RESPONSE OF POST-TENSIONED HYBRID PRECAST CONCRETE WALLS SUBJECTED TO NEAR-FAULT GROUND MOTIONS

H. A. D. SAMITH BUDDIKA[1] and Anil C. WIJEYEWICKREMA[2]

SUMMARY

Post-tensioned hybrid (PH) precast concrete walls have been developed as an alternative to cast-in-place reinforced concrete (RC) shear walls, for lateral-force resisting systems in buildings located in earthquake prone regions. However, the effects of near-fault ground motions on the seismic response of PH precast concrete wall buildings, have been investigated only in a limited manner. Therefore, in the present study, the seismic performance assessment of code-designed PH precast concrete wall buildings with various heights is carried out under near-fault ground motions with forward-directivity and fling-step, using rigorous nonlinear response history analysis, and compared with the response to far-fault ground motions. The ground motions are scaled to represent the risk-targeted maximum considered earthquake (MCE_R) level. The study shows that, the performance of PH precast concrete wall buildings is satisfactory in terms of limiting structural and nonstructural damage under MCE_R-level far-fault ground motions, whereas, unacceptable large demands are imposed by MCE_R-level near-fault ground motions.

Keywords: *forward-directivity; fling-step; post-tensioned hybrid precast concrete wall.*

INTRODUCTION

Ground motions at a particular site, close to a ruptured fault can be significantly influenced by (i) rupture mechanism; (ii) slip direction relative to the site; and (iii) permanent ground displacement associated with the fault rupture. The ground motions with forward-directivity are generated when the rupture propagation velocity towards the site is closer to shear wave velocity and the direction of slip is aligned with the site. The ground motions with fling-step are generated by permanent ground displacement associated with the fault rupture. Ground motions with forward-directivity and fling-step cause most of the seismic energy from the rupture to arrive in a single large-amplitude, long-duration pulse at the beginning of the record. The differences in the ground motions with forward-directivity and fling-step can be clearly observed in the velocity and displacement time histories (see Fig. 1). Ground motions with forward-directivity have two-sided velocity pulse, whereas, ground motions with fling-step have one-sided velocity pulse. The buildings subjected to ground motions with forward-directivity and fling-step are exposed to larger amounts of seismic energy during a very short period of time hence, the buildings are forced to dissipate the input energy with few large nonlinear displacement cycles (Kalkan and Kunnath 2006). However, the current seismic design guidelines given in ASCE/SEI 7 (ASCE 2010) does not properly represent the effects of near-fault pulse-like ground motions, i.e., ground motions with forward-directivity and fling-step.

Effects of ground motions with forward-directivity on simplified multi-degree-of-freedom lumped mass systems (Hall et al. 1995; Alavi and Krawinkler 2004), reinforced concrete (RC) buildings (Calugaru and Panagiotou 2012), steel buildings (Kalkan and Kunnath 2006; Vafaei et al. 2015), and base-isolated buildings (Pant and Wijeyewickrema 2013) have been investigated in the past. Previous studies have indicate that response of structures is substantially influenced by the ground motions with forward-directivity. Moreover, several examples of extensive structural damage from past earthquakes may also have been caused by forward directivity. The Olive View Hospital, which sustained significant damage during the San Fernando, California, 1971 earthquake, is one of the first instances of buildings collapse associated with the near-fault directivity.

The effects of ground motions with fling-step on RC shear wall buildings (Ventura et al. 2011) and steel buildings (Kalkan and Kunnath 2006; Vafaei and Eskandari 2015) are available in the literature. Previous studies have indicate that response of structures is substantially influenced by the ground motions with fling-step

[1] *Graduate Student, Department of Civil and Environmental Engineering, Tokyo Institute of Technology, Tokyo, Japan, e-mail: samithbuddika.h.aa@m.titech.ac.jp*
[2] *Associate Professor, Department of Civil and Environmental Engineering, Tokyo Institute of Technology, Tokyo, Japan, e-mail: wijeyewickrema.a.aa@m.titech.ac.jp*

DAMAGE EVALUATION OF PRESTRESSED CONCRETE BEAMS POST-TENSIONED WITH UNBONDED TENDONS

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H. Watanabe[1], S. Kono[2], T. Obara[33], L. Martinelli[4], and G. Gioia[5]

SUMMARY

In major earthquake, many reinforced concrete buildings were damaged. Some of the buildings had severe economical losses due to structural or nonstructual damage until after the rehabilitation has been completed, even though the buildings did not collapse. For this reason, demand level for structual design shifted to be higher in japanese society. In order to develop such high perfomance buildings, this study focuses on the unbonded prestressing structures as self-centering system. The structures are able to sustain seismic force with small residual deformation. In PRESSS research program, a hybrid system with self-centering system and energy dissipation devices was developed. The small residual deformation and structural performance of the hybrid system were confirmed by structural tests. However, few studies have been reported on evaluating damage after earthquake for the unbonded prestressing structures. In order to fully take advantage of low damage characteristics of the self-centering system, it is important to investigate structural performance and damage after the earthquakes. Thus, this study report on evaluating damage of post-tensioned precast concrete members.

In order to investigate effect of effective prestressing force ratio and shear span ratio on damage or hysteresis characteristics, four specimens were made. A specimen was designed to investigate structural performance of a repaired beam after an earthquake. The structural experiment was conducted using the specimens under cyclic static loading. As an experimental result, small residual drift angle is observed of all specimens. The Shear force versus drift angle responses had S-shape hysteresis behavior which has small amount of energy dissipation

The 2015 AIJ prestressed concrete guidelines define four limit states (Serviceability limit state, Reparability limit state I, Reparability limit state II and Safety limit state) and damage evaluation criteria for the limit states. Damage evaluation was conducted using the experimental results and these criteria. As an evaluation result, effective prestressing force ratio is one of important factors in order to control damage in design for unbonded prestressed concrete beams. Furthermore, except for one specimen, the drift angle at serviceability limit state ranged from R=0.5% to R=1.0%. All specimens did not reach safety limit state until R=4%. Thus, excellent low damage performances of unbonded prestressed concrete beams were confirmed.

Keywords: *prestressed concrete beams; unbonded tendon; damage evaluation; limit state*.

INTRODUCTION

In the 1995 Kobe Earthquake and the 2011 Tohoku Earthquake, many reinforced concrete buildings were damaged. Some of the buildings had severe economical losses due to structural or nonstructual damage until

[1] *Assistant Professor, Urban Disaster Prevention Research Core, Tokyo Institute of Technology, watanabe.h.as@m.titech.ac.jp*
[2] *Professor, Urban Disaster Prevention Research Core, Tokyo Institute of Technology, kono.s.ae@m.titech.ac.jp*
[3] *Graduate student, Dep. of Architecture and Building Engineering, Tokyo Institute of Technology, obara.t.ac@serc.titech.ac.jp*
[4] *Associate Professor, Dep. of Civil and Environmental Engineering, Politecnico di Milano, luca.martinelli@polimi.it*
[5] *Professional Engineer, gianluca.gioia@outlook.com*

DAMAGE CONTROLLING PERFORMANCE OF A FULL SCALE UNBONDED POST-TENSIONED PRECAST CONCRETE BEAM

T. Obara[1], H. Watanabe[2], S. Kono[3] and T. Okayasu[4]

SUMMARY

Twenty-some years ago, it was considered that safety was a major and only concern in designing building structures in an engineering society. However, the 1994 Northridge earthquake and 1995 Kobe Earthquake revealed that the general public does not share the same vision with the engineering society. Urban earthquakes of last twenty years made the general public and the engineering society confident that structures need to perform in a higher standard, that is, damage of structures should be minimal so that structures should keep their functions after multiple earthquakes. As one of the solutions, there is a self-centering rocking system using unbonded post-tensioned precast concrete members. Unbonded post-tensioned precast concrete system is able to reduce cracking as well as residual deformation/displacement and maintain structural functions for its self-centering property. In this research, a static loading test was conducted on a full-scale unbonded post-tensioned precast concrete H-shaped beam-column assemblage. The objective was to quantitatively evaluate the damage at different loading stages. The 600mm x 1000mm beam with clear span length of 11 meters was post-tensioned with four unbonded strands to column at both ends. Two columns had pin-joints at their top and bottom to simulate the intermediate story of a building. Two point loads of 230kN each were applied to the beam to simulate vertical load due to dead and live load. The experimental results showed satisfactory seismic performance with minor damage and less residual drift and the specimen conformed to requirements for continuous use of members defined in the AIJ guidelines. In addition, the residual crack width, residual deformation drift and equivalent damping ratio were evaluated using the AIJ guidelines to study the validity of the criteria. It was concluded that a full-scale unbonded post-tensioned precast concrete beam possessed good seismic performance of quick recovery and functional maintenance.

Keywords: *a full scale frame; unbonded post-tensioned precast beam; damage controlling ability; hysteresis loop.*

INTRODUCTION

In the 1994 Northridge earthquake and 1995 Kobe Earthquake, even if buildings did not collapse, many buildings were not able to serve their functions due to various types of structural and nonstructural damage. The economic loss during this downtime sometimes overpass the building repair cost. In 1981, a new design law of Japan required the minimum performance level in which the building should continue in operation due to medium earthquakes and not collapse due to severe heavy earthquakes. However, the society demands have been shifting to higher levels recently. Therefore, reduction of damage of buildings and quick recovery to maintain the continuity of the building functions are very important issues. One of solutions is base isolation, but the system has high initial cost and extra space, so it cannot be applied for all kinds of buildings. Another solution is a

[1] *Ph.D. candidate, Tokyo Institute of Technology, obara.t.ac@m.titech.ac.jp*
[2] *Assistant Professor, Tokyo Institute of Technology, watanabe.h.as@m.titech.ac.jp*
[3] *Professor, Tokyo Institute of Technology, kono.s.ae@m.titech.ac.jp*
[4] *Research Engineer, Technical Research Institute, Kajima Corporation, okayasu@kajima.com*

MINIMUM COLUMN DEPTH FOR ACCEPTABLE BOND PERFORMANCE OF STRAIGHT BEAM BARS IN BEAM-COLUMN JOINTS

Hung-Jen LEE[1], Hsi-Ching CHEN[2], and Tsung-Chien TSAI[3]

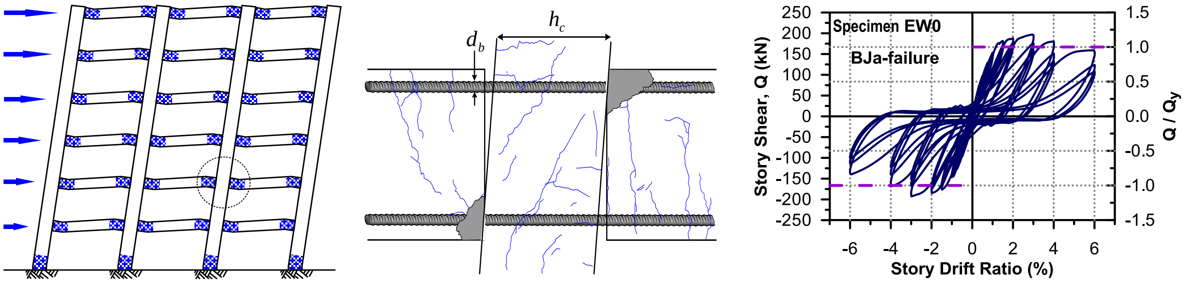
SUMMARY

The use of higher strength reinforcement in earthquake-resistant concrete structures becomes attractive because of congestion relief and cost savings, but it also brings challenges of development and anchorage of reinforcing bars in beam-column joints. This paper reviewed existing design provisions for the development length of straight beam bars in beam-column joints and then proposed a viable set of design equations for the use of high-strength reinforcement. According to ACI standards, the validity of the design equations is assessed by hysteresis performance of beam-column joint tests collected from laboratories in high-seismic areas. Although the investigated database covers a wide range of material strengths, it is recommended to extend the design equations for reinforcing bars with specified yield strength not exceed 700 MPa.

Keywords: *Seismic design; reinforced concrete; Beam-column joints; Bond; Development length; Reinforcing steel.*

INTRODUCTION

Special moment-resisting frames are widely used for the design of reinforced concrete building structures in moderate to high seismic zones. In common design practice, the plastic hinges can be arranged to develop at the beam ends adjacent to joints of a lateral-force-resisting frame, as shown in **Figure 1(a).** During the formation of these beam plastic hinges, the joint concrete resists very high shear stress and bond stress introduced by the beam longitudinal reinforcement. The bond stress is particularly demanding at interior beam–column joints, where the beam bar is subjected to tension at one face of the joint and to compression at the opposite face. Therefore, two primary failure modes, joint shear failure and bond failure, are considered in the current design standards and codes. Once certain degree of bond deterioration occurred within the joint, these beam bars may slip within the joint under large load reversals, resulting a pinched hysteresis behavior, as shown in **Figures 1(b) and (c).**



(a) Beam plastic hinging for a lateral-force-resisting frame
(b) Bond failure in the joint and concrete crushing at beam ends
(c) Cyclic response of an interior beam-column joint with bond failure

Figure 1. Bond failure of interior beam-column joints for lateral-force-resisting frames.

[1] *Associate Professor, National Yunlin University of Science and Technology, Taiwan, e-mail: leehj@yuntech.edu.tw*
[2] *PhD candidate, National Yunlin University of Science and Technology, Taiwan, e-mail: hsiching@nantou.gov.tw*
[3] *Associate Professor, National Yunlin University of Science and Technology, Taiwan, e-mail: tsaitc@yuntech.edu.tw*

EXPERIMENTAL INVESTIGATION ON THE SEISMIC ANCHORAGE BEHAVIOR OF HEADED BARS BASED ON FULL-SIZE SPECIMENS OF EXTERIOR AND INTERIOR BEAM–COLUMN JOINTS

Chien-Kuo CHIU[1], Kai-Ning CHI[1], and Ker-Chun LIN[2]

SUMMARY[1]

The design of headed bars in interior and exterior beam–column joints to ensure satisfactory seismic anchorage performance has recently become a very important issue. This work investigates the seismic anchorage behavior of headed bars using 12 groups of full-size specimens of exterior and interior beam–column joints and suggests design requirements for headed bars in such joints. On the basis of the experimental investigation, this work comprehended the seismic anchorage behavior of headed bars using full-size specimens of exterior and interior beam–column joints and suggested design requirements for headed bars in such joints. Additionally, the application of headed bars with spliced and butted types on interior beam–column joints is also proved in this work. Therefore, based on the experimental results, the minimal required net spacing of headed bars can be set at 2.2d_b, instead of 4.0d_b, for seismic designs, and the minimal required anchorage length in ACI 318-11 for non-seismic design members can be used in seismic members.

***Keywords**: anchorage; beam-column joint; experiment; headed bar; reinforced concrete.*

[1] *Professor, National Taiwan University of Science and Technology, Taiwan, e-mail: ckchiu@mail.ntust.edu.tw*
[2] *Researcher, National Center for Research on Earthquake Engineering, Taiwan, e-mail: kclin@ncree.narl.org.tw*

DIAGONAL CRACK WIDTH OF RC BEAMS WITH HIGH STRENGTH STIRRUP

Jung-Yoon Lee[1], Seong-Ho Choi[2], and Do Hyung Lee [3]

SUMMARY

Primary national codes of standard for reinforced concrete members and structures such as ACI 318-11, EC-04, CSA-04 and JSCE-02 specify a limited value of stirrup yield strength. Salient reason of this restriction is to control diagonal crack width, leading to shear tension failure. Subsequently, influence of high yield strength of stirrup on the failure mode of such members is of great concern. To investigate this, a total of 12 reinforced concrete beam specimens incorporating high yield strength of stirrups are tested. Test variables employed in the present study are three levels of stirrup spacing and four levels of stirrup yield strength. Assessment of test results is conducted on variation of both spacing and a number of diagonal cracks due to the increment of total amount of stirrups. Also performed is evaluation on diagonal crack width measured under service load level. The width is compared with a recommended value by ACI 224R-01. Test results show that the diagonal crack width under service load level is affected by yield strength of stirrups and stirrup spacing. In addition, a number of diagonal cracks are increased and spacing of the cracks is reduced, as the amount of stirrups is increased. In all, average diagonal crack width of all the specimens incorporating high yield strength of stirrups is lower than 0.41mm, recommended value by ACI 224R-01 under service load level.

Keywords *: RC beams; diagonal crack; high strength stirrup; shear behavior*

INTRODUCTION

Compressive strength of concrete used in reinforced concrete (referred to as RC hereafter) structures has been rapidly increased during over last 30 years. This has led to the fact that up to 80MPa of compressive concrete strength is employed for RC members and structures in everyday construction. Whereas the concrete compressive strength is higher, yield strength of reinforcement is kept as constant, i.e. between 300 and 500MPa of normal yield strength. In these days however, construction of large structures such as highrise buildings, long-span bridges, nuclear power plant structures, LNG storage tanks and so on requires large amount of reinforcements. Say for instance, approximately 70 to 100 thousand tons are needed for the construction of a nuclear pressure vessel, and more than 35mm of large diameter reinforcements are employed to construct RC skyscraper buildings of greater than 500ft. Accordingly when normal yield strength of reinforcement is employed in the construction of such structures, very large amount of reinforcements are required and thus congestion of reinforcements cannot be avoided during construction. Moreover, price of reinforcements is increasing nowadays due to the high price of raw materials. Such features continually demand for the employment of high yield strength of reinforcement.

Much research on the high yield strength of reinforcement has been conducted in USA, Japan, Taiwan and Korea for last two decades. As representative cases, research on Grade 100 reinforcement has been actively carried out in USA for last over 10 years (Munikrisha et al. (2011); Hosny et al. (2012); Briggs et al. (2007); Seliem et al. (2009)). In particular, performance evaluation of Grade 100 reinforcement has focused on shear and

[1] *Professor, School of Civil and Architectural Engineering, Sungkyunkwan University, Republic of Korea, e-mail: jungyoon@skku.edu*
[2] *Engineer, Samsung C&T, Engineering & Construction Group, Republic of Korea, e-mail: gmfmsmsekf@naver.com*
[3] *Professor in the Department of Civil, Environmental, and Railroad Engineering at Paichai University, Republic of Korea, e-mail: dohlee@pcu.ac.kr*

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SEISMIC TESTS OF REINFORCED CONCRETE COLUMNS CONFINED WITH A FRP-WRAPPED SPIRAL CORRUGATED TUBE (FWSCT)

Chung-Che CHOU[1], Chung-Sheng LEE[2], Kai-Yi WU[3] and V-Liam CHEN[3]

SUMMARY

This paper presents seismic tests of circular reinforced concrete columns confined with a FRP-Wrapped Spiral Corrugated Tube (FWSCT) for bonding between the FRP and the concrete core. Three column specimens named as FWSCT series used only longitudinal reinforcement to provide their flexural capacities but did not use transverse steel hoops in the entire column. Instead, Specimen FWSCT-0 was confined with a spiral corrugated steel tube; Specimens FWSCT-5 and FWSCT-8 were confined with a spiral corrugated steel tube and additional 5 and 8 GFRP layers, respectively. Test results showed that Specimen FWSCT-0 experienced shear failure at about 1% drift, while Specimens FWSCT-5 and FWSCT-8 exhibited rupture of longitudinal reinforcement at drifts of 6% and 8%, respectively, without shear failure of concrete columns. It was found that the plastic hinge of FWSCT concrete columns was developed only at top and bottom of the column end (i.e. 30 mm), extending into the footing. The analytical model was developed to predict test results by using the observed plastic hinge length.

Keywords: *Fiber Reinforced Polymer (FRP), FRP-wrapped Spiral Corrugated Tube, Concrete Columns, Cyclic Test, Shear Failure*

INTRODUCTION

Fiber Reinforced Polymer, FRP, has been gradually applied to civil engineering structures, especially for retrofitting structural members (Priestly and Seible 1991; Xiao and Ma 1997). Early applications have been focused on retrofitting reinforced concrete columns or beams in bridges and buildings. Nonetheless, the light weight, high strength or stiffness-to-weight ratio, good corrosion resistance, easy handling and installation are making FRP widely adopted in the development of new structural members (Teng et al. 2007; Gould and Harmon 2002). There are three main advantages of using FRP in new structural application: (1) time and manpower saving, for the FRP tube used as a mold of concrete, (2) corrosion-resistant for a protective layer of concrete core, and (3) great confinement due to its linear elasticity and high tension capacity.

Many works have confirmed that using FRP as confinement to concrete columns can significantly increase the flexural and shear strengths of columns, also increasing the column ductility [Innamorato et al. 1995; Seible et al. 1997; Sheikh and Yau 2002; Teng et al. 2007; Shi et al. 2007; Wang et al. 2011]. A composite shell system (CSS) that is used as a structural member to confine concrete was developed early at the University of California, San Diego [Burgueno et al. 1995]. The CSS has a ribbed inner surface for bond and force transfer to the concrete core. The concrete filled CSS for seismic zones is comparable to or better than that of conventionally reinforced concrete columns. However, the application of the ribbed inner surface in the CSS increases the difficulty in the fabrication of FRP tubes. This study presents an alternative to the development of FRP composite wrapped grooved-wall tube that has an inner steel corrugated tube to serve as a permanent ribbed surface between the FRP shell and the concrete core [Chou et al. 2016].

[1] *Professor, Department of Civil Engineering, National Taiwan University, Taiwan, e-mail: cechou@ntu.edu.tw*
[2] *Associate Research Fellow, National Science and Technology Center for Disaster Reduction, Taiwan.*
[3] *Graduate Student Researcher, Department of Civil Engineering, National Taiwan University, Taiwan.*

IN-PLANE CYCLIC BEHAVIOR OF STEEL-PLATE COMPOSITE WALLS WITH BOUNDARY ELEMENTS

Yin-Nan Huang[1], Chang-Ching Chang[2], Yu-Cheng Cheng[3], and Chi-An Ho[4]

SUMMARY

This paper summarizes the results of in-plane cyclic tests of three steel-plate composite (SC) walls with boundary elements. The tests were executed in the laboratory of National Center for Research on Earthquake Engineering (NCREE) in Taiwan. Thick steel plates were used as boundary elements of the three specimens. To study two kinds of failure modes, we included in the test 1) two shear-critical walls with an aspect ratio of 0.75 and a thickness of 3 cm for boundary elements; and 2) one flexure-critical wall with an aspect ratio of 1.22 and a thickness of 2 cm for boundary elements. The failure mode and cyclic behavior of each specimen are reported. The test results are compared with the recommendations of AISC N690s1 and predictions of selected literatures. The impact of wall aspect ratio on in-plane shear strength of SC walls with boundary elements is discussed.

Keywords*: steel-plate composite wall; boundary element; cyclic loading test; aspect ratio; LS-Dyna*

INTRODUCTION

Steel-plate-concrete (SC) composite walls are being constructed in nuclear power plants (NPPs) in the United States and China. These walls are composed of steel faceplates, infill concrete, welded connectors that tie the plates together and provide out-of-plane shear reinforcement, and shear studs that enable composite action of the faceplate and the infill concrete and delay buckling of the faceplate. The use of SC walls in safety-related nuclear facilities in Korea, Japan, and the United States has been studied (e.g., Ozaki et al. 2004, Epackachi et al. 2014, Varma et al. 2014) for the past 20 years. In earlier days, most of numerical studies and test data in the studies of SC walls have focused on the elastic range of response, because NPPs are designed to remain elastic under design basis shaking. Since a decade ago, the nonlinear behavior of SC walls has drawn more and more attention.

A SC wall is very often connected with perpendicular SC walls at the ends. The perpendicular walls become the boundary elements of the longitudinal wall. Ozaki et al. (2004) and Varma et al. (2014) developed an approach to predict the yield point of an SC wall subjected to in-plane later force using composite shell theory. The approach was simplified and codified in AISC N690-12s1 (AISC 2015) for the purpose of design. Booth et al. (2015) further proposed that the in-plane shear strength of a SC wall with boundary elements should include two parts: the shear force required to yield the steel plates and an incremental shear resisted by the concrete in diagonal compression up to the failure of the wall. The study on the nonlinear flexural-critical behavior of SC walls with boundary elements is relatively rare.

[1] *Associate Professor, National Taiwan University, Taiwan, e-mail: ynhuang@ntu.edu.tw*
[2] *Assistant Researcher, NCREE, NARL, Taiwan, e-mail: ccchang@ncree.narl.org.tw*
[3] *Former Graduate Student, National Taiwan University, Taiwan, e-mail: r03521239@ntu.edu.tw*
[4] *Former Graduate Student, National Taiwan University, Taiwan, e-mail: r03521205@ntu.edu.tw*

SEISMIC STRENGTHENING OF RC WALLS BY UHPFRC JACKETING

Sung-Gul Hong[1] and Yu-Sun Yi[2]

SUMMARY

This paper investigates the behavior of RC(reinforced concrete) walls strengthened by UHPFRC(ultra-high performance fiber reinforced concrete) jacketing under lateral loading. To upgrade stiffness and strength of existing wall structures subjected to earthquake loading, the retrofit strategy for walls are reviewed from constructability in-situ. The continuity of reinforcement from foundation definitely increases upgrading of capacity of walls. However, this study investigates the increase in stiffness and strength with ductility of jacketing UHPFRC for walls with discontinuity of reinforcement along the interface between added UHPFRC parts and foundation concrete block. Three specimens for walls were prepared: reference wall, retrofitted wall, and retrofitted wall with slit. The slit was made to reduce the flexural tension force by changing of load paths. The analysis of experimental results showed the jacketing of UHPFRC with the discontinuity of reinforcement at the bottom of walls increase structural capacity of existing RC walls such as strength, stiffness and energy dissipation under cyclic loading.

***Keywords**: Retrofit by jacketing; strengthening of RC walls; UHPFRC; discontinuity of longitudinal reinforcement.*

INTRODUCTION

Cantilever wall structures have provided efficient lateral resistance systems with architectural layout requirement in high-rise residential buildings, especially in Korea. The architectural functions require free-edge wall-ends in cantilever walls rather than use of boundary elements or flanges. These sections are susceptible to crushing failure due to insufficient confinement and thereby instability of the plastic regions of relatively thin walls due to out-of-plane buckling. To prevent premature crushing failure at extreme within plastic hinge region, detailing and minimum dimension of boundary of thin walls is recommended in design provisions (UBC; 1997).

Extensive research on the behavior of wall buildings after Chile Earthquake results in a new design provisions developed by Wallace (1993; 1998) and Moehle (1989; 1992) who pointed out overly conservative design provision for slender walls. Based on these conclusions a displacement-based design methodology was developed and implemented in UBC (1997) and ACI Building Code (1999). However, Sasani (1998) insisted that the effect of concrete strain concentration caused by cracks and ignoring the effects of cyclic loading the wall displacement capacity and UBC design provisions estimate the displacement capacity of structural walls by more than a factor of two.

Among current retrofit techniques for existing reinforced concrete (RC) structures, one of viable options is to use UHPFRC (ultra-high performance fiber reinforced concrete) or UHPC (ultra-high performance concrete) as thin layer over surfaces of RC members due to its notable high strength with cost-effectiveness. Currently, researchers reported practical application of UHPC for the strengthening of existing concrete structures ; 5 mm U-jacket for flexural strengthening (Martinola, Meda, Plizzari, & Rinaldi, 2010) and shear strengthening (Meda, Mostosi, & Riva, 2014) with 3 or 5 mm U-jacketing with wire mesh showed increase in strength and ductility. Furthermore, the retrofit by overlay (Noshiravani, 2012) using UHPC with reinforcing bars of small diameter and the thickness of 10-20% of the thickness of RC members (Habel, Denarié, & Brühwiler, 2006) with numerical analysis have demonstrated significant increase in strength. These strengthening methods relied on tensile strengthening of UHPC with added reinforcing bars. The overlay of UHPC on top faces of slabs and beams contributes to increases in both the negative moment capacity and positive moment capacity.

[1] *Professor, Seoul National University, Korea, e-mail: sglhong@snu.ac.kr*
[2] *Formerly graduate student, Seoul National University, Seoul, Korea,*

SEISMIC DESIGN AND EARTHQUAKE DAMAGE MITIGATION OF INDUSTRIALIZED RESIDENTAIL BUILDINGS

Chikara IIHOSHI[1]

SUMMARY

Japan is seismic prone all over the country and was attacked by several large earthquakes such as Kobe in 1995 (JMA 1997) and Kumamoto in 2016 (USGS 2016). Most people who live in Japan are concerned with earthquake resistance of their houses. The 2011 Tohoku earthquake (JMA 2011) enhanced their worries and anxieties about severe ground motions and their interests in seismic damage mitigation of the residential buildings. Many newly constructed low-rise prefabricated homes in the country got to be designed and built with some kinds of dampers in order to remain low damage even after severe ground motions induced by strong earthquakes. Asahi Kasei Homes (AKH) Corp. is a housing company that first applied dampers to standardised prefabricated residential buildings in Japan (possibly in the world). "Hebel Haus" (its brand name) is the most popular prefab home in Tokyo. And its three-story building is the most constructed mid-rise residential building in Japan. The prefabricated elements used in the prefab buildings include cold-formed steel (CFS) columns, H-shaped beams and autoclaved aerated concrete (AAC) panels as floor/roof diaphragms as well as exterior walls. Dampers installed in the structural systems are low yield steel dampers for the low-rise homes and viscous dampers for the middle-rise ones respectively. The structural and non-structural systems of the Japanese chemical background housing company are presented in details.

***Keywords**: industrialized building; prefabricated building system; damage mitigation; autoclaved aerated concrete panel.*

INDUSTRIALIZED HOUSING IN JAPAN

"Industrialized Housing" shall mean a residential structure that is designed for the occupancy of one or more families; constructed in one or more modules or constructed using one or more modular components built at a location other than the permanent site; and designed to be used as a permanent residential structure when the module or the modular component is transported to the permanent site and erected or installed on a permanent foundation system (Texas Department of Licensing and Regulation 2015). Figure 1 shows annual number of newly built houses in Japan (Japan Ministry of Land, Infrastructure, Transport and Tourism 2015). The country's housing market has shrunk since 1972 as a whole as the county has declined in population. Recent market size is about 800,000 units annually. Figure 2 displays factory-based home industry in Japan over the 10 years. The market share has slightly increased in the period. The 140,000 factory-built homes were sold in 2014 represent about 15 percent of the Japanese housing market. Most houses are still produced by traditional "post-and-beam" frame methods while 13% homes built with American 2 by 4 inch wood stud techniques.

Figure 3 shows comparison of average lifetime of a house among nations. The average lifetime in Japan is 25 years. Meanwhile house lifetime in US and UK are longer than 60 years respectively which means the average lifetime in Japan is less than half that in US also one-third UK. Some foreign people believe it is because of the Shinto belief in the impermanence of things (*wabi-sabi*) and because the seismic regulations are always evolving.

[1] *Chief Researcher, Asahi Kasei Homes Corp., Japan, e-mail: iihoshi.cb@om.asahi-kasei.co.jp*

2015 Three-dimensional Shaking Table Test of a 10-story Reinforced Concrete Building on the E-Defense

Y. Tosauchi[(1)], E. Sato[(2)] , K. Fukuyama[(3)] , T. Inoue[(4)], K. Kajiwara[(5)]

SUMMARY

The E-Defense, which is the world's largest three-dimensional (3D) full-scale earthquake shaking table test facility, was built by the National Research Institute for Earth Science and Disaster Resilience (NIED).In December 2015, NIED tested a 10-story reinforced concrete building frame on the E-Defense in order to gain building engineering knowledge that will facilitate continued use of damaged buildings after a major earthquake. In this experiment, data were obtained from a structure equipped with a base slip mechanism in order to examine the efficacy of the base slip method. After the base slip construction test, the base of the same specimen was fixed in place to simulate conventional construction conditions and testing was conducted in order to compare fixed-in-place behavior with the base slip response behavior, determine the damage process of each member, and examine damage and response evaluation methods.

This paper shows the results of the base slip and fixed base tests. In the base slip construction test, the base was observed slipping while base uplift occurred, which may have caused the base to slip with torsional movement. The maximum story drift angle generated in the specimen under 100% amplitude of the JMA-Kobe excitation was 0.0060 rad for the base slip construction test, which is relatively small compared to 0.0305 rad, which was measured during the fixed specimen foundation test. Additionally, for both tests, the story shear force generated at each floor is clearly larger when compared to the story shear force calculated during the specimen design for the same story drift angle. In the fixed base test, the beam and column main reinforcements that were arranged in cross-shaped joints yielded, and damage was concentrated at beam-column joints from the 3rd to 5th floors, where relatively large story drift angles were generated.

***Keywords**: Earthquake resistance, Damage mitigation, Base slip*

(1) Researcher, Fujita Corporation
(2) Chief Researcher, National Research Institute for Earth Science and Disaster Resilience
(3) Visiting researcher, National Research Institute for Earth Science and Disaster Resilience
(4) Vice director, National Research Institute for Earth Science and Disaster Resilience
(5) Director, National Research Institute for Earth Science and Disaster Resilience

1. Introduction

The E-Defense, which is the world's largest three-dimensional (3D) full-scale earthquake shaking table test facility, was built by the National Research Institute for Earth Science and Disaster Resilience (NIED) with the aim of shedding light on the failure mechanisms of full-scale structures during earthquakes and for verifying the effects of seismic retrofitting. Since its start of operations in April 2005, a wide variety of structures have been tested on this facility.

As part of its "Social Infrastructure Research Utilizing the 3D Full-Scale Earthquake Testing Facility" project, NIED conducted shaking table tests on a 10-story reinforced concrete building frame in December 2015. Previously, NIED had also conducted numerous tests on reinforced concrete

PARAMETER IDENTIFICATION OF FULL-SCALE STEEL BUILDING EXPOSED TO FIRE THROUGH SHAKER AND BASE-ISOLATED PLATFORM

Shih-Yu CHU[1], Yao-Yu YU[2], Yi-Xin WAND[3] and Chien-Tai SHIH[4]

SUMMARY

Architecture and Building Research Institute (ABRI) of Taiwan planned to build a large outdoor fire research facility in the Kuei-Jen campus of National Cheng Kung University. This specific project includes a base-isolated reinforced concrete (RC) platform and a single-story full-scale steel building specimen. The platform's length is 18 meters and the width is 12 meters. There are 12 low-friction pot bearings installed between the RC platform and the raft foundation. A cost-effective outdoor shaking table is designed by adopting an eccentric mass shaker. The base-isolated platform is planned to build a 4 meters high full-scale steel building specimen on top of it, its length and width are both 12 meters with two spans. The major purpose of abovementioned project is focusing on fire-earthquake compound disasters especially. This study tries to establish a framework of parameter identification procedures for each components of the proposed shaker-RC platform-steel specimen combined system. A commercial software is used to simulate the dynamic responses of the combined system subjected different shaker loadings. This study also focuses on the fire damage evaluation of the full-scale steel specimen under different fire scenarios. The dynamic parameters of the fire-damaged specimen can be identified through appropriate arranged eccentric mass shaker excitations.

Keywords*: Fire-earthquake Compound Disasters, Full-Scale Steel Building, Base-Isolated Platform, Pot Bearing, Eccentric Mass Shaker, Torsionally Coupled Building.*

INTRODUCTION

Through the comparisons and verifications of the full-scale fire experimental data, we can systematically and correctly understand the responses and variations of real buildings subjected to fire. However, the structural fire tests adopted by the specifications and codes for the time being only focus on testing and evaluating a single structural component subjected to the fire according to the standard elevating temperature-time curve. For more complicated buildings and fire scenarios, this kind of fire tests usually cannot provide the fire responses of the whole building. Besides, the temperature distribution of the heat flow field for a building in fire varies due to different fire ignition methods, fire spreading speeds and fire spreading scenarios. The temperature distribution of a structural component, such as beam or column, is not uniform in a building fire because of its relative location to floor and wall, and its exposed surfaces to fire. This special kind of thermal-stress condition causes the strength variation of a building structure after fire, and has a large influence for the structural safety of a post-fire building structure. Therefore, the best fire assessment method is to use the full-scale building structure subjected to the real fire scenarios for verification.

[1] Associate Professor, Department of Civil Engineering, National Cheng Kung University, Taiwan, e-mail: sychu@mail.ncku.edu.tw
[2] Master of Science, Department of Civil Engineering, National Cheng Kung University, Taiwan, e-mail: jimblue14836@hotmail.com
[3] Graduate Student, Department of Civil Engineering, National Cheng Kung University, Taiwan, e-mail: stanleyapply@gmail.com
[4] Registered Professional Civil Engineer and Part-time Postdoctoral Fellow, Department of Civil Engineering, National Cheng Kung University, Taiwan, e-mail: sw789@seed.net.tw

STEEL BELT TRUSS-CONCRETE MEGA COLUMN CONNECTION IN LOTTE WORLD TOWER

Hong-Gun Park[1], Jang-Woon Baek[2], Hyun-Jong, Hwang[3], and Tae-Sung Eom[4]

SUMMARY

The structural performance of a steel belt truss-to-concrete mega column connection in the 123-story Lotte World Tower was studied. For the connection, a four-story-high steel belt truss is connected to the exterior face of the concrete mega columns by hundreds of shear connectors that consist of embedded shear plates and dowel bars. In this study, shear tests were performed for short connection specimens with a unit length of 900 mm. The peak strength, residual strength, and stiffness were investigated using various shear transfer mechanisms such as concrete bearing, shear friction, and strut-and-tie mechanism.

***Keywords**: Shear connection; shear connector; shear plate; dowel bar; mega column; and belt truss*

INTRODUCTION

The Lotte World Tower with 555 m height and 123 stories is under construction in South Korea. To support the extremely large gravity and wind loads, the primary structural system consists of an RC core wall and 8 concrete mega columns on the perimeter of the floor plan (see **Fig. 1(a)**). To resist wind load, steel outrigger trusses connect the mega columns to the core wall between the 39th and 44th floors and between the 72nd and 76th floors. In addition, a four-story-high steel belt truss is connected to the mega columns (see **Fig. 1(a)**), to transfer the large gravity load of the steel superstructures between the 76th and 107th floors. The concrete mega columns have square cross-sections varying from 3500 mm x 3500 mm on the lower floors to 2000 mm x 2000 mm on the 76th floor. For fast construction, auto-climbing forms are used for the concrete work of the mega columns, thereby minimizing any intervention of intersecting steel beams. After the concrete work, the belt truss from the 72nd to the 76th floors is connected to the exterior face of the mega columns by shear connectors (i.e. embedded shear plates and dowel bars, see **Fig. 1(b)**). In particular, since the vertical shear demand at the interface between the belt truss and mega column is as much as 120000 kN, many shear connectors are required. Therefore, the performance of the shear connectors at the interface is crucial to the structural safety of the entire structure.

In the design of the shear connection between the steel belt truss and concrete mega column used from the 72nd to the 76th floors of the high-rise building, unlike conventional shear connections, the shear demand (120000 kN) is very large. Further, as shown in **Fig. 2(b)**, large diameter bar grids of longitudinal D51 bars (diameter D_b = 50.8 mm) and transverse D19 hoops (diameter d_b = 19.1 mm) are densely placed. Thus, available connection methods that can fit the space are limited. More importantly, in the vertical four-story-high connection, unlike horizontal connections, uplift failure can cause catastrophic failure of the overall connection (see **Fig. 2(a)**). Therefore, a hybrid shear connector using embedded shear plates and dowel bars, which are simple in detailing and fabrication, was considered. By placing the rigid shear plates in the transverse direction, the shear strength and stiffness of the connection can be increased; by using the flexible dowel bars, the uplift resistance and force-redistribution capacity can be enhanced.

[1] *Professor, Department of Architecture and Architectural Engineering, Seoul National University, Korea, e-mail: parkhg@snu.ac.kr*
[2] *Ph.D Candidate, Department of Architecture and Architectural Engineering, Seoul National University, Korea, e-mail: baekja1@snu.ac.kr*
[3] *Assistant Professor, Colleage of Civil Engineering, Hunan University, China, e-mail: hwanggun85@naver.com*
[4] *Associate Professor, Department of Architectural Engineering, Dankook University, Korea, e-mail: tseom@dankook.ac.kr*

ASSESSMENT OF SEISMIC DRIFT IN TORSIONALLY-UNBALANCED BUILDINGS ACCOUNTING FOR TORSIONAL RESISTANCE BY TRANSVERSE FRAMES

Kyung Ran HWANG[1], Han Seon LEE[2], and Abegaz Ruth ALI[3]

SUMMARY

In this study, the seismic drift demand of torsionally-unbalanced buildings in a low-to-moderate seismicity region is estimated accounting for torsional resistance by transverse frames perpendicular to the direction of ground motion by using the acceleration-displacement response spectrum diagram and the planar analysis approach proposed by Lam et al (2015). The ratio of the maximum edge drift to the central drift (Δ_{max}/Δ_0) of two torsionally unbalanced structures with two different plan aspect ratios are presented by increasing the static eccentricities up to 50% in accordance with the structural response to acceleration, velocity, and displacement controlled excitations. As the contribution of the transverse frame increases by increasing the ratio of lateral stiffness of the transverse frame to that in the longitudinal frame (the ratio a), the maximum edge drift decreases. In the structure with a plan aspect ratio 2, when the ratio a increases from 0.17 to 1.6 at $e_{sx}/2B$ =25%, the ratio Δ_{max}/Δ_0 decreases from 1.9 to 1.7 for the velocity controlled condition. The corresponding roof drift ratios are 0.540% and 0.493%. Although the ratios Δ_{max}/Δ_0 significantly exceed the design limit value, 1.2, the roof drift demands, Δ_{max}, appear to be in the elastic range.

***Keywords**: asymmetric building, torsion, shear, dynamic, earthquake, eigenvalue analysis*

INTRODUCTION

The severe damage and collapse of building structures in earthquake can occur through several phenomena, one of which is the torsion due to the eccentricity between the inertia force and resisting force. To prevent excessive deformation, damage, and collapse caused by torsion, the current seismic design code specifies two torsion design approaches: First, is the use of equivalent lateral force (static) procedure, where the structure is designed by the most adverse combination of the seismic forces (V_D) and the design eccentricities (e_d) given in Eqs. (1) and (2) as shown in Fig. 1(a). Second, is the use of the dynamic analysis such as modal response spectrum analysis or time history analysis, where the center of the mass (CM) at each story is assumed to be the positions, CM1~CM4 in Fig. 1(b). The most adverse results in deformations and member forces of the structure obtained from the dynamic analysis are used for design.

$$V_D = C_S W \tag{1}$$

$$e_d = \alpha e_s + \beta l \quad \text{or} \quad e_d = \delta e_s - \beta l \tag{2}$$

where, e_d is the design eccentricity composed of static and accidental eccentricities; e_s is the static eccentricity determined as the distance between the CM and CS (center of stiffness); βl (=e_a) is the accidental eccentricity, which is included to consider torsional effects due to the uncertainty of CM and CS, the rotational component of ground motion, and other uncertainties not explicitly considered; l is the plan dimension of the building perpendicular to the direction of ground motion; and α, β, and δ are code-specified coefficients.

[1] *Research professor, School of Civil, Environmental, and Architectural Engineering, Korea University, Korea, e-mail: dh8149@korea.ac.kr*
[2] *Professor, School of Civil, Environmental, and Architectural Engineering, Korea University, Korea, e-mail: hslee@korea.ac.kr*
[3] *Graduate student, School of Civil, Environmental, and Architectural Engineering, Korea University, Korea, e-mail: ruthali@korea.ac.kr*

SEISMIC RETROFIT AND DESIGN CONSIDERATION FOR WELDED STEEL MOMENT CONNECTIONS WITH HIGHLY COMPOSITE SLABS

Cheol-Ho Lee[1], Sung-Yong Kim[2]

SUMMARY

In the 1994 Northridge earthquake, connection damage initiated from the beam bottom flange was prevalent. The presence of a concrete slab and resulting composite action was speculated as one of the critical causes of the prevalent bottom flange fracture. Close review of past experimental studies recently conducted by the author clearly indicated that conventional seismic steel moment connections with highly composite slabs were much more vulnerable to the bottom flange fracture. In this study, seismic retrofit schemes are presented for welded steel moment connections with highly composite floor slabs typical of existing steel moment frames in Korea. Because top flange modification of existing beams is not feasible due to the presence of a concrete floor slab, bottom flange modifications by using welded triangular or straight haunches, and by beam web strengthening with heavy shear tab were cyclically tested and analyzed. Test results of this study showed that the retrofit schemes used is effective in eliminating the detrimental effects associated with high composite action and can ensure excellent connection plastic rotation exceeding 4% rad. Side effects as a result of each retrofit scheme as well as design recommendations are also presented.

Keywords: *Seismic retrofit; Welded steel moment connection; Slab effect; Heavy shear tab; Straight haunch; Triangular haunch.*

INTRODUCTION

Most of steel moment frame buildings have composite floor slabs which usually consist of steel beam, shear studs, metal decks, steel rebars or wire meshes, and topping concrete. As was previously discussed by the authors[1], understanding their influence on seismic performance is very important for both new construction and retrofit. Composite floor slab acts as diaphragm and shear studs are provided to help transfer diaphragm loads to the steel moment frame, to laterally support the beams, and to increase overall structural continuity. For gravity beams with simple connection at both ends, it is usual practice and more economical to design them as fully composite. In this case, an adequate number of shear studs must be provided to accomplish full composite action according to the well-established design procedure. It is the moment frame beams that much attention should be paid to. In the past design and construction practice in Korea, more shear studs than needed to transfer diaphragm forces were provided for steel moment frame beams. For example, the use of a pair of shear studs of 16~19mm diameter per deck rib (or about 200 mm on center) for moment frame beams was very common even when they were designed as non-composite beams, or when composite action was not explicitly considered in computing lateral stiffness and strength of the moment frame. Typical slab details in many countries appear to be empirical and vary somewhat depending on locality. For example, providing 19-mm diameter shear studs spaced 300 mm on center is a popular US west coast practice to transfer seismic forces from the slab to steel frame.

[1] *Professor, Dept. of Architecture and Architectural Engineering, Seoul National University, #39-432, 1 Gwanak-ro, Gwanak-gu, Seoul, Korea, 151-744 e-mail:, ceholee@snu.ac.kr*

[2] *PhD Candidate, Dept. of Architecture and Architectural Engineering, Seoul National University, #39-420, 1 Gwanak-ro, Gwanak-gu, Seoul, Korea, 151-744*

THE STATE OF THE PRACTICE OF STEEL PLATE SHEAR WALLS IN KOREA

Kyungtae KIM[1], Dongyeon KIM[2], Kyung-Koo LEE[3], Taejin KIM[4]

SUMMARY

Since 1970s, research on Steel Plate Shear Walls (SPSW) have been done widely in North America and Japan and has proven that it has superior performance against Earthquakes because of its high ductility and energy dissipation rate. Furthermore, because of its thin plate, SPSW can maximize the interior space of the buildings. Advantages of SPSW over traditional lateral resisting system have been exposed to many engineers, however, number of its application is quite negligible in Korea. This paper is geared towards design engineers who might be considering using SPSW. Structural design example in Korea using SPSW is compared and further discussion regarding modeling procedures and analytical studies are presented in this paper. SPSW was modelled using orthotropic membrane model in ETABS 2016

***Keywords**: steel plate shear walls; orthotropic model; lateral system; KBC2016*

INTRODUCTION

Steel plate shear walls (SPSW) consist of a plate bounded at the sides by columns also referred to as vertical boundary elements (VBE), and at the floor levels by beams also referred to as horizontal boundary elements (HBE). The alternate nomenclature for beams and columns emphasizes the boundary elements role of resisting the tension field developed in the plate.

Steel plate shear walls has advantages compared to traditional concrete shear walls that it has higher ductility ratio and great energy dissipation rate through plastic deformation. In architectural perspective, thinner plates allow more floor area so that flexibility in architectural plan is achievable. Many researches on SPSW has been started since 1970, in Japan and North America in order to use SPSW as seismic resistance system. In the beginning stage, stiffener was used in order to prevent out-of-plane buckling of the plate. Later on, unstiffened plate introducing post-buckling strength for seismic design have been studied. (Berman 2010, Roberts 1992, Thorburn 1983, Timler 1983) Also, connection details (Elgaaly 1998), various plate geometry (Roberts 1992), multi-story tests (Caccese 1993, Driver 1998, Lin 2010), finite element analysis (Driver 1997), usage of low yield steel and perforated plate (Bruneau 2005, Vian 2004, 2005) have been adopted and become more concrete in the design code.

Number of international projects have been increasing since 2005 as design code such as AISC Seismic Provision and CSA S16 became more available to engineers. In Korea, however, SPSW is not as popular as other countries although we have the same code provision in Korean Building Code (KBC). So, this paper is geared towards design engineers who might be considering using SPSW and discusses about design considerations, validation of computer model, and design suggestions in the real engineering practice. In order to do so, we picked a typical office building located in Seoul and used this example for SPSW design.

[1] *General Manager, Chang Minwoo Structural Consultants e-mail: ktkim@minwoo21.com*
[2] *Research Engineer, Chang Minwoo Structural Consultants e-mail: dykim@minwoo21.com*
[3] *Associate Professor, Department of Architecture and Structural Engineering, Dankook University, Korea, e-mail: kklee@dankook.ac.kr*
[4] *President, Chang Minwoo Structural Consultants e-mail: taejin@minwoo21.com*

CYCLIC BEHAVIOR OF STEEL BRACES WITH SPLICED MID-SEGMENT

Chui-Hsin CHEN[1], Dong-Yin YU[2], and Chih-Chun,OU[2]

SUMMARY

Conventional steel brace dissipates energy by its tension yielding and compression buckling. Most of the inelastic deformation of a buckling brace concentrates in the mid-length. This region dissipates most energy in a brace under cyclic loading, but it is also vulnerable to failure modes associated with low-cycle fatigue. In this research, we proposed a strategy to improve the cyclic behavior of the conventional steel braces by a spliced mid-segment with more suitable sectional properties and materials. By doing so, we can prevent the local section from concentration of large plastic deformation caused by cyclic tension and compression. Research plan includes numerical investigation and experimental study, which is still in progress on wide-flange braces. The primary variables to be investigated include the length, cross-sectional shape and material of the mid-segment. Finite element analyses from ANSYS compare the distributions of von Mises stress and equivalent plastic strain (PEEQ) which are used as indices of fatigue life in this study. The preliminary results of analyses show that it is ineffective to replace the mid-segment with higher-strength material or slenderer section. Contrarily, the braces with more compact and greater Iy section in the mid-segment show more favorable distributions of von Mises stress and lower PEEQ. Comparing to the conventional unreinforced brace, we can decrease PEEQ by up to 30% for the case that more compact and greater Iy section is used for the mid-segment. If the parameters are properly designed, the fatigue-life and energy dissipation capacity of conventional buckling braces can be enhanced obviously by splicing a mid-segment. Further investigation will be conducted by performing subassembly cyclic tests.

Keywords*: steel brace; spliced mid-segment; finite element analysis; PEEQ; cyclic test.*

INTRODUCTION

Special concentrically braced frames (SCBF) have been developed over a century. It was wind-resisting structural system in the beginning and has been developed into earthquake-resisting structural system. Even today, SCBF systems are still popular among engineers. SCBF systems are widely used for office and residential buildings due to their simple design procedures and efficiency of providing high lateral stiffness. As more and more SCBF systems were used, engineers and researchers gradually identified the seismic behavior and the failure modes of SCBF systems. Such development reflected in the design codes. While the seismic performance of new SCBF buildings satisfy the current codes, that of the existing old SCBF buildings may need to be examined and seismic retrofitting may be required.. To advance seismic retrofitting techniques for existing buildings and to develop new structure with better energy dissipation capacity, this research use braces with spliced mid-segment to improve the ductility of the conventional buckling braces. The proposed braces can be used to retrofit existing buildings or applied directly to new constructions.

When a brace with prismatic cross section is under monotonic tension, theoretically, the whole length of the brace will experience nonlinear behavior and dissipate energy; however, when the brace is under compression, the nonlinear behavior only concentrates in plastic hinges which take place in the mid-span of the brace and

[1] Assistant Professor, National Chiao Tung University, Taiwan, e-mail: chchen.nctu@gmail.com
[2] Graduate Student Researcher, National Chiao Tung University, Taiwan, e-mail: chlnyloo@gmail.com; moods0707@gmail.com

RESEARCH AND DEVELOPMENT OF NATURALLY BUCKLING STEEL BRACES

Po-Chien HSIAO[1], Kazuhiro HAYASHI[2], and Masayoshi NAKASHIMA[3]

SUMMARY

Steel concentrically braced frames inherently provide great strength and stiffness, and are widely used for seismic resisting systems in buildings. These include conventional buckling braced frames and buckling restrained braced frames. Although the latter can prevent brace buckling and provide ductile behavior, both types of braces provide no hysteretic damping at small drift levels and offer very limited post-yielding stiffness. This study proposes a new type of steel brace with a novel mechanism—the naturally buckling brace (NBB). The design combines high-strength and low-yield steels arranged in parallel with a specified initial eccentricity along the brace length, providing ductile seismic behavior. Six tests of various NBB models subjected to cyclic loading were conducted to examine the seismic performance of the proposed NBB. Two specimens out of the six achieved the characteristics intended for NBB. The test results showed that the NBB specimens with appropriate design parameters could achieve early yielding, or hysteretic damping, from around 0.11% story drift and prevent local buckling as well as deformation concentration up to a very large story drift (greater than 3%). A single NBB would provide an asymmetric hysteretic behavior, a large post-yielding stiffness in tension, and a ductile performance with stable energy dissipation. Further systematic studies of NBBs are needed to comprehensively evaluate the capacities and limitations of the NBBs, including the reliability of performance with repeated tests.

***Keywords**: earthquake engineering; steel building structures; braced frames; high performance steel materials; Buckling*

INTRODUCTION

Steel concentrically braced frames (CBFs) are commonly used as seismic resisting systems in buildings in regions of high seismic activity. The CBFs can be classified as conventional buckling braced frames (CBBFs), also called special concentrically braced frames (SCBFs), and buckling restrained braced frames (BRBFs). The CBBFs allow brace buckling during moderate and large earthquakes and form plastic hinges at the mid-length section of the brace and gusset–plate connections for energy dissipation. Many previous studies have helped in understanding the seismic performance of conventional buckling braces (CBBs) (Popov et al. 1976; Black et al. 1980; Lehman and Roeder 2008). However, CBBs have an inherent nature of deformation con-centration resulting from brace buckling. The deformation first concentrates at the brace mid-length as shown in Fig. 1, and causes local buckling (cupping behavior) which rapidly increases the local strain concentration and leads to brace rupture failure Fig. 1 (Hsiao et al. 2013). The buckling restrained braced (BRB) frame was developed to avoid the deficiency resulting from brace buckling. The steel core of the BRB is embedded in a restraining component to prevent the member buckling of the brace. Thus, the BRB can develop full material yield strength under both tension and compression and provide ductile behavior. Many studies have been conducted to examine and optimize the design details of BRBs (Watanabe et al. 1988; Black et al. 2002), making BRBs the most popular metallic damper used in current building practice. However, both CBBs and BRBs still have several weak-nesses: (1) those braced frames inherently provide large stiffness

[1]Assistant Professor, Dept. of Civil Engineering, National Chung Hsing University, Taiwan, e-mail: pchsiao@nchu.edu.tw
[2]Assistant Professor, Dept. of Arch. and Civil Eng., Toyohashi Univ. of Tech., Japan., e-mail: hayashi.kazuhiro@forest.ocn.ne.jp
[3] Professor, Disaster Prevention Research Institute, Kyoto University, Japan., e-mail: nakashima@archi.kyoto-u.ac.jp

SEISMIC ISOLATED ROOF STRUCTURE OF SUITA STADIUM

Takashi KIHARA[1], Hisato OKUDE[2], Hirokazu NOZAWA[3], and Masahito OHNO[4]

SUMMARY

Suita stadium is a soccer-only stadium with 40000 seats as the home stadium of “Gamba Osaka”, the team belonging to the J.League division1. The roof is consisted of a steel structure, the stand is consisted of a reinforced concrete structure. Stand frames are placed in the four laps of the pitch and the roof plan covering all stand structures. There is the roof on the stand structure of approximately 30 m in height. The roof size is 210 x 160 m, and the height of truss is 9 m.

The structural characteristics are “the three dimensional truss structure system (3D truss structure)” and “the seismic isolation system on the roof structure”. “The 3D truss structure” is a rationalizing structural system that has the diagonal direction (45degrees) trusses as well as the X-direction (zero degrees) trusses and the Y-direction (90 degrees) trusses.

Adoption of the seismic isolation system makes decreasing the truss stress fluctuation in temperature, the reduction of the earthquake load for the roof, and the stand, and improving the quality of the design. We adopt 8 high damping rubber bearings and 8 cross linear bearings on the seismic isolation materials. The rubber bearings support 80% of the weight, to absorb the earthquake energy for the whole weight for roof during the earthquake. Cross linear bearings, to resist the pull-out force in the wind load.

***Keywords**: space structure, structural design, 3D truss structure, seismic isolation, seismic response analysis*

INTRODUCTION

This paper describes the structural design of the Suita Stadium, a large stadium characterized by the three-directional mega truss design (hereinafter referred to as “3D truss structure”) of its wide-span roof structure and the seismic isolation bearings that support it. The Suita Stadium, is being constructed to serve as a dedicated football stadium with a seating capacity of 40,000, and as a home ground for Gamba Osaka, a professional football club. (Figure 1)

We have performed time history response analyses on this stadium to verify its safety against earthquakes. The resulting design is reported below. Note that this paper has been written based on Reference [1], which is available only in Japanese.

Figure 1. Completed stadium

[1] *Design department, Takenaka Corporation, Japan, e-mail: kihara.takashi@takenaka.co.jp*
[2] *Design department, Takenaka Corporation, Japan, e-mail: okude.hisato@takenaka.co.jp*
[3] *Design department, Takenaka Corporation, Japan, e-mail: nozawa.hirokazu@takenaka.co.jp*
[4] *Design department, Takenaka Corporation, Japan, e-mail: ohno.masahito@takenaka.co.jp*

SEISMIC DESIGN OF STEEL MOMENT RESISTING FRAMES WITH DAMPING SYSTEMS IN ACCORDANCE WITH KBC 2016

Seong-Ha Jeon [1], and Ji-Hun Park [2]

SUMMARY

Korean building code (KBC) introduced seismic design provisions for structures with damping systems in 2016. KBC 2016 adopted only time history analysis for the analysis and design of structures with damping systems. The provisions define minimum base shear for the design of the seismic force resisting system, and requires elastic response of structural elements in the damping system. The provisions specify a procedure to verify response reduction capacity of the damping system. In this study, a design method based on nonlinear time history analysis is proposed. Proposed design method employs a limited number of time history analyses with incremental damping ratios. As illustrative design examples, three- and six-story steel moment resisting frames with damping systems are designed using the proposed design method. Both nonlinear static and nonlinear dynamic analyses are performed to evaluate the overstrength and collapse margin of the example structures. In addition, conventional moment resisting frames with damping systems are compared in various aspects.

***Keywords**: SEEBUS; Damping system, Damping device, Steel moment frame, Nonlinear time history analysis*

INTRODUCTION

According to the regulation of Korean Building Code(KBC2009, seismic design criteria applied with damping system does not exist in Korea. The design of the structure using damping system has an advantage of both economic feasibility and seismic performance.
However, it is difficult to practically apply it since it does not have damping system design criteria .
For domestic design criteria, there has not been any design criteria of the structure that uses damping system by taking advantage of damping device. However, the newly revised domestic Korean Building Code(KBC2016) addresses the design criteria of the structure applied with damping system.
For this reason, this study is to propose the design process for the steel moment frame based on the seismic design criteria applied with the damping system of the revised KBC2016 code and also aimed at conducting analysis verification for the design process through nonlinear time history analysis

THE STRUCTURE WITH SEISMIC DESIGN TAKING ADVANTAGE OF KBC2016 DAMPING SYSTEM

Damping system

The purpose of damping system is to reduce the seismic load of the structure and it also refers to the structure that has all the structure elements that transfers load to the foundation of the structure and seismic force resisting system from damping device.

[1] Graduate Student, Department of Architecture, Incheon National University, Korea; wings616.ha@gmail.com
[2] Associate Professor, Division of Architecture and Urban Design, Incheon National University, Korea; jhpark606@incheon.ac.kr

OPTIMMAL DISTRIBUTION OF DAMPING COEFFICIENTS FOR VISCOUS DAMPERS IN BUILDINGS

Tzu Kang LIN[1], Jenn Shin HWANG[2], and Kuan Hui CHEN[3]

SUMMARY

In recent times, a guideline for the viscous damper design has been broadly adopted into seismic codes worldwide. Although the relationship between the energy dissipated by a damper and the designed damping coefficient has been theoretically derived, the process of distributing the damping coefficient onto each story is still not regulated among the codes. To properly determine the individual story damping coefficient and the configuration of the entire building, various distribution theories and methods have been proposed. As no systematic comparison between these methods has been made to achieve the optimal damping allocations for structural controllability and economical consideration, two search methods based on genetic algorithms (GAs) are used in this study to verify the optimal distribution of damping coefficients under seismic disturbances. The optimization result is compared with a variety of existing distribution methods under the same design damping ratio for three individual two-dimensional frameworks: a regular structure, a soft-layer structure, and a setback building. The analysis results indicated that similar reduction can be achieved by using different distribution methods, while the optimal story damping coefficient could be obtained by using the proposed optimization method. Moreover, for practicality, the story shear strain energy to efficient stories (SSSEES) method is further suggested.

Keywords*: Damping Coefficient; Optimal Distribution; Viscous Damper.*

INTRODUCTION

Among current earthquake engineering techniques, passive control is considered as the most reliable method to protect structures. To date, passive structural control can be roughly divided into two categories: seismic isolation and energy dissipation (Parulekar and Reddy 2009), and a substantial number of structures equipped with passive control devices have now been constructed with significant improvements in seismic resistance (Soneji and jangid 2006). By implementing passive control, important structures, such as hospitals, school buildings, fire stations, and high-tech industry parks, can survive under major earthquakes.

Nevertheless, with regard to design methods for structures with viscous damping system, substantial improvements and further developments are still required. In the equivalent damping ratio estimation formula proposed by the National Earthquake Hazards Reduction Program 1997 (e.g., FEMA273 et al. 1997), most of the specifications only focus on estimating the overall equivalent damping ratio. However, no further design formula regarding the distribution of damping coefficients of viscous dampers at each story of an entire building has been provided. Although related researches have been widely carried out, limitations or drawbacks still exist. In the sequential search algorithm (SSA) proposed by Shukla and Datta (1999), iterative analysis is required after each placement of one single damper, and the concentration of dampers on selected floors tends to cause problems during engineering implementation. Meanwhile, the distribution of a damper coefficient, which is directly

[1] *Associate Professor, Department of Civil Engineering, National Chiao Tung University, Taiwan, email: tklin@nctu.edu.tw*
[2] *Professor, Department of Construction Engineering, National Taiwan University of Science and Technology, Taiwan, email: JSH@mail.ntust.edu.tw*
[3] *Technician, Land Consolidation Engineering Bureau, Ministry of Interior, Taipei City 10055, Taiwan, email: a0937723177@hotmail.com*

PASSIVE BASE ISOLATION DESIGN OF SEISMICALLY-EXCITED BUILDINGS USING LINEAR QUADRATIC REGULATOR CONTROL ALGORITHM

Chia-Ming CHANG[1], Syuan SHIA [2], and Cho-Yen YANG[3]

SUMMARY

Seismic isolation is a powerful protection strategy for structures against earthquake hazards. Passive base isolation employs flexible devices underneath a structure which shifts the fundamental frequency of the structure away from the dominant frequency range of seismic excitation. Due to the fact that the flexible devices can introduce excessive displacements at base of structures during severe earthquakes, additional damping devices are recommended to install along with the isolation layer in the seismic design code. Therefore, the objective of this study is to develop a new seismic isolation design procedure for lead-rubber bearings of which both stiffness and damping are concurrently determined using the linear quadratic algorithm (LQR). In this procedure, the mass, damping, and stiffness of a superstructure is known, and a mass ratio between the superstructure and isolation layer is predetermined. The stiffness and damping coefficient of the base isolation can be obtained by the LQR, while these two terms vary with the weighting selected in LQR. To determine the most appropriate stiffness and damping, a performance curve is generated by simulating a simplified two degrees of freedom isolated building subjected to a spectrum-compatible ground motion. Subsequently, the stiffness and a part of the damping coefficient are achieved by isolators, while the remaining damping coefficient is realized by additional viscous dampers. Moreover, the detailed design of lead-rubber bearings is parameterized by a bi-linear model, consisting of the designed stiffness, damping coefficient, pre-to-post yielding stiffness ratio, and a target displacement. In this study, a numerical example is carried out to demonstrate the proposed design procedure for the benchmark building in the smart base isolation problem, while this procedure is also employed to design a real-world base-isolated building. As shown in the simulation results, the seismic isolation design procedure is effective to design a passive isolation system for buildings against earthquakes.

***Keywords**: Passive Seismic Isolation; Lead-rubber Bearing Design; Linear Quadratic Regulator Control Algorithm; Bilinear Hysteretic Model.*

INTRODUCTION

Seismic isolation is an effective means of protecting structures against earthquakes and has been widely accepted as a control strategy for structures in seismically active areas. Base isolation can limit the transmission of ground motion to structures, and the main effect is to decouple the structures from ground motion by increasing the resonant period of the structure using flexible isolation elements. Although base shears, floor accelerations, and interstory drifts are significantly reduced, the passive base isolation intrinsically induces larger base displacements (Buckle et al. 2002). One solution is to install additional damping devices that reduce the excessive base displacement. For the application of seismic isolation, the flexible elements are recommended to combine with supplemental damping devices.

[1] *Assistant Professor, Department of Civil Engineering, National Taiwan University, Taipei 10617, Taiwan. Email: changcm@ntu.edu.tw*
[2] *Graduate Student, Department of Civil Engineering, National Taiwan University, Taipei 10617, Taiwan. Email: r04521232@ntu.edu.tw*
[3] *Assistant Researcher, National Center for Research on Earthquake Engineering, Taipei 10668, Taiwan. Email: cyyang@ncree.narl.org.tw*

SHAKING TABLE TESTS OF ROCKING BASE-ISOLATED STRUCTURE WITH FRICTIONAL HINGE DAMPER

Chin-Tung CHENG[1] and Jyun-Syong LIN[2]

SUMMARY

Literatures have shown that traditional sliding base-isolated systems may have long-period resonant-like response induced by a ground motion containing strong long-period frequency spectra. In this paper, a new type of isolator called variable-frequency rocking bearing is proposed. The system allows the bearing remain still under a moderate seismic load but start to rock back-and-forth during severe earthquakes. And the vibration is damped throughout each impact of the bearing on structural footing or foundation surfaces and additional frictional hinge dampers. Based on a modified response spectra, damping ratio of base-isolated structures can be estimated by measured acceleration and displacement at roof floor. The objective of the paper is to investigate seismic performance of rocking base-isolated structure with frictional hinge damper. To validate proposed idea, a one-bay-one-story steel braced structure base-isolated with rocking bearings was constructed. In total, 119 shaking table tests were conducted with various investigated parameters including frictional material (brass or phosphor bronze) in the damper, aspect ratio of the bearing, shapes of rocking plate (plane, spherical or polynomial), with or without the spring force and waveforms of ground motion. Since phosphor bronze plate has higher friction coefficient than brass plate, normal force applied on the damper of phosphor bronze plate is 19kN compared with 27kN for brass plate to achieve similar designed damping ratio of 15%. Test results indicate that vibration behavior was nonlinear with respect to rocking amplitude. With additional damper, base-isolated structures vibrated in less structural response but higher damping than those without damper under the excitation of earthquakes. The averaged damping of 14.5% for structures with dampers was significantly increased 10% from 4.5% for structures without dampers.

***Keywords**: Rocking Bearing; Base-isolation; Frictional Hinge Damper, Damping Ratio; Shaking Table Tests.*

INTRODUCTION

Literatures have shown that seismic isolation is an effective technology to protect structures from earthquake attacks (Naeim and Kelly 1999). Seismic isolation is achieved through the installation of a soft layer below the protected structures, so that seismic energy transmitted up to the super-structure due to a ground motion is isolated. The flexibility required in isolation system to reduce seismic response may be alternatively obtained by allowing part of structure to uplift, rocking or stepping during large horizontal motions (Dowrick 1987). For example, bridges with insufficient tensile pile capacity at footings allow the uplift of structures during strong earthquakes. This rocking behavior dominates the pier response and minimizes the damage to the footing and columns. More recently, control of rocking behavior of steel braced frames has been investigated by researchers (Eatherton et al. 2014, Wiebe et al. 2013). The system consists of three main components: a steel braced frame that remains essentially elastic by uplifting at the column bases during ground motions, vertical post-tensioning (PT) that provides resistance to overturning and provides self-centering forces, and replaceable

[1] *Professor, National Kaohsiung First University of Science and Technology, Taiwan ,e-mail: ctcheng@nkfust.edu.tw*
[2] *Graduate student, National Kaohsiung First University of Science and Technology, Taiwan*

ANALYSES OF RC SHEAR WALLS WITH VERTICALLY ALIGNED DOOR OPENINGS

Hong LIU[1], Yasushi SANADA[2] and Suguru SUZUKI[3]

SUMMARY

This study investigates fundamental behavior of reinforced concrete (RC) shear walls with vertically aligned door openings through laboratory tests and numerical analyses. A series of tests was conducted using scaled models representing typical multi-story RC shear walls in Japan, verifying the influence of the existence/quantity of door openings. In this paper, the test results were simulated by macroscopic models in predicting the nonlinear responses of RC shear walls with vertically aligned door openings. Flexural behavior of walls was represented by fiber models, while shear behavior of beams above openings was replaced by nonlinear shear springs. By the comparison between experimental results and analytical simulation for the specimens with the single-aligned and double-aligned openings, the analytical results showed good agreements with the experimental results, verifying the effectiveness of the proposed analytical models.

Keywords: *Door opening, Fiber Model, Reinforced concrete, Seismic performance, Shear wall*

INTRODUCTION

Reinforced concrete (RC) shear walls have been widely used in building constructions for providing high lateral stiffness and strength to resist lateral loads due to wind or earthquake. However, it is inevitable that openings are planned in shear walls for functional requirements, such as ventilating and lightening. These openings may significantly affect the structural performance of shear walls, such as changing their stress transferring mechanism, lowering the strength and stiffness. Therefore, modeling RC shear walls with openings accurately to predict the seismic behavior is both necessary and important, especially those of which with vertically aligned door openings making shear walls behave like coupled walls.

In order to investigate the fundamental behavior of RC shear walls with vertically aligned door openings, three 1/6 scale shear wall specimens were designed and tested with different door opening configurations (Sanada et al., 2015). The experimental results showed that the maximum strengths were influenced by the existence and the number of door openings, while the shear walls with vertically aligned door openings behaved in a different manner to that without openings. It was also indicated that the flexural behavior of walls with singe boundary column or without boundary column due to the existence of door openings, and the shear behavior of beams played important roles in the seismic performance of the specimens with openings, which seemed to be key issues for numerical simulation of the tests.

Therefore, the purpose of this paper is to propose analytical models to simulate the experimentally observed behavior/performance of the specimens with aligned door openings, where the tested specimens were replaced by beam elements. The verification of the proposed models was also performed through comparisons between the experimental results and the numerical simulations.

[1] *Graduate Student, Graduate School of Engineering, Osaka University, Japan, e-mail: liu_hong@arch.eng.osaka-u.ac.jp*
[2] *Associate Professor, Graduate School of Engineering, Osaka University, Japan, e-mail: sanada@arch.eng.osaka-u.ac.jp*
[3] *Assistant Professor, Graduate School of Engineering, Osaka University, Japan, e-mail: suzuki@arch.eng.osaka-u.ac.jp*

ON THE ASSESSMENT OF RESIDUAL DEFORMATION IN FLEXURAL-DOMINANT RC WALLS UNDER REVERSED CYCLIC LOADING

Luis A. BEDRIÑANA[1], Masanori TANI[2], and Minehiro NISHIYAMA[3]

SUMMARY

Conventionally designed reinforced concrete (RC) structures are likely to retain considerable residual deformation after large earthquake events, leading to undesirable high post-earthquake retrofitting cost. Therefore, modeling and predicting residual deformation in RC structures constitutes an important challenge. The aim of this study is to evaluate the residual deformation in flexural-dominant bearing walls subjected to reverse cyclic loading. Two types of walls were examined: one (BC) with boundary columns and the other (NC) with rectangular section area. Both walls had been previously tested under reversed cyclic horizontal loads (Tani et al. 2013). To assess the residual deformation of the walls, nonlinear finite element analyses were carried out. The analytical models were able to capture the cyclic behavior and the residual deformation of the studied walls. A parametric analysis was carried out for some parameters such as axial load, boundary confinement reinforcement and boundary reinforcement ratio. Furthermore, the effects of boundary columns were also evaluated. According to the analytical results, the axial load level on the walls had a substantial influence on the magnitude of residual deformations. The boundary reinforcement content also had an effect on the progression of residual deformations. By contrast, spacing of confinement reinforcement had little influence on formation of residual drifts. Based on the analytical results a simple relation was proposed to assess the residual drift of walls under similar conditions.

Keywords*: structural walls; boundary columns; residual deformation; finite element analysis; axial load ratio.*

INTRODUCTION

In general, conventional approach in seismic design of reinforced concrete (RC) structures relies on providing enough inelastic deformation capacity to principal elements to ensure protection against collapse during strong ground shakings. However, experience of large earthquakes in the last decades has evidenced that structures under large inelastic deformation are also prone to retain large residual deformation after the event, which may lead to undesirable post-earthquake performance (Kawashima et al. 1998). Therefore, it has been recognized that residual deformations are as important as peak deformation in assessing seismic performance of RC structures (Kawashima et al. 1998; Liossatou and Fardis 2015).

Although evaluation and prediction of peak displacements in RC structures is well documented, little work has been done specifically in the assessment and prediction of residual displacement. Pioneering studies focused into the evaluation of residual displacement in systems by using simplified hysteretic rules (Kawashima et al. 1998; Macrae and Kawashima 1997; Mahin and Bertero 1981; Ruiz-García and Miranda 2006). From these initial studies, it has been pointed out that hysteretic model and its parameters play a key role in assessing residual displacement. However, the use of hysteretic models to predict residual displacement was found to be quite inadequate for practical purposes, since these models need to be well defined prior the design to achieve some accuracy (Dazio 2004; Lee and Billington 2010).

[1] *PhD Candidate, Department of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail: rc.luis@archi.kyoto-u.ac.jp*
[2] *Associate Professor, Department of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail: tani@archi.kyoto-u.ac.jp*
[3] *Professor, Department of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail: mn@archi.kyoto-u.ac.jp*

SLIDING SHEAR BEHAVIOR OF STRUCTURAL WALLS OF 1/3-SCALE 6-STORY REINFORCED CONCRETE BUILDING IN SHAKING TABLE TESTS

Yuki IDOSAKO[1], Haruki INOUE[2], Masanobu SAKASHITA[3], Masanori TANI[4], and Minehiro NISHIYAMA[5]

SUMMARY

The shaking table tests on a 1/3-scale 6-story reinforced concrete condominium building were carried out in E-Defense. This paper mainly describes the observed torsional behavior of the building as well as the sliding of structural reinforced concrete walls. The torsional behavior of the building was well captured by adopting sliding shear springs with an idealized shear force – slip displacement model at the wall bases in the first floor. The shear force - slip model was based on the result of static loading tests on shear walls carried out by the authors. The springs had sliding shear capacity based on friction due to compression force and dowel resistance of vertical reinforcement crossing the bottom section of the walls. The hysteresis curve of the shear force - slip model was idealized as a rigid perfectly plastic relationship. Although sliding shear behavior was also observed in the second story walls in the tests, the springs in the analyses were installed only at the bottom of the first story perimeter walls. In this paper, the results for the JMA Kobe 10 to 100% earthquake waves were discussed.

Keywords: *reinforced concrete building; shaking table tests; structural wall; sliding shear*

INTRODUCTION

As another more national effort to mitigate the damage during bigger earthquakes and to make the urban areas more resilient, Ministry of Education, Culture, Sports, Science and Technology or MEXT started a special project on Reducing Urban Mega Earthquake Disasters in 2012. As a part of the project, a research group on Maintenance and Recovery of Functionality in Urban Infrastructures was organized by Disaster Prevention Research Institute or DPRI of Kyoto University. It consists of several shaking table tests in the E-Defense of National Research Institute for Earth Science and Disaster Resilience or NIED. There are two main purposes; one is to quantify the collapse margin of high-rise buildings and the other is to monitor and assess the condition of buildings during and soon after earthquakes. The first test was conducted on a 20-story steel frame which collapsed after a series of earthquake wave inputs. The second was a test on a 1/3-scale 6-story reinforced concrete condominium, which this paper describes.

The experiment and its results are reported and discussed in [Katsumata, 2015]. This paper focuses on torsional behavior of the building and shear sliding observed at the bottom of the structural walls. In order to capture the wall behavior in the large displacement region, the shear sliding is idealized and included in dynamic response analyses.

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[1] *Graduate Student, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail: rc.idosako@archi.kyoto-u.ac.jp*
[2] *Graduate Student, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail: rc.inoue@archi.kyoto-u.ac.jp*
[3] *Senior Research Engineer, Department of Structural Engineering, Building Research Institute, Japan, e-mail: m-saka@kenken.go.jp*
[4] *Associate Professor, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail: tani@archi.kyoto-u.ac.jp*
[5] *Professor, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan, e-mail: mn@archi.kyoto-u.ac.jp*

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IN-PLANE LOADING TESTS FOR CONFINED AND IN-FILLED MASONRY PANELS IN RC FRAMES WITH ECCENTRIC DOOR OPENINGS

Yi-Hsuan TU[1], Yu-Syuan LIN[2], Min-Shin TSAI[3] and Tsung-Chih CHIOU[4]

SUMMARY

In-plane static loading tests were performed to study the behavior of masonry panels with eccentric openings. This type of masonry panels are commonly placed at the weak axis of typical low-rise RC street-buildings in Taiwan. Four full-sized specimens were designed with two test factors: the construction type and the presence of openings. Confined and in-filled masonry panels were surrounded by identical RC frames with a non-ductile design to simulate old buildings. Constant vertical force was applied to the specimens during the test. Cyclic lateral load with controlled displacement was applied in a shear-building manner. The test results showed that the cracking pattern of the masonry panels was affected by the confining condition. Diagonal cracks appeared in the specimens with no openings. In the specimens with openings, cracks occurred along the column-panel interface and the panels slid due to the lack of confinement around the openings. The difference in construction type affected the failure mode. The maximum strengths of confined and in-filled masonry specimens were governed by diagonal tension and bed-joint sliding failure, respectively. The damage behaviors of the columns were affected by both the construction type and the presence of openings. Shear failure happened at the columns in the two specimens without openings and the column adjacent to the confined masonry panel with opening. The column adjacent to the in-filled masonry panel with opening showed flexural behavior similar to the independent columns. The stiffness and strength of the specimens increased and the deformation capacity decreased when the masonry panels had better confinement. The test results were compared with analytical models. Applicable analytical modes were discussed.

Keywords: *confined masonry; in-filled masonry; opening; in-plane.*

INTRODUCTION

In Taiwan, most of the existing low-rise buildings use RC frames as the skeleton and masonry panels as the partition walls. Before the major revision for seismic regulations of the building codes in 1997, most of the low-rise RC buildings were not constructed with ductile design. The consideration for masonry panels depended on their construction types. Confined masonry (CM) panels that are constructed prior to the boundary frame and connected with the frame with shear keys are regarded as structural elements. The panels in a CM building should provide 75% of the lateral resistance for the building and the frame provide the rest 25% in accordance with the building codes. However, CM buildings are not allowed to exceed three stories. When numerous residences were built due to the rapid economic growth in the 1980s, in-filled unreinforced masonry (URM) panels started to take the place of CM for reducing construction time and the demand of higher buildings. The in-filled panels are usually considered non-structural and not included in the structural design. CM buildings are still built currently in the rural regions in Taiwan, but they are quite different from the standard CM buildings in

[1] *Associate Professor, National Cheng Kung University, Department of Architecture, Taiwan, email: yhtu@mail.ncku.edu.tw*
[2] *Master of Science, National Cheng Kung University, Department of Architecture, Taiwan, email: michelle8040@hotmail.com*
[3] *Master of Science, National Cheng Kung University, Department of Architecture, Taiwan, email: starsbook@hotmail.com*
[4] *Associate Researcher, National Center for Research on Earthquake Engineering, Taiwan, email: tcchiou@ncree.narl.org.tw*

CYCLIC BEHAVIOR OF HIGH-STRENGTH STEEL REINFORCED SQUAT UHPFRC SHEAR WALLS

Chung-Chan HUNG[1] and Hong-Chi CHEN[2]

SUMMARY

Ultra-high performance fiber reinforced concrete (UHPFRC) is characterized by ultra-high compressive strength and ductile tensile strain hardening behavior accompanied by dense fine cracks. This study experimentally investigated the seismic behavior of squat UHPFRC shear walls reinforced with high-strength steel rebar. Various Squat shear walls were tested under displacement reversals, with the experimental variables including the strength of steel reinforcement, shear stress demand for the wall, steel fiber, and dowel bar. The results revealed that the presence of steel fibers enhanced the strength, confinement, and crack-width control ability of squat UHPFRC shear walls, allowing the walls to exhibit ductile flexural-dominant behavior even when the shear stress demand for the wall was 20% greater than the code-specified maximum allowable value. Furthermore, the proposed novel squat shear wall not only took advantage of the ultra-high mechanical properties of high-strength steel and UHPFRC materials, but also resolved the concern of the potential premature failure modes for high-strength reinforcement and concrete.

***Keywords**: UHPFRC; squat shear walls; high-strength steel reinforcement; seismic behavior.*

INTRODUCTION

High performance fiber reinforced concrete (HPFRC) is distinguished from conventional fiber reinforced cementitious composites (FRCCs) by its tensile strain hardening behavior accompanied by dense hairline cracks (Naaman and Reinhardt 2006). In addition to the advantageous tensile response, its compressive strength and ductility are also enhanced compared to traditional concrete materials, due to the confining effect provided by the discontinuous short fibers. These enhanced properties of HPFRC have motivated researchers to explore the benefits of using HPFRC in newly built structures and structural repairing and rehabilitation (Fischer and Li 2002; Kesner and Billington 2005; Parra-Montesinos et al. 2005; Hung and El-Tawil 2011; Lequesne et al. 2013; Hung and Chen 2016; Hung et al. 2016). In addition to the experimental evaluation of HPFRC materials and structures, various computational models have been developed to predict the behavior of HPFRC under external forces (Hung and Li 2013; Hung et al. 2013). Among the HPFRC materials, ultra-high performance fiber reinforced concrete (UHPFRC) is a unique class that has ultra-high compressive strength. This stems from a tailored mixing design, which has a high cementitious material content and an optimized gradation of granular materials with no or minimal coarse aggregate. Unlike the conventional high-strength concrete that has an extremely brittle failure pattern, UHPFRC possesses a substantially improved compressive ductility and residual strength.

The present study investigated the seismic behavior of a novel squat UHPFRC shear wall reinforced with high-strength steel rebar. The replacement of the normal steel reinforcement by the high-strength one in squat shear walls creates an opportunity for designers to use less steel reinforcement, thus simplifying the placement of UHPFRC. In addition, due to the advantageous high strength and high ductility characteristics of UHPFRC

[1] *Associate Professor, National Cheng Kung University,Taiwan, e-mail: cchung@mail.ncku.edu.tw*
[2] *Graduate Student, National Central University, Taiwan*

Evaluation of Shear Performance of Reinforced Concrete Beams in Relation to the Presence of Web Openings

Min-Jun Kim[1], Hyeong-Gook Kim[2], Yong-Jun Lee[3], Sung-Woo Jeong[4], Sang-Woo Kim[5] and Kil-Hee Kim[6]

SUMMARY

This study conducted experiment of reinforced concrete beams to evaluate the shear resistance performance of RC beams in relation to the presence of web openings. The main variables of the experiment were web openings, web reinforcement, and the spacing between web openings. The experimental sections of all specimens were subject to a seismic load with antisymmetric bending moment. The shear performance of the specimens was examined based on the shear force-drift angle relations and the crack patterns on the specimens. In addition, the shear strength of the specimens obtained from the experimental results was compared to the results derived from the existing formula. The experimental results showed that the shear capacity and ductility of the specimens reinforced by the proposed web reinforcement were similar to those of the specimens without web openings. The specimens reinforced with the proposed web reinforcement had improved shear capacity and ductility compared to those without reinforcement.

***Keywords**: Reinforced concrete beams; Web openings; Web reinforcement; shear performance;*

INTRODUCTION

With the construction of more high-rise and large scale buildings, beams with reinforcing methods of web openings have attracted great interest as they are able to reduce story height and the number of facilities involved. The web openings in general are installed in the web of a beam to allow facilities, such as air conditioning ducts, electrical cables and other pipes, to be passed through. The installation of web openings is not only economical as it reduces story height by minimizing the facility space within ceilings of structures, but also secures a larger indoor space with a much more comfortable environment.

The web opening system can be largely divided into steel beams with web openings and reinforced concrete (hereafter called RC) beams with web openings. Steel beams with openings were first studied in the United States by Heller[1)] in 1951. This was followed by various analytical and experimental studies (Bower[2)], Cooper and Snell[3)], Redwood[4)]). On the other hand, less research has been performed on RC beams with web openings because of their complex failure modes and problem with reinforcing web openings. In case of RC beams with web openings, it is necessary to clarify their shear resistance mechanism given the greater influence on shear performance than on flexural performance.

Research on RC web openings was first performed in Sweden by Lorenstren[5)] in 1962. In 1967, Nasser et al.[6)] studied the behavior of RC beams with rectangular web openings and related design methods. Mansur[7)] conducted a study on rectangular web openings in 1983, and Mansur et al.[8)] proposed a method of designing RC beams with large openings in 1985. The Architectural Institute of Japan established a committee to enact criteria for RC beams with web openings in the 1980s, and a strength criterion for RC beams with web openings was finally adopted based on large amounts of analytical and experimental research. In Korea, research on RC beams with web openings also began in the early 1980s.

The disadvantages of existing web reinforcement include poor constructability due to its inability to maintain a position, high construction costs, and long construction period. To resolve these issues, this study developed web reinforcement with good constructability and outstanding structural performance, and evaluated its shear resistance performance.

[1,2,3]Research Assistant Professor, Kongju National University, Republic of Korea
[4]Ph.D. Candidate, Kongju National University, Republic of Korea
[5]Head of R&D Center, Hub Structure Engieenering Co., Ltd. Republic of Korea
[6]Professor, Kongju National University, Republic of Korea, e-mail: kimkh@kongju.ac.kr