STATIC LOADING TEST ON A FULL SCALE FIVE STORY REINFORCED CONCRETE BUILDING UTILIZING WING WALLS FOR DAMAGE REDUCTION
Damage evaluation of the 2nd floor beam with secondary walls and RC slab

Hidekazu WATANABE[1], Fuhito KITAMURA[2], Eko Yuniarsyah[3], Tomohisa MUKAI[4], Masanori TANI[5], Susumu KONO[6], Masaki MAEDA[7] and Hideyuki KINUGASA[8],

SUMMARY

In Japan, reinforced concrete walls with large opening cannot be treated as structural wall in structural design. Those walls are called "non-structural wall" and its capacity is not generally considered in the structural calculation. In the 2011 Tohoku Earthquake, non-structural reinforced concrete walls in many government buildings and residential buildings were heavily damaged. Some buildings whose non-structural reinforced concrete walls were heavily damaged were given up their rehabilitation and demolished due to the problems of huge repair cost etc., even if the structural damage of the building was not so significant. The damage of such member does not affect to the safety of building. However, it is important to evaluate damage of building with non-structural reinforced concrete walls for considering about the continuity and rehabilitation of building function. Behind this situation, static loading test on a full scale Five-story RC building was conducted by the research project of National Institute for Land and Infrastructure Management "Development of function sustaining technologies for buildings used as Disaster Prevention Bases" (Fukuyama et al. 2015). The purpose of this experiment is development of new reinforced concrete system using wing walls for damage reduction of non-structural reinforced concrete walls. The purpose of this study is to evaluate damage level of beam with secondary walls and RC slab using results of full scale five story reinforced concrete building. In the result, the damage level of the 2nd floor beam with secondary walls and RC slab was classified by experimental results (residual crack width and length, concrete spalling area) using existing method.

Keywords*: non-structural RC wall; full scale test; damage evaluation.*

INTRODUCTION

In Japan, reinforced concrete walls with large opening cannot be treated as structural wall in structural design. Those walls are called "non-structural wall" and its capacity is not generally considered in the structural calculation. In the 2011 Tohoku Earthquake, non-structural reinforced concrete walls in many government buildings and residential buildings were heavily damaged. Some buildings whose non-structural reinforced concrete walls were heavily damaged were given up their rehabilitation and demolished due to the problems of huge repair cost etc., even if the structural damage of the building was not so significant. The damage of such member does not affect to the safety of building. However, it is important to evaluate damage of building with non-structural reinforced concrete walls for considering about the continuity and rehabilitation of building

[1] *Assistant Professor, Structural Engineering Research Center, Tokyo Institute of Technology, Japan, e-mail: watanabe@serc.titech.ac.jp*
[2] *Graduate student, Structural Engineering Research Center, Tokyo Institute of Technology, Japan, e-mail: kitamura.f.ab@m.titech.ac.jp*
[3] *Graduate student, Structural Engineering Research Center, Tokyo Institute of Technology, Japan, e-mail: yuniarsyah.e.aa@m.titech.ac.jp*
[4] *Senior Research Engineer, Building Research Institute, Japan, e-mail: t_mukai@kenken.go.jp*
[5] *Associate Professor, Kyoto University, Japan, e-mail: tani@archi.kyoto-u.ac.jp*
[6] *Professor, Structural Engineering Research Center, Tokyo Institute of Technology, Japan, e-mail: kono@serc.titech.ac.jp*
[7] *Professor, Graduate School of Engineering, Tohoku University, Japan*
[8] *Professor, Department of Architecture, Faculty of Science and Engineering, Tokyo University of Science, Japan*

LATERAL LOAD EXPERIMENT FOR CONFINED AND IN-FILLED UNREINFORCED MASONRY PANELS WITH OPENINGS IN RC FRAMES

Yi-Hsuan TU[1], Yuan-Hsin HSU[2], and Yen-Fen CHAO[2]

This paper is also submitted to the 12th North American Masonry Conference

SUMMARY

Three specimens of masonry panels with openings surrounded by reinforced concrete (RC) frames were tested under lateral cyclic load and vertical force. Two of the specimens had confined masonry (CM) panels with two types of openings: a door and a window. The last specimen has an in-filled panel with a door opening. The purpose is to study the difference between various panel construction types and the effect of openings. All specimens have identical RC frames. After the test, the panels of the in-filled type specimen were removed and the bare frame was tested again to investigate the contribution of panels.
The test results show that construction type has obvious effect on structural behavior, including differences in failure mode, maximum strength, and deformation capability. The opening type affects the maximum strength of the panels and therefore affects the performance of the frame to some degree.

***Keywords**: Confined masonry, In-filled masonry, Masonry panel, Opening type.*

INTRODUCTION

Openings are the weakness of structural masonry panels. Panels with openings are not only vulnerable during earthquakes; the variation of openings also adds complications in seismic analysis. When masonry panels are surrounded by RC frames, structural engineers tend to ignore panels with openings. However, this might cause misestimates of the stiffness, strength, and failure mode of the frame and the global structure. Standard confined masonry (CM) buildings are required to have tie members around the openings (Meli 2011) to ensure the panel segments separated by the openings are fully confined and can be analyzed as intact panels. For in-filled unreinforced masonry (URM) panels, there is no similar requirement.

In Taiwan, most of the existing low-rise buildings use RC frames as the skeleton and masonry panels as the partition walls. There were different ways of construction for these buildings in different periods in the past. The oldest type was originally a pure masonry building, RC tie beams and columns were added to confine the existing masonry panels and support newly-built RC slabs when it was renovated. Then people started to build new confined masonry buildings with similar design. The sections of earlier tie beams and columns were small, such as 240mm x 240-400 mm, and became larger as RC became more affordable. However, CM buildings are not allowed to exceed three stories according to local building regulations. Therefore, when numerous residences were built due to the rapid economic growth in the 1980s, in-filled masonry panels started to take the place of CM for reducing construction time and the demand of higher buildings. CM buildings are still built currently in the rural regions in Taiwan, but they are quite different from the standard CM buildings in South Europe, Latin America, and other parts of Asia (EERI 2012). Taiwanese CM buildings have larger beam and column sections, typically 300-350mm x 400-450mm. The beam-column joints are moment-connected to adhere to the building regulation mandating the RC skeleton should carry at least one fourth of the earthquake load. In some buildings,

[1] *Associate Professor, Department of Architecture, National Cheng Kung University, Tainan, Taiwan; yhtu@mail.ncku.edu.tw*
[2] *Master of Science, Department of Architecture, National Cheng Kung University, Tainan, Taiwan*

RETROFITTING OF UNREINFORCED RC EXTERIOR BEAM-COLUMN JOINT BY INSTALLING WING WALLS

Yuebing LI[1] and Yasushi SANADA[2]

SUMMARY

A large number of reinforced concrete (RC) buildings which contain no stirrups in beam-column joints exist even in regions of moderate seismicity. For retrofitting this kind of seismically substandard beam-column joints, a method of installing RC wing walls beside existing columns is proposed, aiming to improve the moment capacity of joints by reaction on the beam from the wing walls. Three 3/4-scale substandard exterior beam-column joint specimens with common structural details were constructed and tested, two of which were retrofitted by wing walls with different retrofit details. The test results verified that the proposed method was applicable for retrofitting seismically substandard beam-column joints, and that the presented design process was reasonable.

Keywords: *Beam-column connection, Reinforced concrete, Seismic strengthening, Substandard joint, Wing wall*

INTRODUCTION

There is a substantial stock of reinforced concrete (RC) buildings containing no stirrups in the beam-column joint regions (named "unreinforced joint"), which were designed according to older design codes or without complying with current seismic codes owing to rough construction management, particularly those designed before the 1970s in the western U.S. and in other seismically active regions worldwide (Park et al. 2009), (Sanada et al. 2009). Laboratory tests have confirmed the seismic vulnerability of this kind of poorly detailed joints (Sashima et al. 2011), and failure of joints can cause a frame-building to collapse, as observed in recent severe earthquake disasters. However, the limited upgrading schemes reported in past studies (Gencoglu et al. 2007), (Misir et al. 2013) are not necessarily practical in developing countries because of the need for advanced materials and the complexity of the construction process. In consideration of the economic situation and technical level of developing countries, installing RC wing walls beside existing columns seems to be a practical way of upgrading these seismically substandard buildings. In this study, a retrofitting mechanism aiming to improve the moment capacity of joint by installing wing walls is proposed, by which success of retrofitting can be validated. Otherwise, loading tests were conducted with three specimens, two of which were retrofitted by installing wing walls to examine the validity of the retrofitting method and examine the retrofitting effectiveness of pulled or pushed wing wall.

INVESTIGATED BUILDING AND JOINT

This paper focuses on the exterior beam-column joint of an RC frame building collapsed in the 2009 Sumatra earthquakes, as shown in Fig. 1. It was built in 2005 with the story height of 3,000 mm and the beam span of 7,000 mm. In this building, damage such as buckling of column longitudinal rebar and concrete spalling was

[1] *Assistant Professor, Department of Global Architecture, Graduate School of Engineering, Osaka University, Japan, li@arch.eng.osaka-u.ac.jp*
[2] *Associate Professor, Department of Global Architecture, Graduate School of Engineering, Osaka University, Japan, sanada@arch.eng.osaka-u.ac.jp*

Efficiency of Strengthening T-shaped RC beams with Ultra High Performance Concrete

Jung-Taek Choi[1], **Sung-Gul Hong**[2]

SUMMARY

This study investigates the structural performance of reinforced concrete T beams retrofitted by ultra-high performance concrete (UHPC) to compare the effectiveness of strengthened T-beams with different regions and thickness of layers. Due to its high compressive and tensile strength, thin UHPC over layers are applied to increase tensile strength in flexural tension region with reinforcement and the compressive resistance in flexural compression region. Therefore, strengthened beams with UHPC enhance both positive and negative moment capacity. Fourteen ordinary reinforced concrete T-beams were prepared to intend shear failure modes. Jacketing methods applied to this experimental program has three different schemes: UHPC U shaped-jacketing, UHPC casting on the flange, and UHPC casting on the flange and Aramid FRP U shaped-wrapping. The results clearly show that the effectiveness of UHPC U shaped-jacketing method is better than that of other methods.

Keywords: *Strengthening, T-beam, composite beams, UHPC*

INTRODUCTION

Owing to its high structural performance of UHPC (Ultra High Performance Concrete), it is possible to apply thin layer of UHPC over ordinary existing structural concrete to increase their insufficient capacity. Many of researchers from literature have demonstrated the applicability of UHPC as one of efficient strengthening methods for retrofit; (Martinola, Meda, Plizzari, & Rinaldi, 2010) 5mm U-shaped jacket for flexural strengthening, (Meda, Mostosi, & Riva, 2014) shear strengthening for beams with 3 or 5mm U-jacketing with wire mesh. (Noshiravani, 2012) the overlay of UHPC with small diameter reinforcing bars and the thickness with 10-20% of depth of RC based on (Habel, Denarié, & Brühwiler, 2006)'s numerical analysis. Most of these strengthening methods have focused on tensile strengthening. Using both the high compressive and high tensile strength of UHPC, the overlay method can be applied not only to tension regions, but also to compression regions. Similarly, UHPC can be applied to the increase of flexural strength of the positive and negative moment (Fig. 1). This paper presents the effectiveness of singly reinforced beams by retrofitting positive/negative moment zone. In addition, AFRP (aramid fiber reinforced plate) strengthening methods are compared with the retrofit by UHPC material and the combination of two methods.

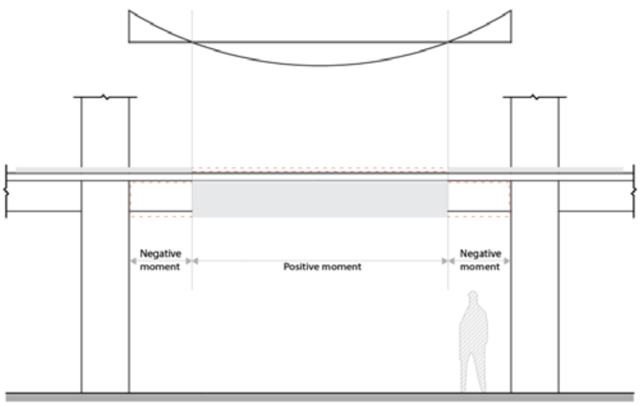

Fig. 1 Prototype of existing concrete building frame

[1] *Dept. of Architecture and Architectural Engineering, Seoul National University, Korea, gdchoice@snu.ac.kr*
[2] *Dept. of Architecture and Architectural Engineering, Seoul National University, Korea, sglhong@snu.ac.kr*

AN EXPERIMENTAL STUDY ON THE LOCAL DAMAGE OF REINFORCED CONCRETE WALLS CAUSED BY COLLISION OF TSUNAMI DEBRIS

(This paper was submitted to 11th International Conference on Shock & Impact Loads on Structures, 14-15 May 2015, Ottawa, CANADA)

Hidekazu WATANABE[1], Satoshi NAKAMURA[2], Susumu KONO[3], Kazunori FUJIKAKE[4], Amornthep SOMRAJ[5], and Hitoshi TANAKA[6]

SUMMARY

The 2011 off the Pacific coast of Tohoku Earthquake triggered devastating tsunami which caused various structural damages to reinforced concrete (hereafter, RC) buildings. Among various damage types, local damage of RC members due to debris impact is taken as a topic of this study. The current Japanese codes and standards of RC buildings do not have any statements on design against tsunami debris impact loads. Furthermore, there are not many experimental studies on local damage of RC buildings by tsunami debris impact loads. In order to design tsunami-resistant RC buildings, development of design methods for debris impact loads is necessary. In this study, impact tests were conducted using a lateral impact loading system to evaluate the performance of RC buildings. Sixteen RC wall specimens were made to investigate the effects of impact load on local damage. The specimens were square plates with 1300mm by 1300mm, and had three variables; wall thickness (80mm, 120mm), specified concrete compressive strength (24MPa, 60 MPa), and impact velocity (ranging from 2.6 to 9.8 m/s).

In this experiment, damage of specimens was classified as penetration, scabbing, and perforation. The limit impact velocity for scabbing and perforation was evaluated by two existing impact formulae (Chang's formula and CRIEPI formula). However, two formulae overestimated the limit impact velocities for scabbing and perforation. Thus, new formulae are proposed to fit the experimental results in low impact velocity (ranging from 2.6 to 9.8 m/s). The proposed formulae can evaluate conservatively the all experimental values of limit impact velocity for scabbing and perforation.

***Keywords**: penetration; scabbing; perforation; impact test.*

INTRODUCTION

The 2011 off the Pacific coast of Tohoku Earthquake triggered devastating tsunami which caused various structural damages to reinforced concrete (hereafter, RC) buildings. According to damage investigation reports of National Institute for Land and Infrastructure Management (NILIM) and Building Research Institute (BRI) , local damage of the RC buildings from tsunami debris is confirmed. The Japanese current codes and standards of RC buildings do not have any statements on design against tsunami debris impact loads. Furthermore, there are not many experimental studies on local damage of RC buildings by tsunami debris impact loads. In order to design tsunami-resistant RC buildings, development of design methods for debris impact loads is necessary.
According to Kennedy (1976), local and overall impact damage of RC walls can be classified as shown in Figure

[1] *Assistant Professor, Structural Engineering Research Center, Tokyo Institute of Technology, Japan, e-mail: watanabe@serc.titech.ac.jp*
[2] *Graduate student, Structural Engineering Research Center, Tokyo Institute of Technology, Japan, e-mail: obara.t.ac@m.titech.ac.jp*
[3] *Professor, Structural Engineering Research Center, Tokyo Institute of Technology, Japan, e-mail: kono@serc.titech.ac.jp*
[4] *Professor, Department of Civil and Environmental Engineering, National Defense Academy of Japan*
[5] *Graduate student, Department of Civil and Environmental Engineering, National Defense Academy of Japan*
[6] *Professor, Disaster Prevention Research Institute, Kyoto University, Japan, e-mail: tanaka@sds.dpri.kyoto-u.ac.jp*

Application of Performance-Based Seismic Design to Tall Ordinary Shear Wall Buildings in Low to Moderate Seismicity Region

Dae-Hwan Kim[1], Minhee Lee[2], Taejin Kim[3] and Yongjei Lee[4]

SUMMARY

The objective of this study is to apply the Tall Building Initiative guidelines to design of tall ordinary shear wall buildings in low to moderate seismicity region like Korean peninsula. The possible issues during its application were studied and the suggestions were made based on the findings from the performance-based seismic design of a building with typical residential multi-unit layout. The lateral-force-resisting system of the building is ordinary load-bearing shear walls with a code exception of height limit. In order to allow the exception, the serviceability and the stability of the ordinary shear wall structure need to be evaluated to confirm that it has the equivalent performance as the one designed under the Korean Building Code 2009. The structure was evaluated whether it satisfied its performance objectives to withstand Service Level and Maximum Considered Earthquake.

Keywords: *Performance-Based Design, Service Level Evaluation, MCE Level Evaluation, Nonlinear Response History Analysis, Ordinary Shear Wall*

INTRODUCTION

For a couple of decades, the significant progress based on findings from recent earthquakes and considerable research has given more explicit approach and procedure for performance-based seismic designs. Although the recent development by PEER (TBI) is still not enough descriptive for structural engineers who are used to simplicity of current codes and standards, it covers detailed aspects involved in conceptual framework of performance-based seismic design and its application. Even TBI addresses lots of aspects for new tall buildings, the application have been mainly used in special shear wall buildings in US. Because there is a height limitation in local jurisdiction codes, most economic application of current status of the status-of-the-art performance based seismic design is focused on special shear wall system under this height limitation.

The Korean building industry embraced the same approach to apply ordinary shear wall system buildings, which has similar height limitation of 60 m at soil profile of Sd. Per KBC 2009 the ordinary shear wall system is not allowed at soil profile, Sd, which triggers more restrictive seismic design criteria. TBI recommends that a tall building should be designed for serviceability for service level effects and stability for ultimate limit states which are commonly used in other loadings to provide compliance of the current standards as the philosophy of Load-Resistant-Factored Design. While the main goal of TBI is to provide the equivalent performance in current seismic design codes with analyses of two discrete performance objectives as aforementioned, the performance objective of the Korean *guideline* [AIK 2015] is limited to merely the Life Safety at so-called the ‘Design

[1] Vice President, Nabih Youssef Associates Structural Engineers, USA, e-mail: dkim@nyase.com
[2] Engineer, Chang Minwoo Structural Consultant, Korea
[3] President, Chang Minwoo Structural Consultant, Korea, e-mail : taejin@minwoo21.com
[4] Sr. Associate, Chang Minwoo Structural Consultant, Korea

DESIGN EXAMPLES OF HIGH-RISE STEEL BUILDINGS USING SM570 STEEL AND F14T BOLTS

Heui-Yung CHANG[1], Chutsen LIAO[2], and Ker-Chun LIN[3]

SUMMARY

In this paper, two 40-story steel office buildings have been designed for the site of Taipei Microzonation II. One of the buildings use SN490B steel (yield strength of 325 MPa) in the box columns. The other also uses SM570 steel (yield strength of 430 MPa) in the 1^{st}-to-14^{th} -story columns. Both the buildings have SM490B steel H-shaped beams. Design examples were also given of web connections using F10T bolts (tensile strength of 1000 MPa) and F14T bolts (tensile strength of 1400 MPa). It was found that the use of SM570 steel plates helps reduce the sizes of base columns, saving 20% steel material. Similarly, the application of F14T bolts to web connections helps reduce 30% labor cost. Furthermore, the buildings with SM570 steel columns are expected to save maintenance cost, because the seismic drift demands and variation are relatively smaller. The examples have shown the benefits of applying high-strength steel materials to tall buildings in seismically active areas.

***Keywords**: high-rise buildings; seismic design; SM570 steel; F14T bolts*

INTRODUCTION

Over the decades, continuous efforts have been made to study high strength steel and the engineering applications. Plate slenderness limits and design strength of stub columns were assessed of the Australian Steel Structures Standard AS 4100, the Load and Resistance Factor Design Specification of the American Institute of Steel Construction (AISC-LRFD), the British Standard BS 5950 Part 1 and the European Convention for Structural Steelwork Eurocode3 for steel with yield strength of 690 MPa. For the ease of fabricating and welding in tall or long-spanned large structures, full-scale testing was made on the beam-to-column connections using 600MPa steel (SM 570 TMC) (e.g. Im and Chang 2003; Kim et al 2008).

To increase the living space in urban areas, tall buildings have been constructed more than ever. For the ease of construction, it is recommended to use high strength construction materials. The structural performance and economic effects were then evaluated for high-strength RC tall buildings in seismic regions (Laogan and Elnashai 1999). Optimization techniques were also applied to reduce the cost and CO2 emission of SRC columns in tall buildings (Park et al 2013). Taipei 101 gives an example of applying SM570 steel to tall buildings in the real world, but limited efforts have been made to analyze the cost-benefit effects (Liou 2014).

In this paper, case studies have been made to evaluate the benefits of applying SM570 steel and F14T bolts to tall buildings in seismically active areas. In detail, two 40-story steel moment frames were first designed for the Taipei Microzonation II. One building uses SN490B steel in the columns. The other uses SM570 steel to replace SN490B in the columns from the 1st to the 14th story. Design examples were also given of web connections using F10T and F14T bolts. The seismic evaluation was then made using nonlinear time history analysis with 14 sets of ground motions scaled to have the return periods of 475 years and 2500 years. By FEMA 356, the building fragility is further assessed using the drift ratios of 2.5% and 5% for life safety and collapse prevention.

[1] *Associate Professor, National University of Kaohsiung, Taiwan, e-mail: hychang@nuk.edu.tw*
[2] *Assistant Professor, National University of Kaohsiung, Taiwan, e-mail: liaoct11@nuk.edu.tw*
[3] *Research Fellow, National Research Center on Earthquake Engineering, Taiwan, e-mail: kclin@ncree.org.tw*

SEISMIC HAZARD LEVEL FOR THE RETROFIT OF EXISTING BUILDINGS

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

SUMMARY

Lower seismic hazard level has been applied for seismic evaluation and retrofit of existing buildings than for seismic design of new buildings. Seismic hazard reduction for the retrofit of existing buildings accounts for remaining lifespan of existing buildings shorter than new buildings and cost effectiveness of retrofit subjected to limited performance of existing part of structural systems. For example, ASCE 31-03 used for the evaluation existing buildings in USA adopted 75% of seismic loads for new buildings and such reduction of seismic load has been adopted in many regions with strong seismicity. Recently published ASCE 41-13 applies reduced seismic hazards prescribed using the probability of annual exceedance to both evaluation and retrofit. In this study, a simple method to calculate reduced seismic demand for retrofit of existing buildings is proposed and validated based on seismic fragility of nonlinear SDOF systems for Korean seismic hazard. The probabilities of limit state are calculated and compared for SDOF systems with different levels of added strength corresponding to different target remaining lifespans. Characteristics of SDOF systems influencing the probability of limit state are hereby identified.

Keywords*: seismic hazard, existing building, remaining lifespan, probability of limit state.*

INTRODUCTION

In many guidelines or codes, seismic hazard has been reduced for the application to the retrofit of existing buildings. For example, ASCE 31-03 for evaluation applies 75 % of seismic hazard for new buildings to existing buildings, while ASCE 41-06 for retrofit design does not reduce the seismic hazard even for existing buildings (ASCE, 2003; ASCE 2006). In spite of conservatism of ASCE 41-06, local building codes used in strong seismicity regions such as Long Beach, Los Angeles, Oakland and San Francisco have applied 75 % of seismic hazard for new buildings to retrofit existing buildings (ASCE 2014). In California Building Code (1998~), seismic hazard of 20%/50 years is applied to the retrofit of public buildings rather than 10% / 50 years for new buildings. Taking such practice into account, ASCE 41-13 that integrates ASCE 31-03 and ASCE 41-06 defines seismic hazard for existing buildings with greater probability of annual exceedance than ASCE 41-06 (ASCE 2014). NZSEE Guideline recommends existing buildings to have 67% of seismic capacity corresponding to new buildings (NZSEE, 2006). In Switzerland, Pre-Standard SIA 2018 (2004) utilizes the compliance factor, a ratio of the seismic demand over capacity considering cost-effectiveness in retrofit, in order to determine implementation and appropriate retrofit measures considering remaining building life (FOEN, 2008).

There are a couple of reasons for that reduction of seismic hazard. First of all, short remaining life of existing building results in relatively low probability of seismic event. In addition, existing parts of buildings that are not designed seismically increase retrofit cost to obtain the same performance level equivalent to new buildings. Even seismically-designed buildings cannot meet the performance objective assumed in the seismic design after deterioration of building materials. In the current study, concentrating on probability of seismic event, a seismic hazard reduction method considering remaining building life is proposed. Then, a validation procedure evaluating limit state probability based on fragility analysis is proposed and applied to simplified inelastic

[1] *Associate Professor, Division of Architecture and Urban Design, Incheon National University, Republic of Korea, e-mail: jhpark606@inu.ac.kr*
[2] *Graduate Student, Department of Architecture, Incheon National University, Republic of Korea, e-mail: wings616.ha@gmail.com*

SUPERTALL COMPACT CITY ABENO HARUKAS

Kiyoaki Hirakawa [1], Kazuhiro Saburi [2], and Souichirou Kushima [3]

SUMMARY

HARUKAS is a vertical city type superhigh-rise building, beyond the bounds of a mixed-use complex and so designed as to maximize the performances of a terminal station, a department store, an art museum, offices, a hotel, an observatory, parking spaces and other diverse uses and functions, which are shifted with different footprints and stacked.
This skyscraper uses a Linked Void Structure utilizing functional, environmental and structural voids, where the infrastructure connecting the vertical city components to one another serves not only as environmental voids through which light and air pass but also as structural voids that withstand seismic, wind and other external forces.
HARUKAS is a high-grade vibration-damped skyscraper with one grade higher seismic and wind performance and has one grade higher seismic and wind performance than those of general skyscrapers in Japan, one of the world's most earthquake- and typhoon-ridden countries, thus ensuring higher safety.

Keywords: *Expecting the unexpected, Innovative framing plan, Effective energy absorbing mechanism, Combination of excellent aerodynamics performance and wind control*

1. INTRODUCTION

ABENO HARUKAS(hereinafter "HARUKAS") is Japan's tallest skyscraper, standing at 300 meters, which was completed in March 2014 (Fig. 1).
The project site is situated in Abeno, Osaka which is a city representative of Western Japan and the world's seventh largest metropolitan area. Abeno is one of the major areas of Osaka, and this area has been growing fast and drawn the most attention in recent years.
ABENO HARUKAS is a building located immediately above the Abenobashi Station owned and operated by the private railway company Kintetsu Corporation. This building is a superhig1h-rise vertical city with the gross floor area of approx. 212,000 square meters. Rising 60 stories above the ground and 5 underground stories, this tower incorporates diverse functions: a terminal station, a department store, an art museum, offices, a hotel, an observatory, parking spaces and more. No other building of this scale has been built above a station in any place of the world.

ABENO HARUKAS stands out from other general skyscrapers because of the following three noteworthy features:
(1) This is a vertical city type skyscraper beyond the bounds of a mixed-use complex;
(2) The existing building was reconstructed into this skyscraper; and
(3) A high-grade vibration-damped building was constructed in Japan, the world's most earthquake-ridden and typhoon-ridden country.

[1] *Structural Design Department, Takenaka Corporation, Osaka, Japan hirakawa.kiyoaki@takenaka.co.jp (corresponding author)*
[2] *Structural Design Department, Takenaka Corporation, Osaka, Japan kazuhiro.saburi@takenaka.co.jp*
[3] *Structural Design Department, Takenaka Corporation, Osaka, Japan souichirou.kushima@takenaka.co.jp*

A STEEL COLUMN BASE CONNECTION WITH SHAPE MEMORY ALLOY RODS

Cheng-Chih CHEN[1], Rung-Shiuan TSAI[2], Huan-Wei LIN[3], and Jheng-Yi LIN[4]

SUMMARY

This study aims to investigate behaviors of the self-centering and energy dissipation of a steel column base connection with shape memory alloy (SMA) rods. Instead of using traditional welded or bolted means, the connection between the column and foundation used super-elastic austenite SMA to achieve the self-centering and without residual deformation of the column. To explore the effects on the self-centering and energy dissipation of the subassembly, the parameters studied included the gravity force at the column tip and the SMA rods. The material test of the SMA rod demonstrated that the heat treatment significantly influenced the super elasticity. The hysteresis curves of the test results showed that the subassembly was capable of the self-centering and energy dissipation attributed to the gravity force and the use of the SMA rod. The increase of the gravity force resulted in the increase of the decompression moment of the subassembly and the decrease of the rotational stiffness after gap opened.

Keywords*: self-centering, shape memory alloy, SMA, energy dissipation, column base connection.*

INTRODUCTION

Special moment frames are frequently used to resist seismic force. However, the structural beams will be expected to form plastic hinges to dissipate energy exerted from the earthquake when structures are subjected to a severe earthquake. Permanent deformation of the structures will be occurred and causes problems of usage and retrofit of the structures. In recent years, a self-centering design philosophy is proposed so that the structures will have no residual deformation after the earthquake (Englekirk 2002). Thus, post-tensioned frames have been intensively studied, and the research emphasizes on the beam-to-column connections (Ricles et al. 2001 and 2002; Christopoulos et al. 2002; Sause et al. 2006; Garlock et al. 2007; Chou and Lai 2009) and column base connections (Chi and Liu 2012).

Shape memory alloys are characterized the behavior of that its deformation can be eliminated after heating or removal of the stress. Recently, the technology has been available to lower the transformation temperature of the shape memory alloy to room temperature. Therefore, the shape memory alloy can eliminate its deformation at room temperature by releasing the stress. The shape memory alloys possess shape memory and super-elastic effects. The use of these effects can lead to self-centering and energy dissipation without heating (DesRoches et al. 2004). The applications of shape memory alloys in building structures have been increasingly. The shape memory alloys are utilized at beam-to-column connections, braces, and shear walls (Ocel et al. 2004; Auricchio et al. 2006; Liao et al. 2006; Sepulveda et al. 2008; Zhang and Zhu 2008).

In this study, the shape memory alloy rods are mounted between the steel column base and foundation. When the

[1] *Professor, Dept of Civil Eng., National Chiao Tung University, Taiwan, e-mail: chrischen@mail.nctu.edu.tw*
[2] *Graduate student, Dept of Civil Eng., National Chiao Tung University, Taiwan, e-mail: jason771016@hotmail.com*
[3] *Graduate student, Dept of Civil Eng., National Chiao Tung University, Taiwan, e-mail: ball1017.cv96@g2.nctu.edu.tw*
[4] *Graduate student, Dept of Civil Eng., National Chiao Tung University, Taiwan, e-mail: jylin.cv97g@nctu.edu.tw*

MODAL PROPERTIES AND SEISMIC APPLICATION OF ASYMMETRIC TUNED MASS DAMPERS

Sung-Yong KIM[1], and Cheol-Ho LEE[2]

SUMMARY

Since the first introduction of tuned mass damper (TMD) in the early 20th century, major TMD applications were directed toward suppressing vibrations caused by wind, harmonic or walking excitations. However, the use of TMDs for seismic application has been rather limited because the efficiency was not well-accepted. The concept of asymmetric tuned mass dampers (ATMDs) was previously proposed by the authors for floor vibration control. Because the concept of the ATMD proposed is generic, this study investigates the applicability of ATMD to seismic application. Modal properties of ATMDs were characterized analytically and experimentally. A case study was then conducted to investigate the effectiveness of conventional and asymmetric TMDs in mitigating seismic damage. Especially, evaluation of the effectiveness of TMDs in face of off-tuning effect due to nonlinear behavior under a severe seismic motion was among the key research interests. Seismic collapse resistance of the structures equipped with TMDs was enhanced by about 5 to 10% depending on the design condition. This study also confirmed that the presence of TMDs is more effective in reducing cumulative inelastic damage than for suppressing the single maximum peak response. Overall, the proposed ATMDs outperformed conventional TMDs by about 5%, in terms of Park and Ang damage index, in a wide range of inelastic response.

Keywords: tuned mass damper; asymmetric tuned mass damper; vibration control; seismic design; robustness

INTRODUCTION

Tuned mass damper (TMD) is an energy absorbing device used to reduce vibration of a primary structure. Since the concept of TMD first appeared in the early 20th century (Frahm, 1911), a lot of studies were carried out for proper selection of TMD parameters. For damped vibration absorbers, which are still being used for most practical application, Den Hartog (1956) developed an analytical procedure and derived the optimal tuning parameters of TMDs for un-damped structure subjected to a harmonic loading. Warburton (1982) provided the optimal parameters of TMDs for damped structures under diverse types of excitations. Conventional TMDs have been used under various types of loading conditions such as wind loading, walking and harmonic forces (Kareem et al., 1999). Kwok and Samali (1995) clearly showed the TMD efficiency in wind-induced response through extensive full-scale measurements. Further, attenuating vibrations under the footfall excitation were also tried by various researchers (Setareh and Hanson, 1992, Setareh et al., 2006, Kashani et al., 2012).

Although the use of TMDs for seismic application was often tried by several researchers, no generally-accepted conclusions on the efficiency of TMDs under seismic excitations have been reached yet. For example, Gupta and Chandrassekaran (1969) claimed that conventional TMDs are not effective in attenuating the response under seismic excitations modeled as sinusoidal loads. Using a TMD with a mass ratio of 0.026% relative to the first-mode effective mass, Sladek and Klingner (1983) found that the optimum TMD designed by Den Hartog's solution is not effective in reducing the maximum seismic response of tall buildings. Clark (1988) reported that single TMD is not effective in reducing earthquake induced building motion, but multiple TMD systems can yield reduction between 40% and 60% for a 5% increase in the mass of the building. However, some other researchers reported the efficiency of TMDs under seismic excitations. Wirsching and Yao (1973) and Wirsching and Campbell (1973) pointed out that the absorber system is effective for moderate earthquake of lesser intensity. They noted that the effectiveness of the absorber may diminish when the primary structure is in the elasto-plastic range because the

[1] *Graduate Student, Department of Architecture and Architectural Engineering, Seoul National University, Korea, e-mail: sungyong.kim7@gmail.com*
[2] *Professor, Department of Architecture and Architectural Engineering, Seoul National University, Korea, e-mail: ceholee@snu.ac.kr (corresponding author)*

EFFECT OF WELD ACCESS HOLE AND FLEXURAL STRENGTH ON DEFORMATION CAPACITY UNTIL FRACTURE OF MOMENT CONNECTION OF TALL STEEL BUILDING

Kouhei TAKATSUKA[1], Keiichiro SUITA[2], and Tsuyoshi TANAKA[3]

SUMMARY

This study analyzes deformation capacity until fracture of welded beam-to-column connections of tall steel building subjected to long-period ground motion and the effect of the weld access hole and the flexural strength of moment connection. For this purpose, deformation capacity of 4 types of the welded beam-to-column connections, i.e. (1) weld access hole detail and low strength (SCW), (2) weld access hole detail and high strength (SCS), (3) no weld access hole detail and low strength (NSW), and (4) no weld access hole detail and high strength (NSS) were investigated.
Time history analysis for a tall steel building subjected to a long-period ground motion was conducted. The fracture of welded beam-to-column connections was estimated by the prediction method which was proposed by previous research using the crack propagation at the CJP weld of a beam flange. In this analysis, long-period ground motion was inputted repeatedly until fractures of moment connections. This analysis reveals that the effect of the weld access hole and the flexural strength on deformation capacity of moment connection under seismic response. The earliest fractures of moment connections are occurred at 1st input for SCW type, while the latest fractures of moment connections are occurred at 5th input for NSS type.

Keywords: *welded moment connection; deformation capacity; long-period ground motion; weld access hole; flexural strength.*

INTRODUCTION

Recent advancement of seismology has enabled precise prediction of long-period ground motions produced by ocean-trench earthquake such as the Nankai trough earthquake in Japan. The characteristics of these ground motions are different from conventional strong motion records by near fault earthquakes such as El Centro 1940, Taft 1952 and so on. The predominant periods are longer by two second or more and the duration of motion can be as long as ten minutes or so, therefore the tall buildings are strongly shaken as was seen in the Great East Japan Earthquake (Kasai K. (2012)). Furthermore, long-period ground motions cause a very large number of cyclic deformation in plastic range to moment connections of steel tall buildings and may cause fracture consequently (Suita K. (2012)).

A number of fractures of welded moment connection were seen in the 1994 Northridge earthquake in U.S. and the 1995 Kobe earthquake in Japan. After these earthquakes in Japan, in order to avoid brittle fracture, two welding details have been recommended in AIJ JASS 6 (2007) as shown in Fig. 1. One is weld access hole detail whose toe is modified to have a 10 mm radius intended to reduce the stress concentration, which is commonly used at field weld. The other is no weld access hole detail which was proposed to avoid premature fracture

[1] *Assistant Professor, Dept. of Architecture and Architectural Eng., Kyoto University, Japan, e-mail: takatsuka@archi.kyoto-u.ac.jp*
[2] *Professor, Dept. of Architecture and Architectural Eng., Kyoto University, Japan, e-mail: suita@archi.kyoto-u.ac.jp*
[3] *Professor, Dept. of Architecture, Kobe University, Japan, e-mail: tanaka@arch.kobe-u.ac.jp*

EXPERIMENTAL ASSESSMENT FOR ROTATIONAL CAPACITY OF COLUMN-TREE STEEL MOMENT-RESISTING CONNECTION WITH BOLT-SLIP

Kyungkoo LEE[1], Jinwon SHIN[2]*, Eunji SON[3], Seungpyo HONG[4], and Eungoo JEONG[5]

SUMMARY

This paper presents an experimental assessment for rotational capacity of column-tree steel moment-resisting connection with the effect of bolt-slip. The bolt-slip effects lead to the yield rotation capacity greater than that of welded steel moment connection and result in bearing force redistribution, improving the energy dissipation capacity. When the span-to-depth ratio increases, the flexural resistance capacity, in the early stage of bolt slip, tends to decrease, whereas the magnitude of stiffness reduction and the yield rotation capacity is likely to increase. Accordingly, for reasonable evaluation for seismic performance of existing column-tree steel moment connections, the moment-rotation relationship should be addressed with the effects of span-to-depth ratio and bolt slip.

***Keywords**: Column-tree, Steel moment-resisting frame, Seismic load, Rotational capacity, Span-to-depth ratio*

INTRODUCTION

Steel moment-resisting frame is a structural system that exhibits an excellent ductility capacity and has been widely used to achieve better seismic performance in zones of high seismicity. However, the 1994 Northridge earthquake in the United States and the 1995 Kobe earthquake in Japan have shown that the existing steel moment-resisting frames may fail in a brittle manner during earthquake primarily due to localized stress concentration in the vicinity of their beam-column connections. To resolve this issue, the United States has proposed qualified moment connection design through the SAC Project for seismic applications and has performed a study to evaluate the seismic performance of the proposed connection design. This has provided a basis on performance model and assessment criteria established by FEMA 350 (FEMA 2000) and ASCE 41-13 (ASCE 2014) to practically evaluate the seismic performance of steel moment connection. Alternate moment connection schemes for the moment-resisting frame in areas of high seismic hazard have been developed based on the criteria described above to resist anticipated seismic forces and to minimize the seismic demands (Chen et al. 1996, Engelhardt and Sabol 1998).

A column-tree connection is one of moment-resisting frames, which has been widely used in Korea and Japan. The column-tree beam-column joint is fabricated by welding in a shop. The bracket welded to the column is typically 0.6 to 1 m long. The girder (link beam) is then spliced to the bracket of the erected column-tree

[1] *Associate Professor, Dankook University, Korea, email: kklee@dankook.ac.kr.*
[2] *Research professor, Dankook University, Korea, email: shin3@buffalo.edu (corresponding author).*
[3] *Researcher, Dankook University, Korea, email: 72150203@dankook.ac.kr.*
[4] *Researcher, Dankook University, Korea, email: ddokki88@nate.com.*
[5] *Researcher, Dankook University, Korea, email: dmsrn98@nate.com.*

MODIFIED EFFECTIVE SLENDERNESS RATIO FOR LIGHT GAUGE BUILT-UP COMPRESSION MEMBER

Kazuya MITSUI[1], Atsushi SATO[2], Hisayuki OKADA[3] and Tetsuro ONO[4]

SUMMARY

In Steel Framed House, built-up member is used for stud. Usually, stud is composed by two light gauges and interconnected by self-drilling screws. In Japan, flexural buckling strength of light gauge built-up compression member is evaluated by effective slenderness ratio that is specified in the Design Standard for Steel Structure. The formula specified in the Standard is based on Bleich's assumption which was applied for heavy section. From this reason, it is questionable that the formula specified in the Standard can be directly applied in light gauge. Firstly, full scale testing were conducted to clarify this issue. Batten type light gauge built-up members were used for specimens in this study. Connectors and chords were connected by pretention bolts. Length of the members were determined to have flexural elastic buckling. From the testing, un-conservative results were observed from all specimens when the flexural buckling strength were evaluated according to the Standard. Additionally, bending moment distribution at the chord that were computed from strain gauge reading differed from Bleich's assumption. Secondly, energy equilibrium theory was used to derive flexural elastic buckling strength of the built-up members. Moreover, modified effective slenderness ratio was proposed for light gauge built-up members. The effects of distance between connectors and separation between chords were included in the formula. Proposed formula was arranged to show these two parameters clearly. Finally, supplemental results were obtained from finite element analysis to verify the proposed formula. From full scale testing and parametric finite element analysis, good agreements were observed from all test specimens when the flexural elastic buckling strength were evaluated according to proposed modified effective slenderness ratio. Compared to Standard, good agreements and improved results were observed from all analysis models when the flexural elastic buckling strength were evaluated according to proposed modified effective slenderness ratio.

Keywords: *Steel framed house; Light gauge; built-up compression member; Buckling stress; Modified effective slenderness ratio*

INTRODUCTION

In Japan, Technical Notification (Ministry of Land, Infrastructure, Transport and Tourism 2001) for Steel Framed House was issued by the Ministry of Land, Infrastructure and Transport in 2001. Light gauge members have become available to the main structure members, and a design using light gauge members was enabled. Also, Technical Notification was revised (Japan iron and steel federation 2014), story limit has been extended up to four story and the middle-rise building has become possible.

Steel Framed House (SFH) model and the detail of shear wall are shown in figure 1. The shear wall of SFH is designed as cantilever column, the lowest story shear wall resist against overturning moment from upper stories. The increase of total story increases the axial force of the built-up member of the shear wall. The shear wall of

[1] *Graduate Student, Dept. of Scientific and Engineering Simulation, Nagoya Institute of Technology, Japan, e-mail: k.mitsui.395@nitech.jp*
[2] *Associate Professor, Dept. of Scientific and Engineering Simulation, Nagoya Institute of Technology, Japan, e-mail: sato.atsushi@nitech.ac.jp*
[3] *Professor, Dept. of Architecture, Aichi Institute of Technology, Japan, e-mail: h_okada@aitech.ac.jp*

A MULTI-FUNCTIONAL FLOOR ISOLATION SYSTEM USING VARIABLE-STIFFNESS SLIDING ISOLATORS

Lyan-Ywan Lu[1], Liang-Wei Wang[2], Shih-Wei Yeh[3]

SUMMARY

Compared to structural base isolation, floor isolation is a more cost-effective and efficient means for seismic protection of vibration-sensitive equipment in a building structure. However, in order to protect the precision equipment, floor isolation usually has to satisfy more stringent isolation performance than that of base isolation; at the meantime, it also calls for less isolation displacement demand due to the indoor space limitation. In order to satisfy these multiple demands, in this study, a multi-functional floor isolation system (FIS) that consists of several variable-stiffness sliding isolators, called polynomial friction pendulum isolators (PFPIs), is proposed and studied. Due to its variable-stiffness nature, the proposed system is able to achieve the desired dual performance objectives that were selected during the design stage for two-level seismic loads. The variable-stiffness hysteretic property of the proposed system was verified by a shaking table test conducted on a prototype PFPI-FIS. Moreover, by using the parameters of the prototype system, the seismic performance of the prototype PFPI-FIS under ten ground motions, which represent earthquakes with different spectral contents and intensity levels, are investigated numerically. The simulated results demonstrate that the isolation performance of the PFPI-FIS does comply with the designated dual performance objectives, which yield either acceleration or displacement control depending on the earthquake intensity and isolator drift.

Keywords: Floor isolation, multi-functional isolator, variable stiffness, pendulum isolator, polynomial function, performance design.

INTRODUCTION

The task of providing seismic protection for interior equipment or non-structural components is important, because the damage of these components may cause operational interruptions of critical facilities or buildings. Relevant research also indicates that attaching vibration-sensitive equipment to the structural floor may prevent the problems of overturning and excessive displacement, but it fails to mitigate the seismic excitation that is usually magnified by the dynamic effect of the underlying structure and can cause serious damage to the equipment (Achour et al., 2011). Seismic isolation which uncouple isolated object from the sources of excitation may provide an effective solution to reduce the amplification of

[1] *Corresponding author: Professor, Department of Civil Engineering, National Cheng Kung University, 1 University Road, Tainan 701, Taiwan. Email: lylu@mail.ncku.edu.tw, Tel: 886-6-2757575 ex. 63138.*

[2] *Graduate Student, National Cheng Kung University, Tainan, Taiwan.*

[3] *Doctoral Student, National Cheng Kung University, Tainan, Taiwan.*

STUDY ON PASSIVE CONTROL DESIGN METHOD FOR MDOF SYSTEM COMPOSED OF BILINEAR + SLIP MODEL ADDED WITH VISCO-ELASTIC DAMPER

Kazuhiro MATSUDA[1], and Kazuhiko KASAI[2]

SUMMARY

Simplified theories are proposed for seismic response evaluation and preliminary design of "Bi-linear + Slip model" assumed wooden structures added with visco-elastic dampers. They are based on the single-degree-of-freedom (SDOF) idealization of multistory building, and produce the control performance curve expressing the peak responses as a function of stiffness of all components, based on spectral characteristics of the earthquake. Against the target peak response, a rule to convert the SDOF design to multistory design, with consideration to distribute damper stiffness over the building height, is also presented. Accuracy of the approach is demonstrated by passive control design and time history simulations on a variety of 2-, 3- and 5-story models.

***Keywords**: Passive Control Design; Bilinear + Slip Model; Equivalent Linearization; Response Spectrum; Visco-elastic Damper; Performance Curve*

INTRODUCTION

Vibration control devices (VCD) are becoming widely used in wooden house structures to reduce the structural vibrations and damage produced by earthquakes. As an effective way to improve the structural energy dissipating capacity, the small-sized visco-elastic (VE) dampers are popularly used due to their economy and constructional efficiency. Many kinds of experimental and analytical studies on the passively controlled timber structure have been carried out (Sakata et al. 2008, Matsuda et al. 2008, 2011), and the effectiveness of damping devices on reducing the response of timber structures has been verified. However, the general design approach is still lack, and the study on the effect of pinched hysteresis on response of passive control system is deficient.

Authors have proposed many design methods to date based on equivalent linearization and response spectra in combination with various mainframes and dampers (Kasai et al. 2003 and 2005). In studies of earthquake resistance, the bilinear + slip (BS) model is often used to express the hysteresis of wooden structures. In this study, the performance curve will be applied into the BS model structures controlled by VE dampers, and a passive control design method particularly for slip-hysteretic structures will be developed.

EQUIVALEMT LINEARIZATION THEORY OF BILINEAR + SLIP MODEL

Outline of Bilinear + Slip (BS) Model

In this study, the bilinear + slip model which are connected a bilinear element and a slip element in parallel (Fig. 1) is used for expressing the timber structure seismic performance. In steady state, the hysteretic energy is dissipated by only a bilinear element. Each stiffness K_1, K_2, K_3 are obtained from bilinear element stiffness K_{B1}, K_{B2} and slip element stiffness K_{S1}, K_{S2} (Eq. 1). Displacement of turning points of skeleton curve are set as u_c and

[1] *Assistant Professor, Structural Engineering Research Center, Tokyo Institute of Technology, Japan; matsuda@serc.titech.ac.jp*
[2] *Professor and Director, Structural Engineering Research Center, Tokyo Institute of Technology, Japan; kasai@serc.titech.ac.jp*

Variable Voltage Sensing Systems for Measurement of Wave Height in TLCDs

Seok-Jung JANG[1], Junhee KIM[2], and Kyung-Won MIN[3]

SUMMARY

In this study, vertical wave motion of a Tuned Liquid Column Damper (TLCD) is measured by a variable voltage measurement system in the electric field and design parameters of the TLCD are determined. A novel liquid level measurement system is developed for measuring vertical motion of the TLCD. For the experimental achievement, experimental characterizations of natural frequency and damping ratio of the TLCD are undertaken utilizing the developed variable voltage sensing. Shake table testing is performed to determine the dynamic characteristics of the TLCD. As a result, the feasibility of the proposed liquid level measurement system is verified by comparison with a conventional capacitive wavemeter.

***Keywords**: Tuned liquid column damper; Variable voltage measurement; Shake table test; Natural frequency; Damping ratio*

Introduction

Recently construction industry is in a trend to increase tall and slender buildings by virtue of state-of-the-art construction and material technologies. However, the tall and slender structures are vulnerable to dynamic loading as wind and earthquakes. For decades, many studies on control devices of the high-rise buildings have been conducted (Kareem et al. 1995; You et al. 2009; Iwan et al. 1972; Wen et al. 1980). Among these studies, Tuned Liquid Damper (TLD) and Tuned Liquid Column Damper (TLCD) have gathered research interests to control of dynamic responses of the buildings. A TLD is simply an open water tank of shapes of rectangle or circle. It is a device for reducing dynamic responses of a building setting the liquid sloshing frequency in a TLD be tuned to the natural frequency of a building (Sun et al. 1992; Lee et al. 2010). Motion of the TLD entails non-linear behaviors, such as jump frequency phenomenon, i.e., peak of liquid levels in the frequency domain is sharply decreased beyond the natural frequency of a TLD, and hardening phenomenon, i.e., the experimental natural frequency of a TLD is greater than that of the theoretical value (Reed et al. 1998; Yu et al. 1999; Yalla et al. 2001; Olson et al. 2001). On the other hand, A TLCD is composed of a U-shaped tube connecting a horizontal and two vertical columns (Sakai 1989). The U-shaped tube facilitates oscillating motion of liquid in a TLCD (Min et al. 2014). By virtue of numerous configuration parameters relating the U-shape tube, frequency tuning is feasible especially in a TLCD. Thus, the practical advantage of a TLCD over a TLD has frequently been emphasized in the literature (Min et al. 2005; Wu et al. 2005; Yalla et al. 2000).

A TLCD has been world-widely adopted in One Wall Center (Vancouver, Canada), Random House (New York, USA), First World Apartment (Songdo , Korea), just to name a few. Prior to installation of a TLCD at a site, a factory test for verification and tuning of dynamic characteristics must be conducted with a pre-fabricated TLCD. Among the dynamic characteristics of the TLCD, optimal tuning frequency and damping ratio are considered primal, since they are directly related to control performance, i.e., vibration suppression of the primary structure. Liquid height of the oscillating liquid column of the TLCD is measured during the factory test and the dynamic characteristics are then estimated from liquid height data measured. A conventional sensor, capacitive wavemeter,

[1] *Graduated Student, Department of Architectural Engineering, Dankook University, Yongin, Korea*
[2] *Assistant Professor, Department of Architectural Engineering, Dankook University, Yongin, Korea, e-mail: junheekim@dankook.ac.kr*
[3] *Professor, Department of Architectural Engineering, Dankook University, Yongin, Korea, e-mail: kwmin@dankook.ac.kr*

STIFFNESS CONTROLLABLE MASS DAMPER SYSTEM WITH LEAST ENERGY CONTROL METHOD

Shih-Yu Chu[1], Lyan-Ywan Lu[2], Shih-Wei Yeh[3] and Chih-Te Chien[4]

SUMMARY

The semi-active mass damper (SAMD) whose designed parameters can be adjusted according to structural responses through the application of appropriate control force to the original passive device has been developed in recent years. To improve the applicability and to avoid the time delay effect of active control, a semi-active control system called stiffness controllable mass damper (SCMD) is proposed in this paper. The least energy control method (LECM) adopted by SCMD is to minimize the whole system’s energy by properly adjusting the SCMD stiffness according to structural responses and input excitation. As a result, the controllability and the damper stoke requirement of the SCMD system with LECM are better than a passive tuned mass damper system. Furthermore, the control performance of a hybrid mass damper (HMD) system is compared with the proposed SCMD system. The numerical results subjected to El Centro earthquake and Northridge earthquake show that the damper stroke of the SCMD system with LECM is smaller than that of a HMD system.

Keywords*: Semi-active Control, Stiffness Controllable Mass Damper, Least Energy Control Method, Hybrid Mass Damper.*

INTRODUCTION

In recent years, the applications of TMDs for aseismic design are increasing [Lin et. al. 2010; Miranda 2005; Pinkaew et. al. 2003]. But the optimal frequency ratio and damping ratio of a TMD need to be determined by parametric study for the best control performance [Marano and Greco 2011; Miranda 2011; Abdulsalam et. al. 2009; Hoang et. al. 2008; Marano et. al. 2007]. In usual, it is difficult to fabricate a TMD whose damping ratio is exactly equal to the designed optimal value in practice; therefore, some researchers proposed a TMD system combined with active control device which is called Hybrid Mass Damper (HMD).

A typical HMD system is adding an active element to a TMD system, and placing sensors on the HMD and the structure for measuring their responses. So, a HMD applies an active force to the mass damper in real time from the feedback responses and its control law [Guclu and Yazici 2008; Han and Li 2006; Chu et. al. 2005; Guclu

[1] *Associate Professor, Department of Civil Engineering, National Cheng Kung University, Taiwan; sychu@mail.ncku.edu.tw (corresponding author)*
[2] *Professor, Department of Civil Engineering, National Cheng Kung University, Taiwan; lylu@mail.ncku.edu.tw*
[3] *PhD Candidate, Department of Civil Engineering, National Cheng Kung University, Taiwan; n68981042@mail.ncku.edu.tw*
[4] *Junior Engineer, Fifth Maintenance Office, Directorate General of Highways, Ministry of Transportation and Communications, Taiwan; a2620998@gmail.com*

The 17th Taiwan-Korea-Japan Joint Seminar on Earthquake Engineering for Building Structures
SEEBUS2015, Yokohama, Japan, September 18-19, 2015

ELASTO-PLASTIC BEHAVIOR OF SHEAR TYPE DAMPER USING LOW-YIELD-STREHGTH CIRCULAR HOLLOW SECTION
(Effective aspect ratio, diameter and diameter-thickness ratio)

Jin-Woo KIM[1], Susumu KUWAHARA[2], Kazuaki MIYAGAWA[3],
Tomohiro KINOSHITA[4], Wataru KITAMURA[5] and Seiji MUKAIDE[6]

SUMMARY

The present study investigates the mechanical behavior of a shear-type damper using a low-yield-strength circular hollow section (LYCHSD). When using a LYCHSD as a seismic energy absorbing device, the hysteresis characteristics and deformation capacity of the LYCHSD should be evaluated accurately. Thus, monotonic and cyclic loading tests were carried out to the LYCHSD, taking the aspect ratio and diameter as variable parameters. The test results were used to establish a method of calculating the full plastic shear strength and initial stiffness. From the result, it was found that the deformation capacity decreases as the aspect ratio and diameter increase. In addition, the monotonic loading test was also simulated using the finite element analysis (FEA) in order to verify the test results and to evaluate the local strain around the welding part due to the crack.

Keywords: *Low-yield-strength circular-hollow-section damper; Aspect ratio; Diameter; Collapse mechanism; Deformation capacity; FEM analysis;.*

INTRODUCTION

A shear-type hysteretic damper is a device that absorbs seismic energy by plastic deformation of steel. This damper mitigates damage to main frames, such as columns and beams, caused by earthquakes. Therefore, low-yield steel is often used for hysteretic dampers because its stiffness is similar to that of common steel but it yields before common steel. Moreover, low-yield steel has a high deformation capacity. In addition, when a circular hollow section is used as a hysteresis damper, it differs from the normally-used H-shaped cross-section in that its deformation characteristics are independent of the direction of the external force. Therefore, the widespread use of circular hollow sections in applications that must deal with displacement in any direction, such as seismic isolation layers, can be expected in the future.

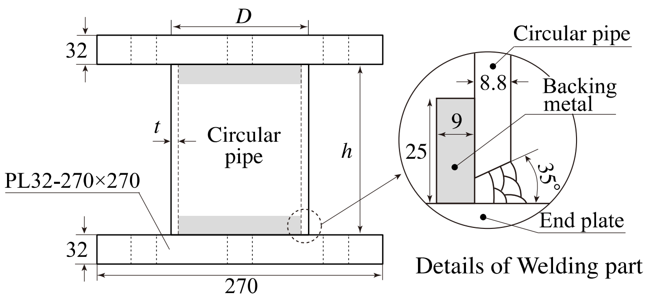

Fig. 1 : Details of Specimens
(Cross-sectional shape : ϕ175X8.8)

The present study attempts to reveal the hysteresis characteristics of initial stiffness, full plastic shear strength, and deformation capacity for a shear-type hysteretic damper with a low-yield-strength circular hollow section

[1] *Ph.D Candidate, Dept. of Architectural Eng., Osaka University, Japan, e-mail: kim_jinwoo@arch.eng.osaka-u.ac.jp*
[2] *Associate Professor, Dept. of Architectural Eng., Osaka University, Japan, e-mail: kuwa@arch.eng.osaka-u.ac.jp*
[3] *General Manager, JFE Civil Engineering & Construction Corp., Japan, e-mail: miyagawa@jfe-civil.com*
[4] *Senior Researcher, JFE STEEL Corp., Japan, e-mail: to-kinoshita@jfe-steel.co.jp*
[5] *Deputy Manager, JFE Civil Engineering & Construction Corp., Japan, e-mail: w-kitamura@jfe-civil.com*
[6] *Assistant Professor, Dept. of Architectural Eng., Osaka University, Japan, e-mail: mukaide@arch.eng.osaka-u.ac.jp*

STRUCTURAL TESTING AND BEHAVIOR OF MULTI-BOLTED JOINTS IN PULTRUDED FIBER REINFORCED POLYMER I-BEAMS

Chung-Che Chou[1], Pi-Fan Sun[2], Kuo-Chun Chang[1], Fang-Yao Yeh[3]

ABSTRACT

Fiber Reinforced Polymer (FRP) composites have been increasingly used in civil engineering application due to their good properties such as high specific strength/stiffness, lightweight and corrosion resistance. Typhoons, floods and earthquakes have caused large natural disaster in Taiwan over the years. In order to provide quick emergency support, a simple construction of a temporary bridge becomes critical for the transportation of foods and medical supplies into the emergency areas. This study focuses on (1) the behavior and strength of multi-bolted joints in FRP plates, made from pultruded FRP I-beam and (2) the ultimate flexural capacity and failure mode of the pultruded FRP I-beam that consists of multi-layer carbon/E-glass with vinyl-ester resin. A total of 30 joint tests were first carried out to determine the effects of joint geometry on load-carrying capacities for FRP flange (20 mm thick) and FRP web (18 mm thick) that were used for connection design in the FPR beams. Then, six FRP beams (410×200×18×20 mm) that were manufactured in Taiwan were experimentally conducted under four-point bending to investigate their structural behaviors and performances of multi-bolted joint connections at the mid-span of 6 m-long FRP beams.

Keywords: Fiber reinforced polymer (FRP) composites, multi-bolted joints, pultruded FRP I-beams, four-point bending test.

INTRODUCTION

Fiber Reinforced Polymer (FRP) composites have been increasingly used in engineering applications due to their good properties, such as high specific strength/stiffness, lightweight and corrosion resistance [Bakis et al. 1997, Karbhari et al. 2000]. To start the development of this cutting edge technology in Taiwan, a project funded by the Ministry of Science was lunched in 2012 for developing a temporary FRP bridge that can be used for rescue purposes after the natural disaster. Taiwan experiences a lot of natural disasters, such as typhoons, floods and earthquakes. Many buildings and bridges were damaged in the 1999 Chi-Chi earthquake and 2009 Typhoon Morako, causing isolated communities in mountain areas to which the emergency supplies could not be easily delivered after the disaster. Therefore, a deployable lightweight bridge for this purpose with transportation of light trucks and short erection time is needed. The aim of the project was to support the design of a temporary bridge that had a span of 20 mm and a width of 3 m, and a live load of 5 ton truck for transfer of rescue goods. To shorten the erection time of the temporary bridge in the field, a multi-bolted joint to connect FRP I-beams that can be fabricated by a

[1] *Professor, Department of Civil Engineering, National Taiwan University, Taipei, Taiwan. Email:* cechou@ntu.edu.tw.
[2] *Graduate Student, National Taiwan University, Taipei, Taiwan.*
[3] *Researcher, National Center for Research on Earthquake Engineering (NCREE), Taipei, Taiwan.*

EXPERIMENTAL STUDY ON MACRO SYNTHETIC FIBER REINFORCED CONCRETE (MSNFRC) CONFINED COLUMNS UNDER CONCENTRIC AXIAL COMPRESSION

Iswandi IMRAN[1], Rosidawani[2]

SUMMARY

Research studies have shown that fibers in reinforced concretes can improve post-peak behavior, ductility and energy-dissipation capacity of concrete elements under flexure, shear or axial loads. Ductile behavior is one of the essential characteristics required for structural elements of buildings located in moderate to high seismic regions. This paper presents an experimental investigation on a series of nine macro synthetic fiber-reinforced concrete (MSNFRC) circular confined columns. The objective of the studies is to investigate the effects of the macro synthetic fibers on strength and ductility of the high strength concrete columns. The test parameters are the spacing of the spiral confining reinforcement and the volume fraction of the macro synthetic fibers. Six of the specimens have the macro synthetic fibers added. In these specimens, volumetric ratio of the spiral confining reinforcement is less than the code-minimum requirement. The remaining specimens are reinforced in accordance with the code minimum requirement for the spiral reinforcement, but with no added fibers, in order to examine the influence of the fibers on the measured strength and ductility. In the tests, a concentric axial compression load is applied monotonically to the column specimens. The results from this study show that the addition of macro synthetic fibers in high strength concrete columns leads to significant improvement in ductility but slight improvement in strength of the specimens. Moreover, based on this study, the code required spiral confining reinforcement may be reduced when combined with higher volume fraction of macro synthetic fibers. It should be noted that the addition of macro synthetic fibers do not directly increase the effective confinement index of the confined concrete columns.

Keywords*: macro synthetic fiber, ductile, confinement, compression*

INTRODUCTION

Previous studies have shown that the effect of steel fibers inclusion in concrete on the compressive strength ranges from negligible to marginal. Among the researchers, Mansur et al. (1999), Campione et al. (1999), and Hsu et al. (1994) have already demonstrated that steel fibers inclusion increases compressive strength and ductility. They also have proved that the combination of discrete steel fibers and transverse steel confinement may reduce the amount of confining reinforcement required by the design codes for high strength concrete (HSC) columns. Some other studies show the advantage of adding discrete fibers to HSC mixtures in reinforced concrete columns, i.e. preventing premature separation of the concrete cover (Foster et al. 1998, Paultre et al. 1996). Furthermore, study on mortar with steel fibers has shown that the addition of fibers leads to increase in mortar compressive strength and the strain at peak stress (Fanella et al. 1985). Investigation on the effect of fiber content on the post-peak softening regime of the compressive stress-strain curve of steel fiber-reinforced concrete (SFRC) found that an increase in the volumetric ratio of steel fibers leads to a relatively flatter post-peak softening regime of the curve (Fanella et al. 1985). Despite a large variety of fibers being developed for FRC (steel, glass, natural, and synthetic fibers) (ACI 544 (1996)), research studies on structural applications tend to focus on the use of steel fibers. Only a few attempts have been made to use the synthetic fiber inclusion for improving the concrete compressive strength and ductility. Several published research studies (Djumbong et

[1] *Professor, Institut Teknologi Bandung, Indonesia, e-mail: iswandi@si.itb.ac.id*
[2] *PhD Student, Dept. of Civil Engineering, Institut Teknologi Bandung, Indonesia, e-mail: rosidawani@gmail.com*

PREDICTION OF LATERAL LOAD DEFLECTION CURVES FOR RC SHORT COLUMNS FAILED IN SHEAR

Yi-An LI[1], and Shyh-Jiann HWANG[2]

SUMMARY

Short columns in building structures are generally considered to be members with high stiffness and governed by brittle shear failure. In the mechanism of structure, short columns are often the first member to fail. Therefore, lateral load deflection curves for short columns have significant influence on the seismic assessment. In this study, a trilinear relationship is proposed to simulate the lateral load deflection curves of short columns. The turning points in the structural behavior are defined as shear cracking, shear strength and collapse point, and straight lines connect these points. Strength at the cracking point is obtained in accordance with the calculation of the strength of shear cracking walls given in ACI 318. The lateral deflection at the cracking point is calculated from a formula based on elastic mechanics. The shear strength of the short columns is evaluated using the Softened Strut-and-Tie Model, and the shear deformation of cracked reinforced concrete is estimated from the strain field in that model. When short columns lose the vertical load-carrying capacity, the lateral shear strength is assumed to be zero, and the associated lateral deflection is estimated using recommended values in ASCE/SEI 41-13. A comparison with published experimental results verified that the proposed curves produce reasonable predictions.

Keywords: *load deflection curve, reinforced concrete, shear deformation, shear strength, short column*

INTRODUCTION

Short columns often appear in buildings due to openings for indoor ventilation or air-conditioning. Reinforced concrete short columns are categorized as short and deep members. Due to their high stiffness and the brittle shear failure during earthquake, short columns often sustain relatively large lateral forces and are the first member to fail. Therefore, their lateral load deflection curves have significant influence on the structures' seismic assessment. To predict the lateral load deflection curves of short columns, in addition to accurately evaluating the shear strength, the change in stiffness needs to be known since it affects the distribution of the lateral load among short columns and other members. Moreover, after shear failure, lateral strength degradation must be clearly defined as it greatly affects the redistribution of internal forces.

According to experiments (Huang and Hwang 2008; Li et al. 2014) conducted on short columns with a height-to-depth ratio less than 2, the force transfer mechanism in short column members is consistent with the strut-and-tie model. Shear failure is presented in concrete crushing at the ends of struts. Therefore, strut-and-tie models should be used to estimate the shear strength. Short columns differ from other typical columns, and lateral stiffness of short columns is greatly reduced once shear cracking occurs. Under the effects of shear cracking, shear deformation accounts for a major part of the total lateral deformation of short columns. Data on the percentages of deformation component at the peak strength shows that the proportion of shear deformation

[1] *PhD and postdoctoral fellow, National Taiwan University, Taipei, Taiwan, e-mail: d96521008@ntu.edu.tw*

[2] *Professor, National Taiwan University, and the deputy director, National Center for Research on Earthquake Engineering, Taipei, Taiwan,*

e-mail: sjhwang@ntu.edu.tw

The 17th Taiwan-Korea-Japan Joint Seminar on Earthquake Engineering for Building Structures
SEEBUS2015, Yokohama, Japan, September 18-19, 2015

SHEAR CRACK CONTROL FOR HSRC BEAMS CONSIDERING THE EFFECT OF SHEAR-SPAN TO DEPTH RATIO OF MEMBER

Chien-Kuo Chiu[1]

SUMMARY

This study tests ten full-size simple-supported beam specimens with the high-strength reinforcing steel bars (SD685 and SD785) using the four-point loading. The measured compressive strength of the concrete is in the range of 70-100 MPa. The main variable considered in the study is the shear-span to depth ratio. Based on the experimental data that include maximum shear crack width, residual shear crack width, angle of the main crack and shear drift ratio, a simplified equation are proposed to predict the shear deformation of the HSRC beam member. Besides of the post-earthquake damage assessment, these results can also be used to build the performance-based design for HSRC structures. And using the allowable shear stress at the peak maximum shear crack width of 0.4 mm and 1.0 mm to suggest the design formulas that can ensure serviceability (long-term loading) and reparability (short-term loading) for shear-critical high-strength reinforced concrete (HSRC) beam members.

Keywords: *High-strength reinforced concrete; Shear crack; Serviceability, Reparability*

INTRODUCTION

According to its design standard, the Architectural Institute of Japan (AIJ) [1] states that building performance consists of serviceability, safety and reparability. Restated, in addition to serviceability and safety, the performance-based design of buildings should incorporate reparability as a factor. As a major determinant in the cost of a building over its life cycle, reparability can also be regarded as a basic economic performance metric of a building; its importance has become evident in many seismic disaster events, including the Northridge Earthquake (USA, 1994), the Kobe Earthquake (Japan, 1995), and the Chi-Chi Earthquake (Taiwan, 1999). Reparability can ultimately reduce reconstruction costs after a seismic disaster. Additionally, a crack-based damage assessment plays a major role in estimating repair costs of a building. Despite the numerous crack-based damage assessments of RC members or structures, related studies have focused mainly on normal-strength RC with little attention paid to HSRC structural members. A crack-based damage assessment can also estimate post-earthquake residual seismic capacity or facilitate damage-controlled design (performance-based) for a building structure.

Given the emphasis on seismic capacity or safety of HSRC in related studies, this study presents design formulas that ensure the serviceability and reparability of HSRC beam members based on experimental results. Therefore, this study investigates the shear crack development of shear-critical HSRC beam members, especially with respect to the relationship between shear stresses and shear crack widths. By setting the allowable shear stress at the peak maximum shear crack width of 0.4 mm and 1.0 mm, this study also derives design formulas that can ensure serviceability (long-term loading) and reparability (short-term loading) for shear-critical HSRC beam members. Additionally, to quantify damage in nonlinear dynamic analysis, the relationship between shear crack width and deformation of a member should be determined based on experimental results in terms of the shear crack, results of which can help engineers to assess the performance or damage state of members under an earthquake in the structural analysis. Correspondingly, in this study, ten full-size simple-supported beam specimens with high-strength reinforcing steel bars (SD685 and SD785) are tested using four-point loading. Design compressive strength of the concrete is 70 and 100 MPa, and the shear-span to depth ratio is 1.75, 2.00,

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

Cyclic Loading Tests for Low-Rise RC Walls with 550 MPa Shear Bars

Baek, Jang-Woon[1], and Park, Hong-Gun[2]

SUMMARY

In the construction of nuclear power plants using massive walls, the use of high-strength reinforcing bars for shear design is necessary, to enhance the constructability and economy. However, in current design code for nuclear power plants, KEPIC SNB and ACI 349, the maximum yield strength of shear re-bars is limited to 420 MPa. Thus, the present study focused on the use of higher strength bars for shear design of RC walls. A series of tests for walls with aspect ratios of 0.5, 1.0, and 2.0 were planned to verify the validity of Grade 550 MPa bars for shear reinforcement. Cyclic lateral loading test was performed for the specimens. The major test parameters were the aspect ratio, grade of shear reinforcement, and failure mode. The test results were directly compared with those of specimens with Grade 420 MPa bars, which is currently specified as the maximum yield strength of shear reinforcement. The results showed that the structural performance of walls with 550 MPa were comparable to those of walls with 420 MPa bars, in terms of failure mode, shear strength, ductility, and average diagonal shear crack width. In particular, the ratio of the test strength to the prediction of ACI 349 was 1.48 to 1.83, which indicates that the walls with 550MPa shear bars are safe against the design load. The test results can be used as an evidence for the applicability of 550 MPa bars to the shear design of walls.

Keywords: cyclic loading, high-strength reinforcing bar, shear strength

INTRODUCTION

In the construction of nuclear power plants, a number of large diameter reinforcing bars are used in massive reinforced concrete (RC) walls, which significantly degrades the constructability and economy. Thus, to enhance the constructability and economy, the use of high strength bars needs to be considered.

The maximum yield strength of shear reinforcement is limited in current design codes. In Eurocode 2 (2002) and CSA (2004), the permitted maximum yield strength of shear reinforcement is 600 and 500 MPa, respectively. In JSCE (2002), the use of Grade 800 MPa bars is permitted when the concrete compressive strength is greater than or equal to 60 MPa. In KEPIC SNB and ACI 349 (2006) [or ACI 318 (2011)], the maximum yield strength is limited to Grade 420 MPa which is less than those of other design codes. Generally, the yield strength of shear reinforcement is limited 1) to ensure yielding of shear reinforcement before shear failure, and 2) to control the width of potential diagonal shear cracks. Currently, test results for walls with high strength shear reinforcement are limited. Particularly, the structural performance under earthquake loading needs to be verified.

In the present study, cyclic lateral loading tests were performed for walls ($h_w/l_w \le 2.0$) with Grade 550 MPa bars to investigate the shear strength before and after flexural yielding. The major test parameter was the wall aspect ratio, grade of shear bars, failure mode. The test results were compared with the shear strengths predicted by current design codes. On the basis of the results, the effect of Grade 550 MPa bars on the structural performance of walls with $h_w/l_w \le 2.0$ was discussed.

EXPERIMENTAL PROGRAM

TEST SPECIMENS

The tests were performed for walls with three aspect ratios: 1) h_w/l_w= 0.5: 1500 mm (length) x 750 mm (h eight) x 200 mm (thickness), 2) h_w/l_w =1.0: 1500 mm x 1500 mm x 200 mm, and 3) h_w/l_w =2.0: 1500

[1] *Graduate Student, Department of Architecture and Architectural Engineering, Seoul National University, Korea, e-mail: baekja1@snu.ac.kr*
[2] *Professor, Department of Architecture and Architectural Engineering, Seoul National University, Korea, e-mail: parkhg@snu.ac.kr (corresponding author)*

LATERAL FORCE-DISPLACEMENT PREDICTION OF UNCORRODED AND CORRODED REINFORCED CONCRETE BEAMS USING MODIFIED AXIAL-SHEAR-FLEXURE INTERACTION APPROACH

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

SUMMARY

A modified axial-shear-flexure interaction approach (MASFI) is proposed for predicting the lateral force-displacement response of uncorroded and corroded reinforced concrete beams. Similar to the original ASFI, two interactive models are considered in the MASFI: the axial-flexure model, which uses fiber analysis to predict flexural behavior, and the axial-shear model, which uses modified compression field theory to evaluate shear behavior. The effect of flexural behavior on shear behavior is taken into account by incorporating axial strain extracted from the axial-flexure model into the axial-shear model. Compared to the original ASFI, two modifications are proposed in the MASFI. First, the flexural compression zone in the axial-flexure model is softened by using average transverse reinforcement strain in the MASFI instead of the principal tensile strain in the ASFI. Moreover, core concrete confinement, and buckling behavior of compression bars are reduced in the MASFI with increasing transverse reinforcement strain induced by shear force, which is not considered in the ASFI. The effects of corrosion are considered in the MASFI by softening cover concrete in compression, decreasing the cross-sectional area, yield and ultimate strengths and ultimate strain of steel reinforcement, and modifying crack spacing due to bond reduction. Residual cross-sectional area of longitudinal reinforcement is calculated based on average corrosion weight loss. Effects of transverse reinforcement on concrete confinement and bond strength are estimated based on minimum residual diameter of transverse reinforcement. Shear strength contributed by steel reinforcement is computed based on the average value of average corrosion weight loss and maximum corrosion weight loss at the pitting location. The MASFI with and without corrosion considerations are verified using experimental results of three uncorroded and 18 corroded RC beams.

Keywords: reinforced concrete beams; corrosion; residual shear strength; ductility.

INTRODUCTION

Displacement-based seismic design for new reinforced concrete structures and seismic evaluation for existing RC structures requires to comprehensive understanding about lateral load-displacement behavior of reinforced concrete members (ATC-40 (1996) and FEMA 356 (2000)). Therefore, estimation load-displacement capacity of RC member has been received a lot of interest in recent years. ATC-40 (1996) and FEMA 356 (2000) provide minimum acceptable values of plastic hinge properties for RC members based on their functions in the structural system; the reinforcement configuration; the material properties; the applied loads; and the geometrical properties.

Considering a RC specimen subjected to in-plane lateral load and/or axial load with one end fixed again rotation and translation at one end and free end at the other end as illustrated in Figure 1 (a). The tip displacement of the RC specimen normally consists of three components: flexural displacement; shear displacement; and pullout deformation. For simplification in engineering practice, the load-displacement behavior of the column could be

[1] *Professor, National Taiwan University of Science and Technology, Taipei, Taiwan, e-mail: yuchenou@mail.ntust.edu.tw*
[2] *Ph.D., National University of Civil Engineering, Ha Noi, Viet Nam, e-mail: dangnguyen1210@gmail.com*

FULL-SCALE EXPERIMENT ON NON-STRUCTURAL R/C WALLS FOCUSED ON FAILURE MODES AND DAMAGE MITIGATION

Masanori TANI[1], Tomohisa MUKAI[2], Masaya OGURA[3], Rafik TALEB[4] and Susumu KONO[5]

SUMMARY

In the 2011 Tohoku Earthquake, many non-structural R/C walls were heavily damaged and the function continuity was broken in many buildings. In this research, analytical and experimental study was conducted to obtain fundamental information of non-structural R/C wall, such as failure mode and relation between lateral deformation and damage state. Failure mode of the non-structural walls which suffered severe damage in the 2011 Tohoku Earthquake was discussed based on the capacity estimation. Four full-scale R/C mullion wall specimens were constructed on the basis of the case study results and tested. The results of case study implied that considerable axial compression load was applied on the walls. Also in the loading test, the specimen with axial compression load showed similar failure mode as seen in the actual buildings. Improvement of maximum lateral capacity and deformation capacity was observed by increasing the amount of horizontal web reinforcement and installing of hook anchorage. In addition to discussions on maximum lateral capacity and failure mode, fundamental experimental data about damage condition especially after unloading was also shown.

***Keywords**: non-structural R/C wall; full-scale specimen; failure mode; damage quantity.*

This paper was presented at the Second European Conference on Earthquake Engineering and Seismology, Istanbul, August 25-29, 2014.

INTRODUCTION

In the 2011 Tohoku Earthquake, structural damage to buildings was not particularly heavy in comparison with the observed JMA seismic intensities, although a part of R/C buildings which was designed by previous building code was heavily damaged. On the contrary, non-structural R/C walls in many government buildings and residential buildings were heavily damaged as shown in Photo 1. In Japan, R/C wall member with large opening which cannot be treated as structural wall by regulations is called “non-structural wall” and its capacity is not generally considered in the structural calculation. Damage of such member does not affect to the safety of building, but the continuity and rehabilitation of building function. Some buildings whose non-structural R/C walls were heavily damaged were given up their rehabilitation and demolished due to the problems of huge repair cost etc., even if the structural damage was not so significant. Behind this situation, there is a problem that structural designers cannot predict what will happen on non-structural wall because information about the relations between structural frame deformation and non-structural wall damage (ex. residual crack width and length, concrete spalling area, etc.), and the effects of damage to the function continuity and reparability (i.e. repair cost) of buildings are extremely limited.

The purpose of this study is to obtain fundamental data of relation between deformation and damage of non-structural R/C wall. First, failure mode of non-structural R/C walls shown in Fig.1 is discussed based on

[1] *Associate Professor, Kyoto University, Japan, e-mail: tani@archi.kyoto-u.ac.jp*
[2] *Senior Research Engineer, Building Research Institute, Japan, e-mail: t_mukai@kenken.go.jp*
[3] *Ex-Graduate Student, Tokyo Institute of Technology (Haseko Corporation), Japan*
[4] *Ph.D Candidate, Tokyo Institute of Technology, Yokohama, Japan, taleb_rafik@yahoo.fr*
[5] *Professor, Tokyo Institute of Technology, Yokohama, Japan, kono@serc.titech.ac.jp*

PERFORMANCE OF POST-TENSIOEND ANCHORS AND POST-TENSIONED BEAM-COLUMN JOINTS

Thomas H.-K. Kang[1], Ah Sir Cho[2], and Byung Un Kwon[3]

SUMMARY

An unbonded post-tensioned (PT) anchor for a 15.2 mm diameter 7-wire strand was developed by Prof. Kang's research team at Seoul National University based on their finite element analysis and experimental verification. As part of the experimental verification of structural performance, static load tests and load transfer tests were conducted following KCI-PS101. The static load tests and additional strand tensile tests confirmed that the developed anchor had a capacity more than nominal tensile strength of a 7-wire strand without any damage or deterioration. According to the result of load transfer tests, specimens with no additional reinforcing bars sustained at least 1.6 times the nominal tensile strength of the strand. This paper describes such a performance verification process conducted through static load and load transfers tests. The developed anchors were then used as fixed end anchorage in monolithically cast beam-column joints of special moment frames. Both roof level and inter-story level PT connection subassemblies were recently tested under reversed cyclic deformations. The preliminary test results showed that seismic performance of the connections with developed anchors was excellent in terms of every measurable seismic indicators.

***Keywords**: Post-tensioned; anchor; static load test; load transfer test; seismic test; joint.*

INTRODUCTION

Post-tensioning (PT) is one of prestressing methods that compensate for concrete which is weak in tension, stressing prestressing steel after concrete is hardened. For this process, post-tensioned (PT) anchors are used as essential structural device. In Korea, the price of imported anchors and accessories is quite high, and PT building systems are not invigorated due to such a high cost. Therefore, domestic structural research and experimentation on PT anchor device is greatly needed.

In this study, a PT anchor for an unbonded PT monostrand is designed based on finite element analyses. Then structural performance of the developed anchor is experimentally verified through static load tests and load transfer tests. The capacity of the anchor is determined using static load tests, and direct comparison of compressive strengths depending on reinforcement details for anchor is made using load transfer tests. Based on the strain data obtained from the load transfer tests, stress distribution in the anchorage zone is also assessed. Finally, seismic applicability of developed anchors is evaluated by performing full-scale quasi-static cyclic tests.

TEST METHODS FOR POST-TENSIONED ANCHOR

According to the concrete standard specification of Korea, the performance of a PT anchor should be examined

[1] *Associate Professor, Seoul National University, Seoul, Korea, e-mail: tkang@snu.ac.kr*
[2] *Doctoral student, Seoul National University, Korea*
[3] *Master's student, Seoul National University, Korea*

AN EXPERIMENTAL STUDY OF THE CYCLIC BEHAVIOR OF LOW ASPECT RATIO SC WALL PIERS

Chang-Ching CHANG[1], Yin-Nan HUANG[2], and Po-An CHEN[3]

SUMMARY

A steel-plate-concrete (SC) composite wall is composed of steel faceplates, infill concrete, and connectors. The faceplates are joined with steel tie rods that ensure structural integrity. Steel studs are welded to the faceplates to ensure composite action. This paper summarizes the results of in-plane cyclic tests of four SC walls. The flexure-critical wall piers are two meters long and one meter high. The faceplates are embedded in the foundation and connected to the foundation concrete using shear studs and transverse reinforcement. The influence of infill reinforcement ratio and wall-foundation connection is studied. The test results confirm the effectiveness of the analytical method of Epackachi et al. (2015) in predicting the in-plane monotonic behavior of low aspect-ratio SC wall piers.

Keywords*: nuclear power plant, steel composite wall, in-plane strength, cyclic loading test*

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. 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. Importantly, most of the studies have addressed shear-critical walls because SC walls have been used in the labyrinthine-type of construction used for containment internal structure. The behaviour of flexure- and flexure-shear-critical walls is not as well understood, in part because only a small number of experiments have been performed on such walls.

Epackachi et al. (2014) tested four SC wall piers with an aspect ratio (height divided by length) of 1.0. The piers were anchored to a re-usable foundation block with a post-tensioned connection. A numerical model for SC wall piers was validated using the test data. Epackachi et al. (2015) developed an analytical procedure to predict the flexure and flexure–shear response of SC wall piers using the calibrated numerical model.

This paper addresses the inelastic response of four SC wall piers subjected to reversed, in-plane cyclic loading. The wall-to-foundation connection for these tests is different from that of Epackachi et al. (2014): the faceplates embedded in the foundation using shear studs and transverse reinforcement passed through holes in the faceplates. The following sections of the paper describe the testing program, present key experimental results, and validate the analytical method of Epackachi et al. (2015).

[1] *Assistant Researcher, National Center for Research on Earthquake Engineering, National Applied Research Laboratories, Taiwan, e-mail: ching@narlabs.org.tw*
[2] *Associate Professor, National Taiwan University, Taiwan, e-mail: ynhuang@ntu.edu.tw*
[3] *Research Assistant, National Taiwan University, Taiwan, e-mail: r02521240@ntu.edu.tw*

BOND ACTION EFFECT OF LONGITUDINAL CUT-OFF BARS ON SHEAR TRANSFER MECHANISM OF RC BEAMS

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

SUMMARY

A method is proposed to estimate the required length of double-layered flexural reinforcing bars for reinforced concrete (RC) beams. This research deals with deep beams in which the shear reaction is dominant and the plastic hinge is not formed. The ACI (American Concrete Institute) and AIJ (Architectural Institute of Japan) guidelines propose several methods to determine the cut-off length of double-layered bars for typical foundation beams. However the theoretical and experimental bases are not yet presented.
A series of experimental works was conducted to measure the bond strength of double-layered bars. A method to estimate the double-layered length and the shear strength of beams with small shear span-to-depth ratio is then developed based on the concept of the partial truss mechanism.

Keywords: *RC beams; bond; shear; high-strength steel bar; cut-off.*

INTRODUCTION

The reinforced concrete beams, which are subject to anti-symmetrical moment and shear force, exhibit tension shift due to the diagonal tension crack occurring at the end of the member. Due to this phenomenon, the bond stress begins to take place at the outside of the area of the tension shift. Thus, the cut-off location of the main re-bar should be determined so as to secure the necessary development length at the outside the area of the tension shift. There are several methods proposed by ACI[1)], AIJ Standard for Structural calculation of Reinforced Concrete Structure[2)], Design Standard for Concrete Structure of KCI[3)] for the determination of the cut-off location on the main re-bar. However, it is the current situation that these methods do not fully reflect the mechanical behavior.

One of example, which illustrates the importance of the determination of the cut-off location, is the foundation beam of the frame structure. The foundation beam must resist the elastic bending moment at the hinge of pedestal of foundation pile. However, because the depth of the beam is very deep and the method to determine the cut-off location in this situation is not established yet, the cut-off on the main re-bar is not exercised and the re-bars are laid out throughout all the length of the beam.

This study assumes a foundation beam, which does not formed plastic hinge, to propose a method to determine the location of the cut-off on the main re-bar. The applicability of the proposed method in this study to the case of forming plastic hinge is left to future research.

OUTLINE OF EXPERIMENTS

Three test specimens were constructed. Figure 1 shows the test specimen and the cross section. All the test specimens have a cover of 16mm on the cross section, shear span ratio of 1.5, and the spacing of 50mm between shear reinforcement. Each specimen has a central test region with 200 x 300 mm sectional

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