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STRUCTURAL CONTROL PERFORMANCE EVALUATION OF AN MR DAMPER BY REAL-TIME SUBSTRUCTURING TECHNIQUE

Sung-Kyung LEE[1], Eun Churn PARK[2], Kyung-Jo YOUN[3], and Kyung-Won MIN[4]

SUMMARY

Real-time substructuring technique (RTST) is a structural dynamic testing method that the numerical calculation of the equations of motion of a structure and the experimental measurement of the reaction forces resulting from the application of this motion to the numerical structure are simultaneously implemented in real time. In this paper, structural control performance of the magneto-rheological (MR) damper installed in a real-scaled 5 story building is experimentally evaluated through real-time substructuring test method. In this method, a numerical substructure is based on a structural model identified from the forced vibration testing results of a real-scaled 5 story building, and an MR damper that is used as an experimental substructure is physically tested with a universal testing machine (UTM). In the test, load cell on the UTM measure the force necessary to attain the required story displacement and these structural reaction forces are returned to the computer for use in the next time step calculation of a numerical structural model. Test results show that the higher level of control force generated by the MR damper causes the lower level of controlled response of a structure.

***Keywords**: Real-time substructuring technique, numerical substructure, experimental substructure, MR damper, real-scaled 5 story building*

INTRODUCTION

By the present time, a wide variety of structural control strategies for civil engineering structures such as bridges and buildings have been proposed by structural engineers. Various types of active and semi-active control dampers also have been developed to enhance the performance of passive control devices. Analytical and experimental studies using MR dampers of these control devices have been performed to reduce the structural response, mainly due to their intrinsic stability and low power consumption. Also, semi-active control strategies using the MR damper were performed by many researchers, and their comparative studies were extensively implemented [Jansen et al. 2000]. In analytical studies, Bingham, Bouc-Wen and phenomenological models were proposed as the analytical model to describe the hysteretic behavior of MR damper [Yang et al. 2004]. Even though these models are useful in the design of MR damper, however, they are inappropriate to characterize the behavior of the MR damper under the excitation of irregular loads such as earthquakes and winds because of its strong nonlinearities such as the dependency on the loading rate and the amplitude of excitations. Also, the performance of MR damper is not guaranteed according to its current providing devices. Moreover, when the MR damper behaves as the semi-active control device, hysteretic model varying with the applied current is unreliable. In these reasons, there can be exist a discordance between the actual responses of the building installed with the MR damper applying the semi-active control strategy and the corresponding analytical results. Accordingly, experimental studies were extensively implemented. One example is found from the experimental results tested for the MR damper with the capacity of 200 kN at Notre Dame University in USA. Another

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CONTINUOUS-OPTIMAL-CONTROL WITH VARIABLE GAINS OF A LARGE-SCALED BASE-ISOLATED STRUCTURE FOR SEISMIC LOADS

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SUMMARY

Validation of Continuous-optimal-control with variable gains (COC-VG) is pursued in this study. COC-VG is designed to manage two 20 kN magnetorheological dampers in the hybrid-controlled base-isolated structure for mitigation of seismic loads. In order to develop a robust controller that is sensitive to a variety of excitations, the traditional clipped-optimal control with variable gains is proposed. Two or more objection matrixes in the LQR control algorithm are designed to handle different intensities of excitations. The designed control gains will vary online with the stroke of the base-isolation system to gain more control efficient with the consideration of the stroke limits. COC-VG are designed and validated through numerical simulation and full-scale experimental shake table tests for a variety of seismic excitation. Results show that COC-VG is robust and effective in reduction of both displacement and acceleration responses for both near- and far-field seismic events.

Keywords: *semi-active control; earthquake engineering; base-isolation; LQR.*

INTRODUCTION

Low power consumption, high reliability and fail-safe operation make the semi-active control technique one of the more promising approaches for mitigation of seismic responses in civil engineering structures. Currently, magnetorheological (MR) dampers are being widely studied for their potential use as semi-active control devices [Chang][Spencer][Lin 2006]. An MR damper resembles an ordinary linear viscous damper but the cylinder of the damper is filled with special fluid that contains tiny polarizable particles. The fluid state can be changed drastically from liquid to solid and vise versa by adjusting the magnitude of an applied magnetic field produced by a coil that is wrapped around the piston head of the damper. When no current is supplied to the coil, the damper behaves as an ordinary viscous damper. On the other hand, when current is sent through the coil, the fluid inside an MR damper becomes a semi-solid, the yield strength of which depends on the level of current applied. Since control commands simply adjust the parameters of a MR damper that is placed in a real structure, control instability never occurs and only a small amount of energy is required. Therefore, MR dampers are reliable and fail-safe.

The base-isolation system can be used in the equipment isolation system, floor isolation system or the structural isolation system as shown in Fig. 1. There are lots of kinds of isolation system; most of them compose of the isolator and some energy dissipation devices. The isolator, such as LRB, RB and FPS, provide the vertical support with suitable lateral stiffness an also some hysteresis. To enhance the control effect or to reduce the stroke of the isolation system, the energy dissipation devices (such as hydraulic dampers) are added. For the same reason, the semi-active controlled MR damper is good to use in the hybrid controlled base-isolation system. In the semi-active control system, the more adjustable range, the more control efficiency. As only the MR damper is adjustable. To maximize the controllable range, the unchangeable hysteresis in the isolator should be

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WIND RESPONSE CONTROL OF A 76-STORY BENCHMARK BUILDING USING A SMART TUNED MASS DAMPER

Hyun-Su KIM[1] and Dong-Guen LEE[2]

SUMMARY

Very often, slender and tall buildings experience excessive wind-induced vibration, which will cause discomfort to occupants or even shatter windows. In this study, a smart tuned mass damper (STMD) was developed for the enhancement of the serviceability of high-rise buildings. To investigate the effectiveness of a STMD for the response control of a wind-excited tall building, a benchmark building was used. The benchmark building considered is a 76-story concrete office tower in Melbourne, Australia. It is a slender building 306 m tall with a height to width ratio of 7.3. Across wind load data from wind tunnel tests are used in the present study. An idealized variable damping device and an MR damper were used replacing a passive damper to improve the control effect of a conventional TMD. These semi-active dampers can change the properties of a STMD in real time based on the dynamic responses of a structure. Groundhook control algorithm is used to appropriately modulate the damping force of semi-active dampers. The control performance of STMD was evaluated comparing with the sample TMD, ATMD and a semi-active variable stiffness TMD employing the numerical simulation of a benchmark building. The numerical studies show that the control effectiveness of STMD is significantly superior to that of the conventional passive TMD. It is also shown that a STMD can reduce the response similar to the active TMD; however, with an order of magnitude less power consumption.

***Keywords**: smart tuned mass damper; vibration control; MR damper; benchmark building; wind response control.*

INTRODUCTION

With higher strength materials and lighter structural systems, building structures become increasingly slender, taller and wind-sensitive. Very often, such slender and tall buildings experience excessive wind-induced vibration, which will cause discomfort to occupants or even shatter windows. Vibration control of tall buildings subjected to wind excitation has been studied extensively, and various types of control devices and algorithms have been developed. The tuned mass damper (TMD) is one of the most widely-used control devices in civil structures for vibration control because it is easy to install no matter for a new or old structure. When properly tuned, it can effectively suppress excessive vibrations of a structure. However, the effectiveness of a single TMD may be impaired because of its high sensitivity to frequency mistuning or to variations of the characteristics of a structure. The mistuning or off-optimum damping of TMD will reduce its vibration control effectiveness significantly. Thus, to improve the effectiveness and robustness of TMD, researchers have proposed the use of more than one tuned mass damper with different dynamic characteristics: multiple tuned mass dampers (MTMD). MTMD is more effective in the mitigation of oscillations of structures than a single TMD. Besides the use of MTMD, the effectiveness of TMD can also be significantly enhanced by introduction of an active force, the AMD, to act between the structure and the additional mass (Ankireddi et al. 1996). Recently, a hybrid mass damper (HMD), the combination of a TMD and an active control actuator, was successfully employed for vibration control of civil structures (Nishitani and Inoue 2001). HMD can improve the adaptability of TMD and

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MODEL REFERENCE ADAPTIVE STRUCTURAL CONTROL ON UNCERTAIN SYSTEMS

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SUMMARY

With the rapid application of digital apparatus and the wide usage of PC-based controllers, the application of adaptive control technique becomes the focus of automatic control practicing engineers and researchers recently. However, its application on civil infrastructures is not well established, especially, by taking the advantage of overcoming parameter uncertainty commonly encountered on the structural vibration control. An adaptive control application based on Lyapunov theory is presented herein. The Lyapunov equation used to define the adaptation law is designed based on a quadratic Lyapunov function candidate. This energy-like scalar function comprises weighted tracking-error states and parameter-estimating-error matrix. The adaptive feedback control force is calculated from both measured states and adjustable parameters estimated from the adaptation law. The error states illustrate the trajectory tracking performance between reference model and controlled system. The global asymptotical stability is guaranteed by choosing a positive definite adaptation weighting matrix for the time-invariant linear system. With the time varying adaptation control force, a simple single-degree-of-freedom model with parameter uncertainty is used to demonstrate the control performance based on a pre-selected target reference model. The advantage of a model-reference adaptive structural control (MRASC) algorithm is to adjust the control command signal while estimating the system's parameters of the controlled structure in real time. The control performance of this MRASC system is verified and compared by using the El Centro earthquake records. The results show that even with parameter uncertainty, the MRASC system can reach the expected control performance of the pre-designed reference model.

Keywords: *active structural control; model-reference adaptive control; nonlinear control; Lyapunov theory; system uncertainty.*

INTRODUCTION

Adaptive control involves modifying the control law used by a controller to cope with the fact that the parameters of the system being controlled are slowly time-varying or uncertain. For example, as an aircraft flies, its mass will slowly decrease as a result of fuel consumption; we need a control law that adapts itself to such changing conditions. Adaptive control is different from robust control in the sense that it does not need a priori information about the bounds on these uncertain or time-varying parameters; robust control guarantees that if the changes are within given bounds the control law need not be changed, while adaptive control is precisely concerned with control law changes. In the traditional structural control framework, researchers usually design time-invariant active control gains based upon minimization of total energy or performance index and implement these control gains through feedback scheme to improve the vibration suppression ability (Soong, 1990; Chu et al., 2005). However, for adaptive control, the time-varying adjustable parameters are generally integrated from parametric adaptation rule, which is designed using direct Lyapunov method in order to achieve asymptotically perfect tracking performance (Slotine and Li, 1991; Åström and Wittenmark, 1995). The basic idea of a Model-

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STIFFNESS OF REINFORCED CONCRETE STRUCTURAL WALL WITH IRREGULAR OPENINGS

Bing LI[1]

SUMMARY

This study addresses the initial stiffness (pre-yield stiffness) of reinforced concrete (RC) shear walls with irregular openings and low aspect ratio. Flexure and shear induced deformations will be taken into account. Based on previous experimental and analytical researches for the behavior of walls with openings, definition for initial stiffness of reinforced concrete structural wall with irregular openings is proposed. An analytical approach for stiffness is also developed. The approach is validated by comparing the theoretical and experimental results from six RC shear walls with irregular openings.

***Keywords**: Wall with Irregular Openings; Initial Stiffness; Flexural Stiffness; Shear Stiffness; Stiffness ratio.*

INTRODUCTION

The major developments in seismic design over the past 15 years have been upon the emphasis on the change from “strength” to “performance” based design. Therefore, the attention has been shifted to the deformation of the structures. To estimate the actual ductility and displacement, or to assess the period of vibration of structures or elements, the correct assessment of the stiffness of these structures or elements becomes inevitable. Some researchers have investigated the initial stiffness of reinforced concrete (RC) solid shear walls, where relative equations for stiffness assessment were proposed [1-5].

In recent years, RC shear walls with staggered openings, also named as walls with irregular openings (see Fig.1), have been utilized in some practical cases. However, few recommendations have been referred to their seismic design. For example, the existing codes and references [6-8] only allow the use of strut-and-tie models to undertake the seismic design of walls with irregular openings. Experiments on RC flanged walls or rectangular walls [9-11] had shown that methods based on strut-and-tie models are viable for the design of RC walls with irregular openings under reversed cyclic lateral forces. Although these experimental results indicated that appropriately designed walls with irregular openings can demonstrate similar behavior and ductility as walls with regular openings, the loading path and the stress distribution together with the failure mode of these kinds of walls are significantly different.

The objective of this research study is to develop a rational approach to calculate the initial stiffness of RC shear wall with irregular openings based on their particular behavior when yielding occurs.

STIFFNESS DETERMINATION FOR A RC STRUCTURAL WALL WITH IRREGULAR OPENINGS

In wall seismic design, bilinear load-displacement curves are generally used instead of non-linear load-displacement curves (see Fig. 2). The slope of the first straight line of bilinear load-displacement curves is the initial stiffness of the structure. There are two common methods [4, 5] used to determine the slope of the

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SHEAR STRENGTH OF COMPOSITE STEEL-CONCRETE WALLS WITH VERTICAL RIBS

Sung-Gul HONG[1], SeungJun LEE[2], Wonki KIM[3], and Donghun LEE[4]

Abstract

This study presents shear strength and deformation capacity models for composite steel-concrete walls (SC walls) of small aspect ratio which are reinforced by vertical ribs. Arch and truss actions for shear transfer in SC walls are considered after diagonal cracking of concrete. Shear behavior depends on the relative stiffness between steel plates and cracked concrete up to the ultimate state. To determine the yielding point von Mises yield criteria for steel plates and Mohr Coulomb criteria for concrete are adopted. Von Mises yield criteria for an isotropic material is modified for an orthotropic steel plate in this study. Steel plates after yielding are modeled as equivalent reinforcing bars in the direction perpendicular to the diagonal struts at ultimate state. Comparison with experimental data shows good agreement with the theoretical results and numerical analysis.

Keywords: *Shear strength; deformation capacity; Orthotropic plates; Diagonal compression fields;*

I. Introduction

Composite steel-concrete wall (SC wall) system is one of viable option to reduce construction duration in lateral resisting system in building structures and nuclear power plant facilities. SC walls with open sandwich slabs as one of modular construction methods for nuclear power plant require the verifaiction of structural performance and efficiency of construction. It is well known that SC walls take advantage of post-buckling behavior of steel plate with rigidity of concrete. Current research on steel plate with pre-cast concrete wall panels shows stable shear resistance against lateral loads

Japanese design guideline for SC wall structures has proposed a skeleton tri-linear curve for SC wall under lateral load by defining cracking point, yielding point, and ultimate point. According to the guideline cracking of concrete inside the steel wall occurs due to diagonal tension and the steel plates are assumed to yield following von Mises yield criteria. At the ultimate state the concrete is assumed to crush. The reduced stiffness of SC wall after diagonal cracking in concrete is maintained up to the point of yielding of steel plate. In this region the cracked concrete is assumed to be an orthotropic elastic material and the steel plate remains in elastic state. The distribution of shear force between core concrete and steel plates are determined by relative stiffness ratio between concrete and steel plates. After the yielding point the plastic state of steel plates with diagonally cracked concrete withstand shear force by arch and/or truss actions until concrete crushes.

The objectives of this study are to investigate the behavior of SC wall with vertical ribs under shear forces

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A MULTIPLE MACRO MODEL FOR REINFORCED CONCRETE STRUCTURAL WALLS WITH OPENINGS

Jiyang WANG[1], Susumu KONO[2], Masanobu SAKASHITA[3] and Hitoshi TANAKA[4]

SUMMARY

In this study, a macro-model of a reinforced concrete structural wall is proposed, where the model is composed of column elements and strut and tie elements which represents the wall parts. The proposed model was established by modifying the model proposed by Tekehara et al. to obtain more accurate load-deformation relationships in the case of multi-story walls with eccentric openings. The main modified items are the constitutive law for the concrete and the additional modeling of the concrete struts and tension ties located around the openings. With respect to the former item, the Vecchio-Collins modified constitutive law of concrete compressive strength was used to take the transverse tensile strain into account, while the Popovic's constitutive law had been used in the model proposed by Tekehara et al. The proposed model was applied to two of the three-storied reinforced concrete structural walls with eccentric openings, where the adjacent walls between the stories were connected by rigid beam elements and the springs expressed by bi-linear models. The results obtained from the proposed model were compared with those from experiments and the model by Takehara et al. and the adequacy of the proposed model was confirmed.

Keywords: *Structural Wall; Macro Model; Reinforced concrete, Strength Capacity, Openings*

INTRODUCTION

The structural walls are normally designed to be the main earthquake resistant components of a building. However, they usually have some openings according to the intention of the architectural design. Extensive experimental and analytical studies have been conducted mainly on the seismic behavior of the structural walls with central openings, although the openings for the entrance/exit are large and eccentric in many cases. However, the previous studies are not sufficient for such cases. When the opening locations are too eccentric and/or the openings are too large, linear or non-linear FEM analyses are often used, even though it takes more time for the design. Hence, it is desirable to establish a simple but rational method. TAKEHARA et al. tried to evaluate the strength and deformation of a single-story structural wall with an eccentric opening by a macro model based on the strut and tie model. Though the good result was found, the applied examples were only a few, and also no method for applying to the cases of continuous walls over the height of a multi-story building was exhibited. Moreover, the interaction between wall reinforcement and concrete struts possibly formed along the perimeter of openings were neglected in their model. The neglect of such interaction will result in an underestimate of seismic strength of walls when the openings are comparatively large. In this study, a multiple macro model which takes account of the possible interaction mentioned above was proposed by modifying the model by Tekehara et al.

The adequacy of the proposed model was testified by carrying out seismic loading tests on two of the 40% scale specimens of reinforced concrete structural walls having different opening ratios. From the comparison between the experimental and the analytical results, it was concluded that the proposed multiple macro model was

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PREDICTION OF LOAD CARRYING CAPACITY OF PRECAST PRESTRESSED CONCRETE COLUMNS

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SUMMARY

The results of loading tests on six precast prestressed concrete columns were reported last year. It showed that the specimens with the higher axial force lost their axial load capacity at relatively small column rotation angle. The amount and detailing of shear reinforcement were added to the experimental parameters and additional four specimens were constructed. Two specimens had tie-bars with high axial force and the other two specimens were tested under smaller axial force than last year. In this paper, loading tests on those ten precast column specimens assembled by post-tensioning which were conducted to investigate seismic behavior, such as failure modes, lateral load capacities and tensile force variation of post-tensioning (PT) bars were reported. Different failure modes were observed depending on axial load levels. An evaluation method for lateral load capacities of the specimens was proposed and proved to be successful.

Keywords: *precast concrete; prestressed concrete; post-tensioned; unbond; column-foundation assemblage.*

INTRODUCTION

Many structures have been built in precast prestressed concrete construction because it has the advantages over ordinary reinforced concrete. Unbonded tendons for prestressing steel make re-tensioning of prestressing tendons and disassembling of jointed members possible. In Japan, use of prestressed concrete members post-tensioned by unbonded tendons had been restricted to secondary structural members such as floor beams and slabs. It was because of concerns that the stress variation in unbonded tendons at the anchorage was larger than that in bonded tendons, which might lead to the low-cycle fatigue failure at the anchorage. However, past research has revealed that stress variation in unbounded tendons is smaller than that in bonded tendons because stress in unbounded tendons is uniform over the whole length. The latest code revision last June enables us to use members post-tensioned by unbounded tendons as primary earthquake-resistant structural members. Experimental and analytical research on post-tensioned beam-column assemblages has been carried out in the past and we now have some amount of knowledge on their seismic behaviors. However, researches on column-foundation assemblages post-tensioned by unbonded or bonded tendons are very few (e.g. Yoon et al. 1996; Ishida et al. 1998; Osako et al. 2003; Nishiyama et al. 2006; Tani et al. 2006).

The results of the loading tests on six precast prestressed concrete columns reported last year showed that the specimens with the higher axial force lost their axial load capacity at relatively small column rotation angle. The confinement of core concrete was supposed to be not enough because all shear reinforcement was peripheral hoop and no tie-bar was provided. Therefore, the amount and detailing of shear reinforcement were added to the experimental parameters and additional four specimens were constructed. Two specimens had tie-bars with high axial force and the other two specimens were tested under smaller axial force than last year.

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DESIGN OF PRECAST CONCRETE BUILDINGS EMULATING MONOLITHIC CONSTRUCTION

Fumio WATANABE

SUMMARY

Seismic design of precast concrete buildings is conducted based on the Design Requirements of the Building Standard Law, which are prepared for monolithic construction. Therefore a designed building is to show equivalent structural behavior to monolithic construction, and specifications for materials and detailing should follow the Building Standard Law.
Equivalent monolithic structural behavior of precast system is generally demonstrated by tests on precast beam-column sub-assemblages, composite beams or columns, and other types of sub-assemblages. Experimentally observed data should satisfy the requirements indicated in the AIJ Guidelines for Structural Design of Precast Connection Emulating Cast-in-place Reinforced Concrete, 2002.

1) Drift at yielding should be greater than 0.8 times and less than 1.2 times the drift at yielding of the emulated monolithic construction. Lateral load at yielding should not less than that of emulated monolithic construction.
2) With regard to the degradation of load carrying capacity during seismic load cycling, the maximum load in the second cycle should be greater than 80% of that in the first cycle in the same drift amplitude.
3) Energy dissipation of a precast system should not be smaller than 80% of that of emulated monolithic construction.

This paper describes the basic requirements to emulation design procedure, typical connection details, design equations of connections for shear and some examples of connection design.

Keywords: *Precast concrete, Emulation design, Equivalent monolithic connection*

GENERAL DESIGN RULE OF PRECAST CONCRETE BUILDINGS

In Japan precast concrete building is generally designed in accordance with the Seismic Design Requirements of Building Standard Law 1981: These are basically prepared for monolithic construction. Therefore a designed building should show equivalent structural behavior to monolithic construction, and specifications for materials and detailing should be followed. This design method is called emulation design method.

Equivalent monolithic structural behavior of precast system is generally demonstrated by tests on precast beam-column sub-assemblages and others. Experimentally observed data is compared with that of simultaneously constructed pair specimens or with past experimental data in view of lateral stiffness, lateral strength, structural ductility and hysteretic behavior (energy dissipation). When well established connection details are used, tests on precast elements or connections are not required. These conditions are indicated in the AIJ Guidelines (***AIJ Guidelines for Structural Design of Precast Connection Emulating Cast-in-place Reinforced Concrete, 2002***) and briefly introduced in the next paragraph.

When the advanced verification procedure (design based on time history analysis) or the performance based

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ANALYTICAL MODEL FOR PREDICTING SHEAR AND DEFORMATION CAPACITIES OF CONCRETE BEAMS UNDER CYCLIC SHEAR

Kyoung-Kyu CHOI[1], and Hong-Gun PARK[2]

SUMMARY

The inelastic deformation of reinforced concrete beams causes the degradation of shear capacity and early shear failure of concrete. In the present study, an analytical model which can evaluate the shear and deformation capacities of RC beams was developed based on material failure criteria of concrete. Addressing the effect of flexural deformation on the shear capacity of concrete, the degradation of the shear capacity of RC beams was evaluated. In addition, primary well-known failure mechanisms of RC beams due to the cyclic shear were incorporated: buckling and fracture of reinforcing bars. The predictions by the proposed analytical model were compared with the results of previous tests. In the comparison, it was found that the deterioration of the shear capacity of RC beams subjected to cyclic shear according to the inelastic flexural deformation was well described by the proposed method.

Keywords: *earthquake design; cyclic shear; reinforced concrete; shear capacity; deformation capacity; failure criteria.*

INTRODUCTION

According to the results of experimental studies performed by Ang et al. (1989), Aschheim and Moehle (1992), and Wong et al. (1993), the shear strength of RC members subjected to cyclic loading is dependent on the inelastic deformation of the members. This is attributed to the fact that the flexure-shear cracks (or orthogonal tensile strain) in the plastic hinge widen as the rotation of plastic hinge increases, and thus the shear transfer by aggregate interlock and the shear strength of the members are deteriorated (Priestley et al. 1994).

To address the degradation of shear strength in hysteretic response, ACI 318-05 (2005) and New Zealand Concrete Code (1982) neglect the concrete contribution to shear resistance (V_c) in potential plastic hinge regions of special moment frames subjected to earthquake load. On the other hand, ATC seismic design guideline (1996) proposed an analytical model using shear capacity curve, which represents the decrease of the shear strength according to ductility (or deformation demand) of the RC members (Fig. 1). Priestley et al. (1994) proposed an improved shear capacity curve for the better correlation with experimental results, and Watanabe and Ichinose (1991) developed an explicit design equation to present the shear capacity curve.

These models mentioned above can illustrate conceptually the relationship between the shear strength and deformability of the RC members at different deformation demand level. In these models, the shear strength and deformability are determined at the intersection between shear capacity and shear demand curves. However, the shear capacity curve, which is used to describe the degradation of the shear strength, was developed empirically based on test results, and they do not represent the shear deformability itself. These empirical formula do not directly assess the damage state according to deformation level. However, in recent seismic design approach for reinforced concrete members, accurate prediction of deformability is emerging as a primary concern (Lee and Watanabe 2003) for ensuring the aimed ductile behavior of the members.

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IMPROVING SEISMIC BEHAVIOR OF RC INTERIOR BEAM-COLUMN CONNECTIONS WITH HEADED REINFORCEMENT

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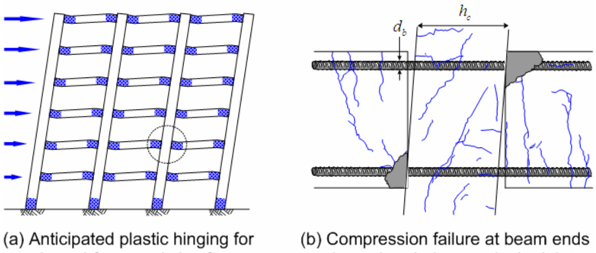
SUMMARY

The straight beam bars passing through interior RC beam-column joints may slip during the formation of adjacent beam plastic hinge. The slip of beam bars reduces the stiffness, strength, and energy dissipation capacity of beam-column connections under cyclic loading. To improve the bond performance of beam bars passing through the joint, innovative detailing with headed reinforcement was investigated by testing of six interior beam-column connections. First two benchmark units with beam bars passing through the joint had significant bond failure beyond 3% drift followed by strength degradation and very pinched hysteretic loops. To enhance the bond of the beam bar, another two units added double mechanical devices on each beam bars within the joint. The cyclic responses of these two units were remarkably improved up to 6% drift level without bond deterioration. The use of mechanical devices is concluded to be a viable option to improve bond performance of beam-column joints under large inelastic displacement reversals. Alternatively, last two units were designed to move the plastic hinge zone away from the joint using second layers of beam bars, which were extended into the beams and terminated by mechanical devices. These two units exhibited excellent cyclic response due to very complete formation of plastic hinging mechanisms. This paper also recommends how to determinate the relocated critical section away from the joint with headed reinforcement.

***Keywords**: beam-column joint; bond; headed reinforcement; plastic hinge; seismic*

INTRODUCTION

Special moment-resisting frames are widely used for the design of reinforced concrete building structures in moderate to high seismic zones. If properly detailed, the plastic hinge can be arranged to develop at the beam regions adjacent to the joint when the frame subjected to large lateral loads, as shown in **Figure 1(a)**. During the formation of these beam plastic hinges, extremely high bond stresses can be developed along the straight beam bars passing through the joint, because these bars may be forced to yield in tension at one column face and be close to yield in compression at the opposite column face. Once certain degree of bond deterioration occurred within the joint, these beam bars may slip within the joint under large load reversals.



(a) Anticipated plastic hinging for a lateral-force-resisting frame

(b) Compression failure at beam ends due to bond damage in the joint

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

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SEISMIC RETROFIT OF RC STRUCTURES WITH PRESTRESSED CFT AND FRC BRACES

Mamoru ODA[1], Susumu KONO[2], and Fumio WATANABE[3]

SUMMARY

This research aims to propose a simple seismic strengthening method which satisfies short construction period and low construction cost by not using rebar or bolt anchorage. A prestressed precast concrete brace system was proposed by authors in 2001 and this study introduces a new brace system using concrete filled tube and fiber reinforced concrete to enhance its deformation capability and esthetics. Two half scale portal frames were constructed based on the old Japanese building standard and strengthened with two kinds of brace without using rebar nor bolt anchorage; one made of precast fiber reinforced concrete and the other made of concrete filled tube. Either brace was a single line element and easily placed in a frame. Braced frames showed more than 70% increase in shear capacity and enhancement in ductility. It was also shown that the current design equations for axial strength of a brace, shear strength of column-beam joint, and bearing strength can be applied to design the proposed brace system.

Keywords: *seismic retrofit; precast-prestressed concrete brace; no bolt anchorage.*

INTRODUCTION

Japanese buildings for earthquakes

After major earthquakes such as the Northridge and Kobe earthquakes in 1990's, the seismic upgrading of existing buildings has been attracting more attention than ever. Upgrading of seismic performance of buildings can be achieved by increasing strength or ductility. However, many existing buildings in Japan have not had seismic upgrading since construction is costly due to intensive labor work and long suspension of service. This research aims to develop a simple seismic strengthening method which satisfies short construction period and low construction cost by not using rebar or bolt anchorage. For this purpose, an X-shaped precast prestressed concrete brace system was developed in 2001 at Kyoto University. In this report, a new brace system made of concrete filled tube and fiber reinforced concrete is introduced for better esthetics and deformation capability.

Previously proposed retrofit method

The previously proposed X-shape precast prestressed concrete brace consisted of four precast units as shown in Fig. 1(a). They were assembled at construction site and prestressing force was introduced to two lower legs. Gaps between brace ends and frame corners were filled with high strength no-shrinkage mortar. After hardening of mortar, the prestressing force was released. Then the X-shape brace extended by itself and was fixed to a boundary frame.

When a frame with an X-shape brace is subjected to lateral seismic load, only one of diagonal members works effectively in compression. However, the remaining diagonal member becomes free because concrete does not

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EXPERIMENTAL STUDY ON AN INTERLOCKING BLOCK INFILL FOR RETROFITTING EXISTING BUILDINGS

Yasushi SANADA[1], Naruhito YAMAUCHI[2], Eri TAKAHASHI[3], and Yoshiaki NAKANO[4]

SUMMARY

This study proposes a new retrofit method for existing structures using masonry infill walls. This paper introduces the concept of a new interlocking block infill developed in this study, and describes its availability. Although no reinforcement is provided in the proposed infill, it can resist out-of-plane lateral loads by the interlocking mechanism between ductile blocks. Therefore, several advantages of this method, such as application of easy-to-install interlocking units, no reinforcement work, and no construction noise and vibration, decrease construction costs and allow buildings to remain occupied during retrofitting. In this study, two reinforced concrete frames were prepared, and one of them was retrofitted by installing the proposed infill. Both specimens were subjected to cyclic lateral loading in the in-plane and out-of-plane directions, to compare their seismic performance. The un-retrofitted frame finally failed in shear and lost lateral and vertical resistance. In the case of the retrofitted one, however, the installed infill supported axial loads instead of the collapsed columns. Moreover, the infill also exhibited a substantial lateral resistance, which was caused by friction between the blocks under axial loads transferred from the collapsed columns.

***Keywords**: cyclic loading test; fiber reinforced cement composite; masonry wall; reinforced concrete frame; seismic performance; seismic retrofit.*

INTRODUCTION

Interlocking blocks have been used not only for paving streets but also for accelerating masonry construction and/or improving structural performance in various countries (Ramamurthy et al. 2004). Focusing on past studies, although construction and/or structural performance was discussed on individual interlocking block walls using many differently shaped blocks (e.g. Hatzinikolas et al. 1986 and Anand et al. 2000), few studies have been reported on their retrofit applications or interactions with other structural elements. On the other hand, seismically vulnerable masonry structures are not common in Japan, based on lessons learned from past earthquake disasters. When retrofitting existing buildings, however, they have several advantages such as utilization of easy-to-handle masonry units, and no noise and vibration during construction work. Therefore, the authors proposed a new retrofit method using interlocking block walls (Sanada et al. 2006 (1)), and have investigated their seismic performance for application to seismically vulnerable structures.

This paper proposes a new detailing interlocking block infill, and introduces its development concept and retrofit application. Availability of the proposed method was discussed through seismic loading tests of two reinforced concrete frames, which were conducted to compare the seismic performance between the retrofitted and un-retrofitted cases.

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SHAKING TABLE TEST FOR VERIFICATION OF EFFECT OF LONG-PERIOD GROUND MOTION ON FURNITURE AND EXTERNAL WALLS OF HIGH-RISE BUILDINGS

Hideo FUJITANI[1], Kazuyoshi OHNISHI[2], Shiro KIDO[3], Hidenori KAWABE[4], Takuya NAGAE[5], Koichi KAJIWARA[6], Kunio FUKUYAMA[7] and Masayoshi NAKASHIMA[8]

SUMMARY

Hyogo Prefecture Government, Japan, is composing a manual for measures against earthquakes, aiming at the prevention of falling of dangerous objects from building and of injury or functional degradation caused by movement of indoor furniture and fixtures. For that purpose, the behavior of exterior materials and external and indoor installations with no appropriate countermeasures in the case of a large earthquake will be studied, by using a shaking table experiment of "E-Defense" of Hyogo Earthquake Engineering Research Center, NIED. This paper outlined the experimental test results.

Keywords: *shaking table test; long-period earthquake ground motion; furniture; external wall; High-rise building.*

INTRODUCTION

Although structural members of building have been verified to have seismic safety of a certain standard level, exterior materials might receive damage in response to deformation that is greater than the designed level by unexpectedly strong ground motion in the event of a large earthquake. Exterior fixtures might cause injury to those outside the building. In particular, high-rise buildings constructed might response with large displacement amplitude for a long time because of ground motion with a dominant long-period component, resulting from a distant inter-plate earthquake1). A pulse-like ground motion with large velocity amplitude contained in a near-fault ground motion might induce a great deformation in the exterior fixtures. Essential subjects in such a situation are: to protect the neighborhood from damage from falling exterior materials or external installations of high-rise buildings; and to prevent any injury by movement and falling of indoor furniture and fixtures and the functional degradation of building.

Hyogo Prefecture Government, Japan, is composing a kind of manual for measures against earthquakes, aiming at the prevention of falling of dangerous objects from building and of injury or functional degradation caused by movement of indoor furniture and fixtures. For that purpose, the behavior of exterior materials and external and indoor installations with no appropriate countermeasures in the case of a large earthquake will be studied by cooperation with Hyogo Earthquake Engineering Research Center, National Research Institute for Earth Science and Disaster Prevention (NIED), by using a shaking table experiment of E-Defense. Then, regarding those for which sufficient countermeasures have been taken, their effects will be verified. Furthermore, the results of this

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SEISMIC ANALYSIS OF ASYMMETRIC BUILDINGS UNDER BI-DIRECTIONAL GROUND MOTIONS

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

SUMMARY

An approximation approach of seismic analysis of two-way asymmetric building systems under bi-directional seismic ground motions is proposed. The procedures of uncoupled modal response history analysis (UMRHA) are extended to two-way asymmetric buildings simultaneously excited by two horizontal components of ground motion. Constructing the relationships of two-way base shears versus two-way roof translations and base torque versus roof rotation in ADRS format for a two-way asymmetric building, each modal pushover curve bifurcates into three curves in an inelastic state. A three degree-of-freedom (3DOF) modal stick is developed to simulate the modal pushover curve with the stated bifurcating characteristic. It requires the calculation of the synthetic earthquake and angle β. It is confirmed that the 3DOF modal stick is consistent with single-degree-of-freedom modal stick in an elastic state. A two-way asymmetric three-story building was analyzed by UMRHA procedure incorporating the proposed 3DOF modal sticks. The analytical results are compared with those obtained from nonlinear response history analysis. It is shown that the 3DOF modal sticks are more rational and effective in dealing with the assessment of two-way asymmetric building systems under two-directional seismic ground motions.

Keywords: *asymmetric buildings; modal analysis, eccentricity; bi-directional ground motions; response time history analysis*

INTRODUCTION

Under the earthquake loads, plane-asymmetric buildings with irregular distributions of mass or stiffness are likely to undergo torsional responses coupled with the translational vibrations. This type of structures is likely to suffer more severe displacement demands at the corner elements under earthquake ground motions. It has been found that this is one of the key factors that have caused many buildings to collapse in recent earthquakes around the world. The seismic analysis of one-way asymmetric buildings has been studied by many researchers (Chopra and Goel 2004; Myslimaj and Tso 2002). However, the most general case for plane-asymmetric buildings under horizontal earthquake loads is the two-way asymmetric type under bi-directional seismic ground motions. Due to the complicated coupling effects between one rotational and two translational inelastic vibrations, it is difficult to analyze the seismic behaviors of such buildings. Not only the translational time histories at center of mass, but also the rotational time history is required to compute the corner responses. Hence, except the time-consuming nonlinear response time history analysis, there seems to be very few simplified methods, which can obtain the approximate response time histories of the noted buildings. Chopra and Goel (2004) have indicated that simultaneous action of two horizontal components of ground motion and structural plans unsymmetric about both axes remained unsolved and required further investigations.

The inelastic response of one-story system with one axis of asymmetry subjected to bi-directional base motion

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EVALUATING DEFORMATION DEMANDS FOR SAC BUILDINGS USING NEAR-FAULT RECORDS WITH AND WITHOUT PULSES

Heui-Yung CHANG[1], Keh-Chyuan TSAI[2], and Ker-Chun LIN[3]

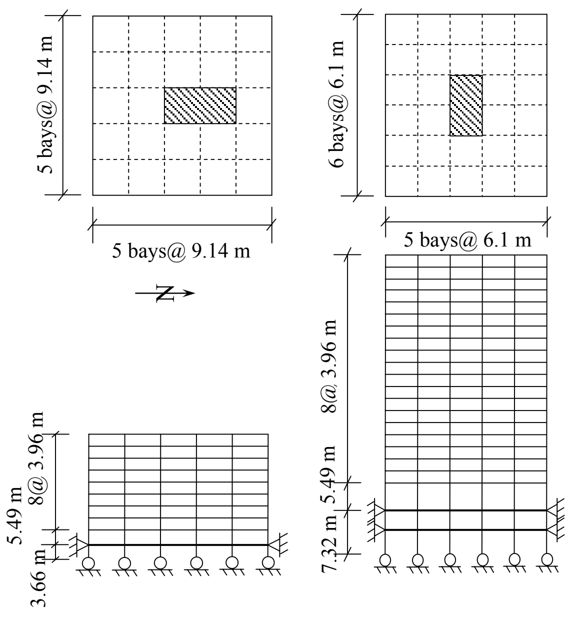
SUMMARY

This study investigates the near-fault pulse effects on the deformation demands of middle-rise and high-rise steel moment frames. The maximum interstory and roof drift ratios and beam plastic rotations (i.e. δ_{max}/H, Δ_{max}/h and $\theta_{p,max}$) were evaluated for the SAC Phase II 9- and 20-story buildings, using a total of 145 near-fault records with and without pulse signals. Analysis results show that near-fault pulses may complicate the global and local deformation demands of middle-rise and high-rise steel moment frames in different ways. The deformation demands of near-fault ground motions, on average, were found to increase by 8%-64% under pulse excitation. The variation has been interpreted as an interaction of structural periods and pulse periods. Despite that, most of the deformation demands were confirmed not to exceed the post-earthquake requirements (e.g. $\Delta_{max}/h \leqq 4\%$ and $\theta_{p,max} \leqq 3\%$).

Keywords: *deformation demands, steel moment frames, near-fault records, pulse signals.*

INTRODUCTION

The connection fractures observed in the 1994 Northridge and 1995 Hyogo-ken Nan-bu (Kobe) earthquakes are thought to highly correlate with the high deformation demands submerged in the near-fault ground motion records in those earthquakes (Somerville et al 1997). Since that, many analytical and experimental studies have been made using near-fault ground motions, especially those which contain pulse signals in the wave form and appear to be more critical to structures. Previous analytical work indicated that the uncertainty in near-fault ground motions may complicate the local deformation demands of structures, and the variation can be interpreted as an interaction of structural periods and pulse periods of near-fault ground motions (Krawinkler et al 2003). The following analytical work proved the point that when the pulse periods of the near-fault ground motions move towards to structural period*s*, the increase in maximum elastic response may pull up (e.g. Akkar et al 2005). A recently reported experimental work also gave some inelastic examples illustrating the above point (Rodgers and Mahin 2006).



(a) SAC-9 **(b) SAC-20**

Figure 1 Floor plans and elevations

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CONTROL PERFORMANCE OF FRICTION DAMPERS INSTALLED IN A SHEAR WALL-FRAME SYSTEM

Ji-Hun PARK[1] , Gil-Hwan KIM[2]

SUMMARY

Control performance of the friction damper installed in a shear wall-frame system, of which main lateral force resisting system is the shear wall, is addressed. A 12-storey building with the shear wall-frame system is designed according to the old Korean structural design code and evaluated for a ground motion based on the enhanced code. The shear wall of the building is modeled with MVLEM (multiple vertical line element model) and frame members are with fiber elements with inelastic material models. Then, damper install schemes for various configurations of the friction damper, different total friction forces and their distribution along storeys are modeled and evaluated through nonlinear time history analysis. Energy dissipation capacity, lateral load reduction and member deformations for various installation schemes and total friction force levels are compared and analyzed to obtain qualitative basic design principles.

Keywords: *friction damper, shear wall, nonlinear time history analysis*

INTRODUCTION

Many reinforced concrete building structures has shear walls as their main lateral force resisting system. Even if the shear wall can resist lateral loads effectively, old buildings may not be designed against seismic loads, or may have insufficient strength due to enhanced design code. In particular, the lower storey shear wall has low ductility capacity due to gravity loads and dissipates only a little energy correspondingly. Therefore, seismic control using energy dissipation systems can be an efficient retrofit method. However, most researches on the design of the energy dissipation system focus on frame structures without wall or simplified shear building model, which show considerably different behavior compared to the shear wall-frame system. Therefore, researches on the shear-wall frame system retrofitted with energy dissipation systems are required.

Among various energy dissipation systems, the friction damper has simple and efficient energy dissipation mechanism and can be manufactured with low cost. In the design of the friction dampers installed in a multi-storey building structure, the friction force and supporting member stiffness are main design parameters and many researches on systematic design procedure have been conducted. Filiatrault and Cherry (1990) proposed a design procedure for equally-distributed friction dampers minimizing the sum of normalized displacements and dissipated energy through parametric study on the natural period, the frequency content of the ground motion and the friction force of damper. Fu and Cherry (2000) proposed a systematic design procedure of the friction dampers using force modification factors. Moreschi and Singh (2003) proposed an optimization procedure using a genetic algorithm that can overcome difficulty of the gradient-based optimization in dealing with nonlinear dampers such as friction dampers. Lee et al. (2007) verified superiority of friction force distribution proportional to the storey shear force through numerical analysis and proposed an equation to determine the optimal number of installation storeys. However, these studies are limited to framed structures or shear building models.

This study addresses control performance of the friction damper installed in a shear wall-frame system, of which main lateral force resisting system is the shear wall. A twelve storey building with the shear wall-frame system is designed according to the old Korean design code and evaluated for a ground motion based on the enhanced code. The shear wall of the building is modeled with MVLEM (multiple vertical line element model) and frame members are with fiber elements with inelastic material models. Then, according to the design factors, such as

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POLYNOMIAL FRICTION PENDULUM ISOLATORS (PFPI) FOR BUILDING STRUCTURES SUBJECTED TO NEAR-FAULT GROUND MOTIONS

Lyan-Ywan LU[1], Jain WANG[2], and Shih-Wei YEH[3]

SUMMARY

A building structure isolated by conventional passive isolation bearings is usually a long-period structural system with a constant isolation frequency. Even though a conventional isolation system may effectively mitigate the seismic response of a building structure in a regular earthquake, recent studies have also revealed that the conventional system may also suffer the problem of excessive isolator displacements when subjected to a near-fault ground motion that usually possesses a long-period pulse-like waveform. In order to overcome such a problem a new type of sliding isolation bearings called Polynomial Friction Pendulum Isolator (PFPI) is proposed and studied in this paper. Different from a conventional friction pendulum system (FPS), the geometry of the sliding surface of a PFPI isolator is defined by a sixth-order polynomial function. As a result, the stiffness of the PFPI has a softening behavior followed by a hardening behavior, when the isolator displacement increases. The softening and hardening behaviors aim to reduce the structural acceleration and isolator drift, respectively. In order to verify the feasibility of the PFPI concept, an element test on prototype PFPI isolators and a shaking table test on a structure isolated by PFPI were all conducted. The result of the shaking table test demonstrates that when subjected to an earthquake with a long-period pulse component, the PFPI system is able to effectively suppress the maximum isolator drift, without causing the amplification of the structural acceleration.

***Keywords**: seismic isolation, near-fault earthquake, sliding isolation, variable-frequency isolator*

INTRODUCTION

The seismic isolation using sliding bearings has been proven to be one of effective technologies for the protection of seismic structures (Yang et. al. 2005, Naeim & Kelly 1999, Lu et. al 1997). The friction pendulum system (FPS) is presently one of widely used sliding isolation systems (Mokha et. al. 1991). The sliding surface of a FPS isolator is usually made concaved and spherical, so that the gravitational load of the structure applied on the slider will provide a restoring stiffness that help reduce residual isolator displacement. However, this restoring stiffness, which is proportional to the curvature of the sliding surface, will inevitably introduce a constant isolation frequency to the isolated structure. Due to the existence of this constant frequency, a resonance-like problem may occur when the FPS is subjected to a ground excitation containing strong low-frequency components, such as a near-fault earthquake (Lu et. al. 2002). Recent studies have shown that when a structure isolated by conventional sliding isolators is subjected to a ground motion with near-fault characteristics, the base displacement may be considerably amplified due to the long-period pulse-like wave component possessed in most near-fault earthquakes (Makris & Chang 2000, Zayas & Low 2000, Jangid & Kelly 2001, Lu et. al. 2003, Lu et. al. 2004).

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BRIDGING NCREE-ISEEDB AND UI-SIMCOR FOR HYBRID EARTHQUAKE ENGINEERING SIMULATIONS ACROSS HETEROGENEOUS ENVIRONMENTS

Shang-Hsien HSIEH[1], Cheng-Tao YANG[2], Li-Xin LIN[3], Yuan-Sen YANG[4], and Keh-Chyuan TSAI[5]

SUMMARY

This paper presents a bridging approach that permits networked hybrid earthquake engineering simulations across two different Hybrid Simulation Environments, namely NCREE-ISEEdb and UI-SimCor. The approach allows the two different environments to share their software and hardware resources with each other for collaborative hybrid simulations through Internet. In this paper, the requirement analysis as well as the software design and implementation for realizing the proposed bridging approach are discussed. In addition, simulation examples are provided to validate and demonstrate the proposed bridging approach.

Keywords: *collaborative hybrid simulation, earthquake engineering, integration of heterogeneous simulation environments*

INTRODUCTION

As the scale and the complexity of modern earthquake engineering experiments increase, existing laboratories, often with limited resources (e.g., space and equipments), inevitably face difficulties to accommodate such experiments. To address this issue, the Hybrid Simulation (HS) approach is proposed and several HS Environments (HSEs), such as ISEE (Yang et al. 2007 and Wang et al., 2007), UI-SimCor (Kwon et al., 2005 and 2007), and OpenFresco (Takahashi and Fenves, 2006; Schellengerg and Mahin, 2006), have been developed recently. An HS divides a test structure into physical parts and analytical parts. Only the physical sub-structures need to be constructed and tested in the laboratory while the analytical parts are modeled and analyzed by computers. With the support of HSEs, the results of both the laboratory tests and analytical computations are integrated in HS to study the behavior of the test structure. Because HS takes advantage of modern structural analysis technology to simulate parts of the test structure with reasonable accuracy, the need for laboratory resources to study the behavior of the test structure can be greatly reduced. Furthermore, the aforementioned modern HSEs employ network technology to achieve collaborative hybrid simulations among two or more laboratories at different geographical locations. The sharing and integration of resources among laboratories further increase the capability of an HS to tackle large-scale and complex earthquake engineering experiments.

A Hybrid Simulation Environment (HSE) usually consists of the following major software modules: (a) a simulation coordinator for driving the HS process, (b) analysis engines for carrying out numerical analyses, and (c) facility controllers for manipulating various laboratory equipments (e.g., actuators). For an earthquake engineering laboratory to adopt an existing HSE, customization on the HSE's modules, especially the facility

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STRUCTURAL MONITORING SYSTEM FOR A BUILDING STRUCTURE USING WIRELESS MEMS

Hongjin KIM[1], Whajung KIM[2], Sang-Hyun LEE[3] and Jae-Seung HWANG[4]

SUMMARY

The structural health monitoring has been gaining more importance in civil engineering areas such earthquake and wind engineering. The use of health monitoring system can also provide tools for the validation of structural analytical model. However, only few structures such as historical buildings and some important long bridges have been instrumented with structural monitoring system due to high cost of installation, long and complicated installation of system wires. In this paper, the structural monitoring system based on cheap and wireless monitoring system is investigated. The use of advanced technology of micro-electro-mechanical system (MEMS) and wireless communication can reduce system cost and simplify the installation. Further, the application of wireless MEMS system can provide enhanced system functionality due to low noise densities. For the system identification, a FE model updating through correlating the analytical model of a structure to the measured one is used. The direct model updating method minimizes the variation of system matrices such as stiffness and mass matrices based on modal properties while satisfying appropriate constraints. Since the model updating in the direct model updating method is performed using measured natural frequency and mode shape, it has an advantage that the system identification is possible with limited information. The proposed method is evaluated experimentally using a three story frame model. Identification results are compared to ones using data measured from traditional accelerometers and results indicate that the system identification using wireless MEMS system estimates system parameters with quite accuracy.

Keywords: *MEMS; health monitoring; wireless monitoring; FE updating.*

INTRODUCTION

The structural health monitoring has been gaining more importance in civil engineering areas such earthquake and wind engineering. The use of health monitoring system can also provide tools for the validation of structural analytical model. However, only few structures such as historical buildings and some important long bridges have been instrumented with structural monitoring system due to high cost of installation, long and complicated installation of system wires (Lynch et al., 2003).

In this paper, the structural monitoring system based on cheap and wireless monitoring system is investigated. Recently, micro-electro-mechanical system (MEMS) became one of the most rapidly developing technologies for the structural health monitoring (Obadat et al., 2003, Staszewski et al., 2004, Zhang et al., 2005). MEMS is a small integrated device or system that combines electrical and mechanical components. It ranges in size from the sub micrometer level to the millimeter level. For MEMS, the term micro suggests a literally small system, electro suggests electricity and/or electronics, and mechanical suggests moving parts of some kind (Lin and

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SIMPLIFIED NONLINEAR STATIC AND DYNAMIC PROGRESSIVE COLLAPSE ANALYSIS OF WELDED STEEL MOMENT FRAMES

Cheol-Ho LEE[1], Seonwoong KIM[2] , and Kyungkoo LEE[3]

SUMMARY

Simplified methods for nonlinear static and dynamic progressive collapse analysis of welded steel moment frames were proposed in this study. A simplified tri-linear model for the vertical resistance versus chord rotation relationship of the column-removed double-span beams was first developed. The application of the developed model to the energy-based simplified nonlinear static progressive collapse analysis was then proposed. An equivalent moment versus rotation plastic hinge model which can consider axial-tension effect was also derived for a simplified and efficient nonlinear static/dynamic progressive collapse analysis using commercial codes. Finally, the proposed analysis methods were verified based on the high-fidelity inelastic dynamic finite element analysis.

Keywords*: progressive collapse; steel moment frames; catenary action; energy-based method; simplified model; equivalent plastic hinge; finite element analysis.*

INTRODUCTION

Partial and total progressive collapses of many buildings in the world have occurred under the abnormal extreme loading conditions due to fires, impact, and blast during past several decades. Interest has increased because building collapse is a quite rare event but results in significant casualties and property loss when it occurs. Especially, the collapse of the World Trade Center by 911 terrorist attack in 2001 triggered a big interest in blast and progressive collapse resistant design (for example, Guo and Gilsanz 2003; Longinow and Alfawakhiri 2003; Hamburger and Whittaker 2004).

To prevent the progressive collapse of structures, an initial localized damage or local failure should not spread out from element to element, which may result in failure of the entire structural system disproportionate to the original cause. The most viable approach to limit the propagation of localized damage is to maintain the integrity and ductility of structural system. The commentary in ASCE 7-05 (2005) suggests general design guidance for improving progressive collapse resistance of structures, but provides no specific implementation rules. Recent design approaches to mitigate progressive collapse potential in structures can be found in both documents issued by the U.S. General Services Administration (GSA 2003) and the Department of Defense (DoD 2005). The direct or explicit approach is an alternate path (AP) method. In this method, a single column is typically assumed to be missing and analysis is conducted to determine whether or not the structure can bridge across the missing column.

Figure 1 shows progression of the load transfer mechanism in a column missing event. In the initial stage of the event, the flexural capacity of beams, which are on both sides of the removed column, resists vertical loads (see Figure 1 (a)). As undergoing significant inelastic rotations and large deflections, the beams start to act more like

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HYSTERETIC BEHAVIOR OF CONCENTRICALLY BRACED FRAMES BASED ON THE REVIEW OF EXISTING EXPERIMENTAL DATA

Kangmin LEE[1] and Michel BRUNEAU[2]

SUMMARY

Design and detailing requirements of seismic provisions for Concentrically Braced Frames (CBF) were specified based on the premise that bracing members with low KL/r and b/t will have superior seismic performance. However, relatively few tests investigate the cyclic behavior of CBF. It is legitimate to question whether the compression member of CBF plays as significant a role as what has been typically assumed explicitly by the design provisions. In this paper, the existing experimental data is reviewed to quantify the extent of hysteretic energy achieved by bracing members in compression in past tests, and the extent of degradation of the compression force upon repeated cycling loading. While it is recognized that many parameters have an influence on the behavior of braced frames, the focus of this paper is mostly on quantifying energy dissipation in compression and its effectiveness on seismic performance. Based on the experimental data review from previous tests, it is found that the normalized energy dissipation of braces having moderate KL/r (80 - 120) do not have significantly more normalized energy dissipation in compression than those having a slenderness in excess of 120. The normalized degradation of the compression force envelope depends on KL/r and is particularly severe for W-shape braces.

***Keywords**: Concentrically Braced Frames; Effective Slenderness Ratios; Energy Dissipation; Compression Members; Strength Degradation; Braces.*

INTRODUCTION

Seismic provisions for the analysis, design, and detailing of Concentrically Braced Frames (CBF) were gradually introduced into seismic regulations and guidelines in California in the late 1970's (SEAOC, 1978) and on a nationwide basis in the early 1990's (AISC, 1992). In these documents, design and detailing requirements were specified based on the premise that bracing members with low effective slenderness ratio, KL/r, and low width-thickness ratio (low local buckling slenderness ratio), b/t, will have superior seismic performance. The philosophy was that low KL/r ensures that braces in compression can significantly contribute to energy dissipation. Upon buckling, flexure develops in the compression member and a plastic hinge eventually develops at the middle length of the brace. It is through the development of this plastic hinging that a member in compression can dissipate energy during earthquakes. Furthermore, in these code provisions, low b/t limits were prescribed to prevent brittle failure due to local buckling. Indeed, the reversed cyclic loading induced by earthquakes leads to repeated buckling and straightening of the material at the local buckling location, which combined with very high strains present at the tip of the local buckle, precipitate low cycle fatigue.

Although much attention has been paid to Moment Resisting Frames (MRF) after the 1994 Northridge earthquake, with a large number of tests conducted since, relatively fewer tests exist that investigate the cyclic behavior of CBF. This is surprising given the reliance imposed on compression brace energy dissipation (complementary to the primary energy dissipation in tension) by the existing codes and guidelines.

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CYCLIC TESTING OF STEEL BEAM-TO-COLUMN CONNECTIONS WITH SUPPORTING MEMBERS

Cheng-Chih CHEN[1], Jen-Fu LO[2], Yuh-Chyurn DING[3], Lo MAO[4], and Chi-Chun TAO[5]

SUMMARY

This paper presents an experimental investigation of moment connections supported by an additive member to move the maximum flexural moment of the beam away from the beam-column interface, and to reduce the moment demand at the beam-column interface. Theoretical study shows that the yielding mechanism of the beam depends on the elastic stiffness of the supporting member. The beam can form plastic hinge on the beam at the supporting location or develop shear yielding within the supporting span of the beam. Full-scale tests confirmed the behavior, and demonstrated that the proposed connections can diminish the potential brittle failure in the beam-column interface and develop satisfactory hysteresis behavior.

Keywords: *beam-to-column connection, supporting member, plastic hinge, hysteresis.*

INTRODUCTION

Steel moment-resisting frames (MRFs) have been widely used to resist earthquake-induced forces because of their ductile behavior. MRFs are designed to remain elastic during small to medium earthquakes, but the frames are expected to develop inelastic, ductile behavior when subjected to a large earthquake. However, numerous beam-to-column moment connections in steel moment-resisting frames were damaged with limited inelastic behavior during the 1994 Northridge and 1995 Hyogoken-Nanbu earthquakes (Miller 1998; Nakashima et al. 1998). To improve the cyclic behavior of the moment connections, research conducted after the earthquakes emphasized on either reinforcing the beam-column interface or weakening the beam section, and both improvements led to the formation of the plastic hinge in the beam section away from the beam-column interface. Reinforced connections can be achieved by using cover plate, wing plate, vertical rib, and haunch (Engelhardt and Sabol 1998; Chen et al. 2004; Uang et al. 1998). Reduced beam section (RBS) has been verified to be an effective improvement for the moment connection and has been widely used in the US (Chen et al. 1996; Engelhardt et al. 1998).

Rather than forming the plastic hinge at the beam-column interface, the post-Northridge connections were improved to develop the plastic hinge on the beam section away from the interface to assure reliable, stable plastic deformation capacity. Nevertheless, for both RBS and reinforced connections, the beam-column interface still has the maximum bending moment when the moment-resisting frames are subjected to the lateral force exerted from the seismic excitation. In this paper, an intention to reduce the moment demand at the beam-column interface is proposed by adding supporting members to enhance the seismic performance of the moment connection. Based on the additional stiffness provided by the supporting member, the maximum bending moment may occur away from the beam-column interface. By varying the elastic stiffness provided by the supporting member, the beam may either develop flexural plastic strength or reach shear yielding capacity. Full-scale experiment was carried out to verify the cyclic behavior of the moment connections with the

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NUMERICAL STUDY ON COLLAPSE RESISTANCE OF WELDED STEEL MOMENT CONNECTIONS

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SUMMARY

Since the world trade center was collapsed by the terrorist attack in 2001, a progressive collapse of high-rise buildings has drawn researchers and engineers' attentions. To prevent this progressive collapse, the structural system should be robust, redundant, and ductile. For steel structures, the performance of the system mostly relies on the effectiveness of connection elements. For this demand, ductile seismic connections tested by SAC/FEMA could be a candidate containing the properties. However, they lack resistance to consequences of removal of a column member, which are beam axial tension and moment interaction or beam tension through a catenary action, since they were cyclically tested to evaluate rotational capacities and ductility only for the seismic design. This study extensively discussed the collapse resistant performance of welded beam-column connections in steel moment frames. Three Post-Northridge connections, WUF-W, RBS, and WCPF were selected as analysis models to understand their global and local behaviors when their supporting columns were suddenly removed. In this case, the catenary action was included in the analysis model. For the purpose of comparison, additional analyses using identical models were conducted in which the catenary action was not considered. The behavior of the welded moment connection including the catenary action was found to be quite different from that of the same connection without the catenary action.

Keywords: *progressive collapse; connections; fracture; local buckling; catenary action.*

INTRODUCTION

Welded steel moment-resisting frames (WSMFs) are being widely used for a major lateral force resisting structural system in high seismic regions. WSMF had been recognized as an excellent seismic resisting system, but it was revealed that a vital damage could happen in beam to column joint regions of the WSMFs after the 1994 Northridge earthquake in the U.S. and 1995 Kobe earthquake in Japan. Since then, reliability analyses for the WSMFs have been extensively conducted. As a result, SAC Joint Venture was launched and funded by the Federal Emergency Management Agency, where analytical and experimental studies were conducted for investigating performance of old and new connection types. The results of the SAC research were summarized in series of the FEMA reports and are providing structural engineers with valuable information about design of the beam to column connection in the WSMFs (FEMA 2000a, b). In the SAC, new seismic connections have been developed moving plastic hinge locations from critical regions (i.e., beam to column joint regions) to safe regions (i.e., beam flange far from the critical region). These seismic connections have been prequalified by full scale tests so they are generally being used in WSMFs as a major lateral load (or seismic load) resisting element.

Meanwhile, the collapse of the World Trade Center in 2001 from terrorist attack brought about big attention to the structural design for the safety of buildings against abnormal loading such as not only earthquakes but also explosions or fires. Therefore, many researchers are currently studying progressive collapse and protection from those abnormal loading. Among various topics for collapse resistance issues, beam to column connections in

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CYCLIC TESTING OF POST-TENSIONED CONNECTIONS INCLUDING EFFECTS OF A COMPOSITE SLAB

Chung-Che CHOU[1], Yu-Chi WANG[2], and Jun-Hen CHEN[2]

SUMMARY

Three series of six full-scale, cyclic tests on post-tensioned (PT) beam-to-column connections were conducted to evaluate cyclic performance. Each specimen represented an interior connection with steel beams PT to a column. Reduced Flange Plates (RFPs) were added at the beam-to-column interface for energy dissipation, and flange reinforcing plates were provided outside the beam flanges to minimize yielding upon decompression. The objectives in this study investigated durability of the proposed connections with improved details, effects of beam web stiffeners in preventing beam from buckling, and interaction of a composite slab. The first test series demonstrated that as long as beam yielding could be prevented before an interstory drift of 4%, the PT connection was capable of reaching an interstory drift of 5% or experiencing the AISC seismic loads twice without beam buckling. The second test series showed that beam buckling could be prevented by utilizing web stiffeners instead of increasing the length of the flange reinforcing plate. The third test series included a matched pair of specimens, one bare steel beam and one including a composite slab. Results that emphasize the influence of the composite slab on re-centering behavior and specific comments on the slab response were presented. The presence of a composite slab corresponded to higher-achieved moments due to tensile capacity of the metal deck flute, wire mesh, and longitudinal reinforcement, which were placed parallel to the beam. However, the re-centering behavior could be maintained after fractures of the wire mesh. Estimates of beam moments with presence of a composite slab were presented based on test results.

Keywords: *Post-tensioned connection, Strands, RFP, Composite slab, Cyclic test*

INTRODUCTION

A connection with steel beams PT to a column is intended to exclude field welding at the beam-to-column interface and limit damage to replaceable energy-dissipating devices. Chou et al. (2006) experimentally and analytically investigated three connection subassemblies with steel beams PT to a column. Because the length of the flange reinforcing plate was determined based on the axial force and moment demands in the beam at the gap opening angle of 0.02 rad., beam buckling occurred at an interstory drift of 4%, leading to a loss of re-centering response. Chou et al. (2005) also tested one PT connection specimen with a composite slab. The composite slab, following steel construction practice in Taiwan, had metal deck flutes, longitudinal reinforcement, and welded wire mesh placed parallel to the beam. For the beam under negative bending, the test results showed significant residual deformation and no re-centering response due to tensile capacity of the slab. Therefore, the objectives of this research (Wang and Chou 2006) investigated (1) durability of the PT connection designed based on the beam force and moment demands at the gap opening angle of 0.03 rad., (2) whether beam local buckling could be prevented by adding web stiffeners near termination of the flange reinforcing plate, and (3) interaction of a composite slab with improved slab details.

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SHAKING TABLE TESTS FOR MASONRY WALLS IN RC FRAMES

Yi-Hsuan TU[1] , Pai-Mei LIU[2] , Tsung-Hua CHUANG[3]

SUMMARY

In Taiwan, confined masonry is commonly used for in-filled walls in low-rise RC buildings. Although the un-reinforced masonry (URM) walls displayed well in-plane strength in the past earthquakes, they are also fragile when subjected to out-of-plane loading.

In order to study the out-of-plane failing behavior of un-reinforced confined masonry walls, a series of single-axis shaking table tests had been taken. Four full-scaled one-story RC frames consist of no wall, half-brick thick pre-laid, half-brick thick post-laid, and one-brick thick pre-laid URM in-filled walls, respectively, were designed to investigate the effect of thickness and construction methods on the out-of-plane strength. A time-history record of Chi-Chi earthquake is chosen to excite the specimen while the intensity is gradually enhanced each excitation.

For all the URM walls, horizontal cracks occurred at the top and the bottom edges almost as soon as the tests began. The post-laid walls began to split from the RC frame during the later stages of the loading procedure and eventually collapsed in pieces. Contrarily, the pre-laid walls remained in the frames throughout the entire test. The test results suggest that increasing thickness of URM walls usually leads to slightly higher stiffness. However, there is no apparent relationship between thickness and out-of-plane strength. Generally, specimens with pre-laid walls show higher strength and stiffness than the one with post-laid wall. All the specimens with URM in-filled walls show higher strength than the one without walls. Therefore, the possible out-of-plane contribution of URM walls is confirmed in this test.

Keywords*: Un-reinforced Masonry; Shaking Table Test; Out-of-Plane*

INTRODUCTION

In Taiwan, un-reinforced masonry (URM) walls are the most common type of in-filled walls for low-rise RC buildings due to their low cost and ease of construction. At 1970s and 1980s, most of them were built as “confined masonry”, which consists of pre-laid URM brick walls and post-constructed reinforced concrete (RC) boundary frames. It is believed that the RC boundary frames provide confinement to the walls because of shrinkage of concrete. Usually the URM walls in confined masonry are 1-brick thick (22~24cm) since they are expected to provide in-plane seismic capacity. However, a confined masonry building is not allowed to exceed 3 stories or 10m high by the Taiwan Building Code. Therefore after 1990s, post-laid URM partition walls in pre-constructed RC buildings became more popular. Although the post-laid URM walls are usually half-brick thick (11~12cm) since they are not

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FINITE ELEMENT MODEL UPDATING OF SMALL SCALE REINFORCED CONCRETE WALL BUIDING USING SHAKING TABLE TEST RESULTS

Ho-Geun KIM[1], Eunjong YU[2], Sang-Hyun LEE[3], Lan CHUNG[4] and Seung-Ho CHO[5]

SUMMARY

This paper focused on the application of new finite element model updating technique to evaluate the change of structural properties of a 1/5-scale, five-story, one-bay reinforced concrete bearing wall building specimens using the data collected from shaking table tests. The specimen was subjected to six El Centro 1942 ground motion histories with different Peak Ground Acceleration (PGA) ranging from 0.06g to 0.5g. For model updating, flexural stiffness values of the structural members (walls and slabs at each story level) were chosen as the updating parameters. Frequency response functions (i.e. transfer functions) and natural frequencies obtained using the acceleration measurement at each floor and given ground acceleration history were used as the input for the model-updating problem. To address numerical difficulties associated with ill-conditioning, two type of constraint were added in the solution of least-squares problem. At each stage of shaking, severity of damage that results from cracking of the reinforced concrete member was quantified from the updated flexural stiffness values. Distributions of obtained flexural stiffness showed that larger stiffness degradation occurred at the slab members than the wall members, which is consistent with the observed cracks pattern of the test specimen.

***Keywords**: Finite element mode; Model Updating; Ill-conditioning; Bounded Variables Least-Squares; Building structures; Effective Flexural Stiffness Factor.*

INTRODUCTION

The dynamic structural response obtained from vibration measurements may show significant discrepancies with the responses estimated using an idealized parametric structural model. “Finite Element (FE) model updating” is an analytical procedure used to minimize this discrepancy by adjusting, for example, the stiffness, mass, and/or damping parameters of the discrete numerical model. As such, model updating techniques combined with vibration measurement are useful tools for assessing the health or damage of a structure.

A plethora of model updating techniques had been proposed in literature [Mottershead and Friswell, 1995]. Depending on the objectives of updating, some methods update the coefficients of system (i.e., mass, stiffness, damping) matrices, while others update the structural parameters comprising these coefficients. Between the two approaches, the methods that update the structural parameters by iteratively solving sensitivity-based updating equations are the most common. This is because such methods have the advantage of being based on updating parameters that are directly related to the physical attributes of the structure. However, independent of the approach used, physically admissible solutions may not be attained and the solutions may yield erroneous or physically meaningless results depending on the selection of updating parameters. This phenomenon is caused by the “rank deficiency” or “ill conditioning” of the sensitivity matrix used in the updating equations. Rank deficiency occurs when information is insufficient to yield a unique solution to the model updating problem,

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SEISMIC PERFORMANCE OF HYBRID SYSTEM WITH CORRUGATED STEEL SHEAR PANEL AND RC FRAME

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ABSTRACT

This research aims to propose an economical seismic response controlling system of RC frames using corrugated steel shear panels. This hybrid system was originally proposed by Mo and Perng in 2000. Their experimental results showed the poor seismic performance due to large relative displacement at the interface between surrounding RC members and a corrugated steel shear panel, and hence the obtained pinched hysteresis loops dissipated small amount of energy. Their test results made this hybrid system inappropriate to use in practical building design although great potential of the system was demonstrated. In this study, the hybrid system was revised to prove that it is excellent to resist the seismic force. In an experimental phase a stud-type anchorage often used in bridge box girders was employed in four half-scale specimens to fix the corrugated shear panels, and then static cyclic loading was applied to the specimens. All hybrid frames showed more than 30% increase in lateral load carrying capacity as originally designed. The degradation of lateral load carrying capacity after the peak load was small compared to reinforced concrete shear walls due to stable manner of yielding and buckling of corrugated steel shear panels. The final failure mode of the hybrid system was the tearing of the corrugated steel shear panel and the formation of a collapse mechanism of the surrounding reinforced concrete frame. In an analytical phase, a nonlinear frame analysis was conducted to evaluate the effect of corrugated steel shear panel on the performance of the reinforced concrete frame. The analysis simulated the behavior of the specimen with sufficient fixity of the shear panel but the specimens with insufficient anchorage had some room for improvement.

***Keywords**: corrugated steel shear panel; damage control; shear wall.*

INTRODUCTION

It is a common practice to use reinforced concrete shear walls in reinforced concrete structures to maintain high lateral load carrying capacity and stiffness. However, high lateral stiffness with brittle ultimate failure mode of RC shear walls often increases the required lateral load carrying capacity. In order to improve the ductility of reinforced concrete shear walls, some efforts have been made such as using low yield strength reinforcement and introducing slits but the ductility enhancement was not very prominent. Use of steel shear walls in order to increase ductility has some decades of research history. In 1973, Takahashi et al. [1] studied the characteristics of load-deflection relations of steel shear walls obtained experimentally and reported the effects of configuration, width-thickness ratio, stiffeners' stiffness, etc. on the load-deflection relations. Studies on steel shear walls have been continued since then [2, 3]. However, flat steel shear panels need stiffeners to prevent plate buckling,

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STRESS TRANSFERRING MECHANISM OF INTERIOR BEAM-COLUMN JOINTS FOR COMPOSITE *CES* STRUCTURAL SYSTEM

Tomoya MATSUI[1] and Hiroshi KURAMOTO[2]

SUMMARY

Three-dimensional non-linear FEM analyses of Concrete Encased Steel (CES) beam-column joints, which are composite structural systems consisting of encased steel and fiber reinforced concrete, were conducted to examine the stress transferring mechanism for CES beam-column joints, together with verifying applicability of the analytical method for a CES structural system. The analytical results showed good agreement with the experimental ones on the story shear versus story drift relationship for CES beam-column joints, it is found that the behavior of CES beam-column joints can be approximately simulated by the analytical method. It is confirmed that the consequent compressive strut of concrete was formed in the outer concrete without steel as well as in the inner concrete surrounded by the steel flange of the beam and column in this analysis, it is shown that analyzed shear force in the outer concrete are contributed as almost the same level as those in the inner concrete at the maximum capacity of the beam-column joint. Furthermore, It is shown that the joint shear strength of interior CES beam-column joints could be evaluated by a method based on the AIJ design standard for SRC structures using the above sizes.

***Keywords**: Composite structure; CES beam-column joints; Fiber reinforced concrete; Three dimensional FEM analysis; Shear force distribution; Shear capacity magnification factors.*

INTRODUCTION

Steel Reinforced Concrete (SRC) Structures developed in Japan have good structural performance for resisting lateral forces imposed by wind and earthquakes, and have been adopted for medium-rise, high-rise, and super high-rise buildings. However, the number SRC of structures constructed has decreased since the 1990s. The decrease in the number of constructions has been caused by the development of a new structural engineering system called the High-strength concrete structure or Concrete-Filled Steel Tube (CFT) structure, but the main reason thought for the causes of the decrease is the construction problems that increase construction costs and lengthen construction schedules. Even so, it could be important that SRC structures provide better seismic performance in comparison to other structural systems. So, the authors aim to develop a structural system with as good seismic performance as SRC structures and good workability, and have conducted a continuing development study on composite Concrete Encased Steel (CES) structures composed of steel and fiber reinforced concrete (FRC).

In an experimental study on CES columns using High Performance Fiber Reinforced Cement Composite (HPFRCC) conducted as a feasibility study, it was confirmed that damages due to cracks and compressive failure were reduced under a large drift angle by using HPFRCC, and the restoring force characteristics of CES columns was almost equal to that of SRC columns. However, because HPFRCC is mortar, problems of initial stiffness reduction and shrinkage appeared. To improve these issues, FRC has been used for concrete in the studies that followed. The experimental results for CES columns using FRC showed that performance to restrain crack damage was slightly less than columns using HPFRCC, while the CES columns exhibited stable restoring

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INVESTIGATION AND EXAMINATION OF DAMAGE CONDITIONS OF AN OLD REINFORCED CONCRETE SCHOOL BUILDING DUE TO SELF-STRAIN STRESSES

Masaru TERAOKA[1] and Yasuhiro FUKUHARA[2]

SUMMARY

This paper describes the investigation and examination of damage conditions of a reinforced concrete school building with three stories which was constructed on a soft stratum in 1966. In this study, the self-strain stresses due to the differential settlement and concrete drying shrinkage were evaluated with the related damage, and the influence of such damage on the structural performance was discussed.

The following findings were obtained from this study:

(1) A part of the structural frames has been reached to yield conditions due to remarkable differential settlement and, as a result, the apparent rigidity of the structure was estimated as to be about 1/3 of the elastic rigidity.

(2) The magnitude of differential settlement was significantly influenced by the structural rigidity as well as the soil condition. This was confirmed by the theoretical values of the differential settlement which was almost corresponded to the measured one.

(3) Based on the measured concrete drying shrinkage strain, the average strain of main beams in the statically indeterminate rigid frames was estimated as about 150×10^{-6}, and that of each of the slab and the sub-beam was to be about 200×10^{-6}.

Keywords: self-strain stress; reinforced concrete structure; soft subsoil; concrete drying shrinkage; differential settlement

INTRODUCTION

In the coast area of Japanese cities, a lot of reinforced concrete structures were constructed on the reclaimed ground with soft subsoil during the rapid-economical-growth era around 1960s. In these structures, the damages such as concrete cracks, yielding of members, et al. were caused mainly by the self-strain stresses. The self-strain stresses are induced by concrete drying shrinkage, a differential settlement due to subsidence by consolidation of the clay layer, and expansion/contraction of concrete due to the temperature fluctuation 〔1〕,〔2〕.

Regarding these damage conditions in reinforced concrete structure, the evaluation method of the differential settlements, where the rigidity of superstructure was considered, was developed in the field of geotechnical engineering. In the studies of such field, the relation between the crack generation of outside wall and deflection angle of members in the frame due to a total settlement or a relative settlement was also examined〔1〕,〔2〕. However, the influence of various self-strain stresses mentioned above was not analyzed.

On the other hand, the examination of damages accompanied by the differential settlement has not been conducted in the field of the concrete technology and the structural study of reinforced concrete. But in that field, the evaluation studies of the damage of the reinforced concrete structure due to strain of concrete at hardening and drying and the temperature stress has been consecutively conducted since prewar era〔3〕,〔4〕. It is noted that the influence of the self-strain stresses is not taken into account when determining the aging index adopted in the Japanese earthquake-proof diagnosis method for existing buildings. Although the self-strain stress problem can be recognized as an important factor of the aging index, the research on that subject is hardly seen〔5〕.

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