shunned in favor of complete demolition. This provides a significant impetus to prove and illustrate the seismic performance of beam-column joints repaired and strengthened using various rehabilitation methods. Knowledge gained from the test results in this paper will be useful in developing effective and economical techniques to rehabilitate such non-seismically designed beam-wide column joints.

Fiber reinforced polymer (FRP) composite materials have grown in popularity over the past decade, valued for their high strength-to-weight ratios, corrosion resistance, ease of application and tailor-ability. Fiber orientation in each ply can be adjusted to meet specific strengthening objectives Engindeniz *et al*. (2005). A detailed review of literature shows that the beneficial effects of FRP composites on seismic behavior of non-seismically

[1] *Associate Professor, School of Civil and Environmental Engineering, Nanyang Technological University, Singapore 639798*
e-mail: cbli@ntu.edu.sg

SEISMIC PERFORMANCE OF REINFORCED CONCRETE SPECIAL MOMENT FRAMES DESIGNED BY PERFORMANCE-BASED PLASTIC DESIGN METHOD

Wen-Cheng Liao[1] and Suhbash C. Goel[2]

SUMMARY

Reinforced concrete special moment frames (RC SMF) have been widely used as part of seismic force-resisting systems, design methodologies and systematic procedures are needed which require no or little iteration after initial design in order to meet the targeted design objectives. This paper presents first time application of the Performance-Based Plastic Design (PBPD) approach to RC SMF. Four baseline RC SMF (4, 8, 12 and 20-story) as used in the FEMA P695 were selected for this study. Those frames were redesigned by the PBPD approach. The baseline code designed frames and the PBPD frames were subjected to extensive inelastic pushover and time-history analyses. The seismic responses of the study frames met the targeted performance criteria with significant improvement over the corresponding baseline code designed frames.

***Keywords**: performance-based plastic design; reinforced concrete moment frames; earthquake resistant design.*

INTRODUCTION

Reinforced concrete special moment frames (RC SMF) comprise of horizontal framing components (beams and/or slabs), vertical framing components (columns) and joints connecting horizontal and vertical framing components and deemed to satisfy the special requirements in seismic provisions (ASCE/SEI 41-06, ACI 318). In seismic provisions, certain special requirements such as special proportioning and detailing requirements result in a frame capable of resisting strong earthquake shaking without significant loss of stiffness or strength. Since RC SMF have been widely applied as part of seismic force-resisting systems, design methodologies and systematic procedures are needed which require no or little iteration after initial design in order to meet the targeted design objectives.

In order to achieve more predictable structural performance under strong earthquake ground motions, knowledge of the ultimate structural behavior, such as nonlinear relations between force and deformation, and yield mechanism of the overall structure are essential. Consequently, design factors such as, determination of appropriate design lateral forces and member strength hierarchy, selection of desirable yield mechanism, and structure strength and drift etc., for given hazard levels should become part of the design process right from the beginning. Therefore, a complete seismic design method should include not only determination of proper design base shear, but also a systematic procedure for proportioning members by considering inelastic characteristics of the overall structure. One such complete design methodology, which accounts for inelastic structural behavior directly, and practically eliminates the need for assessment or iteration after initial design, has been developed in the recent past for practical design work (Goel at el., 1999~2009). It is called Performance-Based Plastic Design (PBPD) method.

The PBPD method uses pre-selected target drift and yield mechanism as key performance limit states. These two limit states are directly related to the degree and distribution of structural damage, respectively. The design base

[1] *Assistant Professor, Department of Civil Engineering, National Taiwan University, Taiwan , e-mail:wcliao@ntu.edu.tw*
[2] *Emeritus Professor, Department of Civil & Environmental Engineering, University of Michigan, USA, e-mail: subhash@umich.edu*

STUDY ON THE EARTHQUAKE DAMAGE EVALUATION PROCEDURE FOR RC AND CONFINED MASONRY BUILDINGS

Yi-Hsuan TU[1], Lai-Cheng AO[2], Wen-Yu JEAN[3]

SUMMARY

Data collection is an important work in the earthquake reconnaissance. Damage data that was not collected or archived properly might cause error in the following researches using the data. For instance, different damage evaluation procedures are used to describe the damage state of buildings in different earthquakes, causing the difficulty and misunderstanding in sharing the data. This paper aims at establishing an earthquake damage evaluation procedure that is objective and easy to use for low-rise RC and confined masonry buildings. Several current damage evaluation standards were reviewed and summarized to determine the evaluation factors. The relationships between evaluation factors and damage conditions were discussed by applying the presented procedure to three in-situ test specimens. The ability to distinguish medium damage states of the procedure was also verified by using the data of 10 school buildings damaged during recent moderate earthquakes. Twelve professionals with different backgrounds were asked to evaluate the damage state of the buildings with both their subjective judgment and the presented procedure. It was found that the damage state determined by the presented procedure showed less dispersion and stricter result than the subjective judgments.

Keywords: *earthquake damage; data collection; damage evaluation; reinforced concrete; confined masonry.*

INTRODUCTION

The past experience is a good teacher for people learning to face repeatable disasters, such as earthquakes and floods. Such effort has been initiated early. One of the best-known examples is the Learning from Earthquakes (LFE) Program by Earthquake Engineering Research Institute (EERI) that started from 1973 (http://www.eeri.org/site/lfe-introduction). As it points out, making observations and keeping records of the damage and effects following a disaster are critical to managing emergency response activities in the short term and improving the understanding of natural hazards in the long term.

Damage data has been widely used in earthquake-related researches, such as seismic assessment, loss estimation, and establishment of vulnerability function. The ATC-13 report (ATC, 1985) presented a methodology for estimating earthquake damage/losses by using existing damage data from California. However, it is usually difficult to apply the damage data out of the region where it was originally collected. The difficulty comes not only from the difference in structural characteristics of the building culture in different areas, but also from the difference in description and definition of damage. The definition of damage can be qualitative or quantitative, chosen by the research subjectively. Qualitative damage state could be simply expressed as damaged/collapsed or subdivided into several discrete levels (Whitman et al. 1973; EERI, 1996; Dolce et al. 2006); quantitative damage index might be defined as cost/number/range of repair/replacement for individual building or the percentage for a category of buildings (Scawthorn et al. 1981; Miyakoshi et al. 1997; Nagato and Kawase, 2004).

[1] *Assistant Professor, Department of Architecture, National Cheng Kung University, Taiwan, e-mail: yhtu@mail.ncku.edu.tw*
[2] *Former Master Student, Department of Architecture, National Cheng Kung University, Taiwan*
[3] *Research Fellow, National Center for Research on Earthquake Engineering, Taiwan*

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SEISMIC PERFORMANCE OF FRICTION DAMPERS INSTALLED AT COUPLING BEAMS BETWEEN SHEAR WALLS

Byoung-Wook Moon[1] and Kyung-Won Min[2]

SUMMARY

Friction dampers using the flexural deformation of shear walls can be installed as coupling beam system between two adjacent walls of shear wall structures. To verify the seismic control performance of proposed friction dampers, numerical nonlinear analysis of shear walls governed by flexural behavior is conducted. Control effectiveness of shear walls connected by beams with the proposed dampers are compared for single shear wall with same flexural rigidity. Average responses of the shear walls with the dampers are found with seven scaled-downed earthquakes based on KBC 2005 design spectrum. Slip load is the most important design parameter. It is designed to be 5, 10, 20, 30, 60, 90% of total vertical shear force at damper location to prevent damper slip in specific stories. Nonlinear time-history analysis is conducted by using SeismoSturct analysis program. Seismic control performance of the dampers is evaluated for base shear, energy dissipation and top-floor displacement. Results show that the dampers are the most effective in reducing the responses when their total slip load is 30% of total vertical shear force.

***Keywords**: SEEBUS; earthquake engineering; building structures; refined-plastic hinge method; spread of plasticity.*

INTRODUCTION

Shear wall type apartment that occupies most of apartment houses of Korea may be regarded as an excellent structure from viewpoints of easiness of floor plan and utilization of internal space. However, as recently the shear wall type apartments are being constructed in high story over 20 floors, importance to secure seismic resistance performance has been emphasized.

This paper deals with a retrofitting method using a damping device instead of an existing stiffness retrofit in order to enhance the seismic performance of shear wall apartment buildings. Damping devices minimize damages on structure through energy dissipation. Generally, the damping device using a brace is used in frame structure. The brace, in which shear force is concentrated in case of frame structure, performs a role that the lateral force such as an earthquake transfers to damping device. The damping system using a brace was progressed many studies by other researcher and applied in many cases. However, for shear wall type structure, using brace to install the damping device is not effective. Because stiffness of shear wall is so large and load to be transferred to the brace is not enough strong to dissipate sufficient energy. Thus, in order to overcome the problem, it is required to develop damping device and installation format appropriate for shear wall type apartment.

The existing researches related to reinforcing method using damping device on shear wall are as follows. Hong Sung-Gul and Lee Ji-Hyeong had confirmed that stayed column has damping effect and stiffness increasing effect to reinforce shear wall, and Park Ji-Hun and Kim Gil-Hwan had performed non-linear analysis for installation pattern and amount of friction strength in order to verify performance of friction damper for shear

[1] *PhD. Candidate, Department of Architectural Engineering, Dankook University, Korea, e-mail: byoungwook.monn@gmail.com*
[2] *Professor, Department of Architectural Engineering, Dankook University, Korea, e-mail: kwmin@dankook.ac.kr*

BI-DIRECTIONAL COUPLED TUNED MASS DAMPERS FOR THE MODAL CONTROL OF TWO-WAY ASYMMETRIC-PLAN BUILDINGS

Jui-Liang LIN[1], and Keh-Chyuan TSAI[2]

SUMMARY

The proposed BiCTMD was developed from the three-degree-of-freedom (3DOF) modal system, which represents a single vibration mode of a two-way asymmetric-plan building. The performance of the proposed BiCTMD for the reduction of the seismic responses of elastic two-way asymmetric-plan buildings was verified. In addition, the investigation showed that the proposed BiCTMD is effective in reducing the seismic damage of inelastic two-way asymmetric-plan buildings. Therefore, the BiCTMD is an effective approach for the seismic response control of both elastic and inelastic two-way asymmetric-plan buildings.

***Keywords**: tuned mass dampers; asymmetric-plan buildings; modal control; seismic response; bi-directional ground motions.*

INTRODUCTION

Tuned mass dampers (TMDs) are commonly recognized as an effective approach for reducing the seismic responses of elastic buildings (Tsai and Lin 1993; Igusa and Xu 1994). Moreover, Rana and Soong (1998) showed that a TMD can be effectively designed for controlling the selected vibration mode of an elastic multi-story symmetrical building. Due to the modal contamination problem, the multiple TMD (MTMD) is not recommended for the simultaneous control of several vibration modes of an elastic multi-story building (Rana and Soong 1998). At the same time, the performance of elastic TMDs for displacement control is not as good for inelastic buildings as it is for elastic buildings due to the de-tuning effect when the building yields (Pinkaew et al. 2003). Although the reductions in the displacement responses of inelastic buildings caused by using elastic TMDs are not significant, some researchers have confirmed that the hysteretic energy demand of symmetrical buildings can be significantly reduced by using elastic TMDs (Pinkaew et al. 2003; Wong 2008). Thus, generally speaking, elastic TMDs are effective for the seismic response control of both elastic and inelastic symmetrical buildings.

In fact, due to their architectural and functional requirements, most real buildings are plan-asymmetric. Keeping in mind that the vibration modes of asymmetric buildings are translation-rotation coupled, using a single translation-only TMD is therefore not a very effective approach for the modal control of asymmetric-plan buildings. In order to control the coupled vibration of asymmetric-plan buildings by using conventional TMDs, the typical approach is to adopt the MTMD (Ahlawat and Ramaswamy 2003). Compared with the design of a single TMD, the design of the MTMD is potentially much more complicated for a practicing engineer. For example, there are additional parameters, such as the number of TMDs, the spacing between the TMDs, the frequency ranges of the TMDs, the eccentricity between the TMDs and the building, and others. All of these parameters must be optimized when the MTMD is selected to control an asymmetric-plan building. In order to simplify these complications, a single coupled TMD (CTMD) was developed that vibrates simultaneously in

[1] *Associate Researcher, National Center for Research on Earthquake Engineering, e-mail: jllin@ncree.org.tw*
[2] *Professor, Department of Civil Engineering, National Taiwan University, Taiwan, e-mail: kctsai@ncree.org.tw*

PROPOSAL OF ELASTIC BUCKLING STRENGTH FORMULA FOR COLUMNS IN STEEL MOMENT FRAMES CONSIDERING ANTI-SYMMETRIC AXIAL FORCES CAUSED BY HORIZONTAL LOADS

Akinobu TAKADA[1], Motohide TADA[2], Seiji MUKAIDE[3], and Yoshikazu ARAKI[4]

SUMMARY

Elastic buckling strength of columns in steel moment resisting frames had been studied for uniform vertical loads by many researchers. But in Japan where seismic loads are significant, the axial forces corresponding to overturning moment due to seismic loads should be considered in design. In such load conditions, the column that suffers larger compressive force can be laterally supported by other columns, which sustain moderate or tensile axial forces. This paper proposes a method to estimate elastic buckling strength of columns in moment resisting frames under anti-symmetric vertical loads on beam-to-column nodes. The accuracy of estimation is confirmed by comparing to the results obtained by geometrical non-linear analyses or eigenvalue analyses for portal frames.

Keywords: *flexural buckling; frame buckling; buckling length; overturning moment*

INTRODUCTION

Considering about flexural buckling is important for designing of steel constructions. Calculation of effective length of unbraced uniform frames is shown in “Design Standard for Steel Structure”(AIJ, 2002) and so on. There are some studies to raise prediction accuracy of cases of using this method for ununiform frames (Mitani et al., 2002; Suzuki, Morino and Kawaguchi, 1993). Alternatively other methods are proposed by Tsuda (2001, 2003) and Shibata(2003). Above studies assume vertical loads. But in Japan where seismic loads are significant, the axial forces corresponding to overturning moment due to seismic loads should be considered (Figure 1). In such load conditions, the column that suffers larger compressive force can be laterally supported by other columns, which sustain moderate or tensile axial forces. So the effective length is shorter than that in vertical load condition.

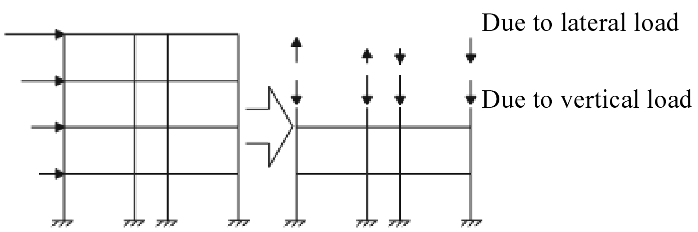

Fig. 1. Axial forces due to lateral load

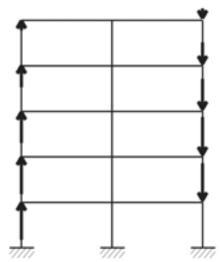

Fig. 2. Anti-symmetric load

[1] *Graduate Student,* *Graduate School of Engineering, Osaka University, Japan, e-mail:takada_akinobu@arch.eng.osaka-u.ac.jp*
[2] *Professor,* *Graduate School of Engineering, Osaka University, Japan, e-mail:tada@arch.eng.osaka-u.ac.jp*
[3] *Assistant Professor,* *Graduate School of Engineering, Osaka University, Japan, e-mail:mukaide@arch.eng.osaka-u.ac.jp*
[4] *Associate Professor,* *Graduate School of Engineering, Kyoto University, Japan, e-mail:araki@archi.kyoto-u.ac.jp*

IN-SITU FORCED VIBRATION TEST AND SYSTEM IDENTIFICATION OF SECONDARY MASS DAMPERS

Hongjin Kim[1], Je-Woo Park[2], Jee-Hoon Lee[3], Young-Ju Jang[4], Sun-Young Choi[5]

SUMMARY

For a secondary mass damper such as tuned liquid damper (TLD) or tuned liquid column damper (TLCD), whose moving mass is liquid, it is impossible to pre-fabricate the damper in a factory for the identification of dynamic properties. Also, it is not easy to pre-fabricate a concrete tuned mass damper (TMD) in a factory whose moving mass is made of concrete. In this paper, an identification method of dynamic properties of secondary mass dampers based on the forced vibration test is presented. Decoupled equations of motion are derived from a coupled equation of motion of building and damper. The decoupled equations of motion are then applied to the system identification using the response of the damper as an input and the response of the building as an output. The proposed method is applied to numerical examples and an actual TMD and TLCD installed in buildings.

***Keywords**: natural frequency, damping ratio, system identification, TLCD, TMD.*

INTRODUCTION

As the design and construction of skyscrapers has increased growing in recent years, the interest in the countermeasures to reduce wind-induced motion has gradually increased. In particular, most tall buildings in construction recently have been residential buildings, and thereby the issue of residents' serviceability against wind-induced vibration has received more attention. To solve this matter, it has become a more common practice to install supplementary damping devices for dissipating the energy caused by the external load rather than to increase the lateral stiffness (Soong and Dargush, 1997).

The most typical supplementary damping devices are inter-story installation type dampers and secondary mass dampers (Kim and Adeli, 2005a, Irwin, 2009). The inter-story installation type dampers provide energy dissipation utilizing hysteretic force-displacement behavior induced by inter-story drift. The secondary mass dampers such as a tuned mass damper (TMD), a tuned liquid damper (TLD), and a tuned liquid column damper (TLCD) are devices relying on the motion of a secondary mass, which is about 0.3 to 1.0% of the entire mass of the building, to counteract the forces acting on a building. Because the secondary mass dampers are often installed on the top floor, they provide fewer architectural constraints than inter-story installation type dampers.

When designing a secondary mass damper, the ratio of moving mass to the first modal mass, or called generalized mass, of a building is determined depending on the required reduction ratio of wind induced vibration. Then, the optimal tuning frequency and damping ratio of the secondary mass damper are calculated based on the determined mass ratio. In the case of a TMD design, the stiffness and number of springs are decided from the optimal frequency ratio, and the damping coefficient and number of oil dampers are decided from the optimal damping ratio (Rana and Soong, 1998, Ghorbani-Tanha et al., 2008).

[1] *Assistant Professor, School of Architecture & Civil Engineering, Kyungpook National Univ., Korea, e-mail:hjk@knu.ac.kr*
[2] *Graduate student, School of Architecture & Civil Engineering, Kyungpook National Univ., Korea, e-mail:unmetoo@knu.ac.kr*
[3] *Graduate student, School of Architecture & Civil Engineering, Kyungpook National Univ., Korea, e-mail: lotus2918@knu.ac.kr*
[4] *Graduate student, School of Architecture & Civil Engineering, Kyungpook National Univ., Korea, e-mail: mansky43@knu.ac.kr*
[5] *Researcher, POSCOE&C, Korea, e-mail: sunyoung@poscoenc.com*

SEISMIC RESPONSE OF STIFFNESS-DEGRADING SDOF STRUCTURES WITH DISPLACEMENT-DEPENDENT DAMPING DEVICES

Park, Ji-Hun[1], Kim, Hun-Hee[2], Kim, Ki-Myon[3]

SUMMARY

Seismic control response of reinforced concrete structures idealized as a SDOF system with degrading stiffness is studied through nonlinear time history analysis. Elasto-perfectly-plastic nonlinear spring is added to the primary SDOF system to model a displacement-dependent damping device. Several curve fitting equations are proposed for prediction of controlled ductility demand and compared with existing equivalent linearization method. The optimal strength of the damping device based on the proposed equation is significantly different from those obtained by the existing analytical formula. It is observed that the reduction of the ductility demand for the shorter natural period system can be reduced more remarkably by the displacement dependent damping devices.

***Keywords**: seismic control; elasto-plastic damper; equivalent damping ratio; stiffness degradation; ductility demand*

INTRODUCTION

In the design of passive energy dissipation systems, energy dissipation effect is reflected in the form of equivalent damping ratio. 2003 NEHRP Provision and ASCE/SEI 7-05 determines seismic displacement response of the structural system with passive energy dissipation systems by dividing the linear response with damping coefficient corresponding to the sum of two damping ratios of which one is inherent structural damping ratio and the other is the equivalent damping ratio obtained from hysteresis of the damping device (BSSC, 2004; ASCE, 2005). However, interaction between a main frame responding nonlinearly and added dampers is not considered clearly.

In preceding research by Ramirez et al. and Whittaker et al., the total equivalent damping ratio is defined by the simple sum of the inherent damping ratio, the equivalent damping ratio from hysteretic behavior of the main frame and the equivalent damping ratio from the damper device and predicts the seismic response of the steel moment frames with velocity-dependent or displacement-dependent dampers considerably well (Ramirez et al., 2002; Ramirez et al., 2003; Whittaker et al., 2003). This study became an important basis of the Chapter 9 in ASCE/SEI 41-06 for seismic rehabilitation of existing building. However, their study dealt with structural systems with little degradation in stiffness or strength and the main frame and the displacement-dependent damper have similar characteristics in hysteresis (ASCE 2007).

In this study, seismic control response of reinforced concrete structures idealized as a SDOF system with degrading stiffness is studied through nonlinear time history analysis. Elasto-perfectly-plastic nonlinear spring is added to the primary SDOF system to model a yielding damper. Based on the result of the analysis, existing nonlinear static procedures for structural systems with displacement-dependent dampers is validated. Several curve fitting equations are proposed for prediction of controlled ductility demand and optimal yield strength and feasible ductility demand reduction effect of the displacement-dependent damper is proposed.

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[1] *Assistant Professor, University of Incheon, Korea, Email: jhpark606@incheon.ac.kr*
[2] *Graduate Student, University of Incheon, Korea*
[3] *Graduate Student, University of Incheon, Korea*

SEISMIC EVALUATION OF A RETROFITTED 5-STORY RC RESIDENTIAL BUILDING MODEL

Han Seon LEE[1], Chang Seok CHO[2]*, Kyung Bo LEE[2]**, Seong Jun HWANG[2]***

SUMMARY

This paper presents the evaluation on the seismic responses on a 1:5 scale model of 5-story piloti-type RC residential building before and after retrofit at the first story. The model survived the design earthquake simulation with the PGA of 0.187g, as specified in KBC2005 without severe damage, even though it was not designed against earthquakes. The same model was retrofitted to prevent the brittle shear failure in columns by using FRP and to reduce the lateral drift and increase energy dissipation capacity by adding buckling-restrained braces (BRB's). The results of earthquake simulation tests on this retrofitted 1:5 scale 5-story model reveal that the retrofitted model behaved inelastically at a low level under design earthquake (PGA=0.187g) and that a significant amount of inelastic behavior occurred under the maximum considered earthquake (PGA=0.3g) in Korea.

Keywords: *Reinforced concrete residential building, Irregularity, Buckling-restrained braces, FRP, Earthquake simulation test*

INTRODUCTION

Many low-rise residential apartment buildings have recently been constructed in the densely populated areas of Korea. As a result of the lack of available sites, the ground floor is usually used for a parking lot and a piloti story is adopted. This type of buildings as shown in Fig. 1, commonly has a high irregularity of soft story, weak story, and torsion simultaneously at the ground story. Observations of the damages to the structures imposed by the severe earthquakes, such as the 1995 Kobe and 2008 Sichuan earthquakes (Zhao, Taucer and Rossetto 2009), have drawn the conclusion that this type of building structures are vulnerable to severe damages or complete collapse of the ground story. A large number of these buildings have been constructed without considering earthquake resistant design requirements in Korea. However, the Korean Building Code (KBC) 2005, which basically follows the framework of the International Building Code (IBC 2000) with some minor modifications, and the other related building laws enforce the seismic design of these building structures. Retrofitting the existing building structures has been one of the main research topics, worldwide, since the 1990's. The objective of this paper is to investigate the effect of a retrofit to an existing nonseismic low-rise RC residential building by comparing the responses of the 1:5 scale model, with and without retrofit, subjected to a series of earthquake simulation tests.

DESIGN AND CONSTRUCTION OF THE RETROFIT

The prototype was determined based on the inventory study and designed by considering the gravity loads only. Drawings of the prototype are shown in Figures 2(a), (b), and (c). The reinforcement details are non-seismic details according to the construction practice in Korea and the details of main members at the ground story are given in Figures 2(d) and (e). The scheme of the retrofit is shown in Fig. 4. Table 1 compares the earthquake

[1] *Professor, School of Civil, Environmental and Architectural Engineering, Korea University, Korea, e-mail: hslee@korea.ac.kr*
[2] *Graduate Student, School of Civil, Environmental and Architectural Engineering, Korea University, Korea: e-mail *csjo@korea.ac.kr, **skyungbo@korea.ac.kr, ***first106@korea.ac.kr*

EFFECT OF OPENING ARRANGEMENT ON SHEAR FAILURE MODE OF RC SHEAR WALLS WITH MULTI-OPENINGS

Masato SAKURAI[1], Tomoya MATSUI[2], and Hiroshi KURAMOTO[3]

SUMMARY

In this paper, static loading tests on RC shear walls with diagonally arranged openings and parallel placed a window and a door carried out to investigate the seismic performance as a part of studies on the shear evaluation for RC shear walls with multi-openings. The results showed that the central panels between openings contribute little to shear forces in the shear walls with proximally-positioned openings. It is also clarified that the calculated shear strengths proposed by Ono and Tokuhiro give good agreement with the test results, though the shear design equation by the AIJ design standard for RC structures gives conservative evaluation for the test results.

Keywords: *RC shear wall with openings; multi-openings static test; shear strength; equivalent perimeter ratio*

INTRODUCTION

In Japan, the shear strength of RC shear walls with openings are generally evaluated by using an equivalent perimeter ratio of openings in the AIJ design standard for RC structures (AIJ, 2010). Therefore, even for the shear walls with different opening layouts, the calculated shear strengths of the shear walls with the same value of the equivalent perimeter ratio are to be the same values.. On the other hand, looking at the existing experimental results and the earthquake damage to RC buildings in the past, the failure mechanisms of RC shear walls with openings are complicated. Especially, in case of those with multi-openings, it was reported that the shear strength, failure mode and deformability were significantly affected by the difference of the number and layout of openings.

In static loading tests of RC shear walls with openings carried out in previous study (e.g., Suzuki et al. 2007 and Sakurai et al. 2008), the shear wall in which two openings are arranged close to each other in the center of the wall showed that the shear strength and stiffness are smaller than that of other shear walls with multi-openings while the deformation capacity after the maximum capacity is larger than the others because frame behavior becomes dominant. In the AIJ design standard, RC shear walls in which two or more openings are arranged closely each other are evaluated by estimating as one opening enveloping the openings. However, but this method is empirical and not discussed based on stress transferring mechanisms and failure modes in detail.

The main objective of this study is to grasp the seismic performance such as failure mode, hysteresis characteristic and deformability and to improve the evaluation method of the shear strength of RC shear walls with multi-openings. Results of a static loading test carried out on RC shear walls which have diagonally arranged openings and parallel placed a window and door were discussed in this paper.

[1] *Graduate Student, of Dept. of Architectural Engineering. Osaka University, Suita, Japan, e-mail: sakurai_masato@arch.eng.osaka-u.ac.jp*
[2] *Assistant Professor, Dept. of Architectural and Civil Engineering, Toyohashi University of Technology, Toyohashi, Japan e-mail: matsui@tutrp.tut.ac.jp*
[3] *Professor, Dept. of Architectural Engineering. Osaka University, Suita, Japan, e-mail: kuramoto@arch.eng.osaka-u.ac.jp*

ROBUST NUMERICAL MODELING APPROACHES FOR UNBONDED POST-TENSIONED CONCRETE MEMBERS

Changsoon RHA[1] and Thomas H.-K. KANG[2]

SUMMARY

Numerical modeling approaches for unbonded type post-tensioned concrete structures were presented. Unbonded tendon was modeled using the finite element software package, Abaqus. Two modeling approaches were studied for the simulation of unbonded conditions: 1) a direct approach using contact element from element library of the software package, and 2) an indirect approach using a multiple-spring system. The modeling with a multiple-spring system showed more flexibility in modeling and robustness in convergence issues. Analytical results showed a reasonably good agreement with experimental results.

Keywords: *concrete constitutive model; unbonded tendons; post-tensioned concrete structures.*

INTRODUCTION

A post-tensioned system is a useful framing method for long span structures. Despite its advantages, theoretical and experimental research is needed to better understand the behavior of post-tensioned members especially in the case of an unbonded system. In this study, two modeling approaches are presented to develop a model for an unbonded post-tensioned concrete structure from an engineering viewpoint. The model were simulated using the proprietary software package Abaqus (2003) and compared to the experimental studies of four unbonded post-tensioned specimens tested by Foutch et al. (1990). In an unbonded post-tensioning system, tendons slip freely within the plastic tube in the concrete; thus, boundary nonlinearity between the tendons and the tubes is required. It indicates that contact techniques in the finite element analysis can be employed for physical modeling of the system. The first approach is just such a direct method that utilizes tube-to-tube contact elements. The friction between the tube and the tendon is always assumed to be zero as the curvature of the tendon is small. This approach depicts real geometry and may give the most reliable result theoretically; however, computational costs of this method are very high and may not be affordable even in a research context. The second approach uses a multiple-spring system in which boundary nonlinearity vanishes. This approach results in reduction of modeling and computational efforts in the analysis while the responses using both approaches are almost identical. Furthermore, the spring system model provides robustness in convergence issues; therefore, the spring system approach is chosen for further analyses in detail.

The concrete constitutive relation employed in this study is a built-in damaged plasticity model in the software package. For modeling members with low reinforcing ratio, small tension stiffening effect may result in severe discontinuity in the process of solving nonlinear equations. Traditional static implicit analysis causes convergence problems for lightly reinforced post-tensioned members, whilst explicit dynamic analysis with a quasi-static analysis avoids convergence issues. Tension stiffening effects are considered for the lightly reinforced post-tensioned member and the results are compared in each modeling approach.

[1] *Assistant Professor, Kookmin University, Korea, e-mail: csrha@kookmin.ac.kr*
[2] *Assistant Professor, The University of Oklahoma, USA, e-mail:tkang@ou.edu*

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

Kil-Hee Kim[1], Young-Seek Kim[2], and Sang-Woo Kim[3]

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

[1] *Associate Professor, Kongju National University, Republic of Korea, e-mail: kimkh@kongju.ac.kr*
[2] *Ph.D. Candidate, Kongju National University, Republic of Korea*
[3] *Research Assistant Professor, Kongju National University, Republic of Korea*

EXPERIMENT ON IMPACT OF BASE-ISOLATED BUILDING MODEL AGAINST RETAINING WALL ON SHAKING TABLE

Toshimoto MAENO[1], Shunji YAMAMOTO[2], Takuya KINOSHITA[3], Naohiro NAKAMURA[4], Shin-ichi MATSUSHIMA[5], Hiroshi KAWASE[6]

SUMMARY

In recent years, some of the earthquake motion levels observed in Japan have been in excess of those currently used in designing base-isolated buildings. Also, the possibilities of major earthquakes are often discussed due to the advanced prediction procedures and seismographic network. To design a base-isolated building against those earthquake motions, larger horizontal clearances are not sufficient due to the restrictions in the bearing strains and damper strokes. One of the potential design methods is a method considering the impact of a building against a retaining wall, but there have been a few researches conducted on the impact behaviors and therefore it cannot be considered in design at present. Under the circumstances, we conducted an experiment on the impact of a base-isolated building model against a retaining wall on a shaking table, for the purposes of clarifying the impact behaviors and using the simulation data for the basis of the design method in consideration of impacts.

Keywords: *base-isolated building; retaining wall; impact; shaking table test.*

INTRODUCTION

In recent years, the seismograms of large-amplitude vibrations have been observed all over Japan due to the dramatic progress of strong motion seismographic network and the repeated occurrence of mega-earthquakes. Some of the earthquake motion levels observed in the seismograms far exceeded the existing design earthquake motions. Also, it is extremely likely that design in consideration of measures against the excessive inputs will be inevitably required in a few years because of such reasons as the often discussed possibilities of major near-field earthquakes and the trend of making it obligatory to use long-period earthquake ground motions as the input earthquake motions for design to be employed for time history response analysis.

Typically in the current design process for base-isolated buildings, horizontal clearances larger than the responses are determined against inputs larger than expected, by referring to the analysis on seismic margin levels. On the other hand, there should be no such damage as building collapse in case of any earthquake larger than predicted. Larger horizontal clearances are not sufficient to withstand those excessive inputs, due to the restrictions in the bearing strains and damper strokes. One of the potential design methods is a method considering the impact of a building against a retaining wall, but there have been a few researches conducted on the impact behaviors. Therefore, currently it cannot be considered in design.

Under the circumstances, we conducted an experiment on the impact of a base-isolated building model against a

[1] *Structural Engineer, Building Design Dept., Takenaka Corporation Osaka Main Office, Japan, e-mail: maeno.toshimoto@takenaka.co.jp*
[2] *Structural Engineer, Building Design Dept., Takenaka Corporation Osaka Main Office, Japan, e-mail: yamamoto.shunji@takenaka.co.jp*
[3] *Dr. Eng., Takenaka R&D Institute, Takenaka Corporation, Japan, e-mail: kinoshita.takuya@takenaka.co.jp*
[4] *Dr. Eng., Takenaka R&D Institute, Takenaka Corporation, Japan, e-mail: nakamura.naohiro@takenaka.co.jp*
[5] *Assoc. Professor, Dr. Eng., Disaster Prevention Research Institute, Kyoto University, Japan, e-mail: matsushima@zeisei.dpri.kyoto-u.ac.jp*
[6] *Professor, Dr. Eng., Disaster Prevention Research Institute, Kyoto University, Japan, e-mail: kawase@zeisei.dpri.kyoto-u.ac.jp*