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HYSTERETIC CHARACTERISTICS OF PRECAST PRESTRESSED CANTILEVER BEAMS WITH GRADED COMPOSITE STRANDS

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SUMMARY

Precast prestressed (PCaPC) structures have high restoring force and their damage concentrate at gaps between members. Therefore, they require very little repair after earthquakes. However, they dissipate small energy and lateral displacements tend to be larger than those of RC structures under seismic loadings. In order to enhance the energy dissipation of PCaPC structures, Graded Composite Strand (GCS) has been proposed. In this study, new types of GCS’s were developed and five PCaPC cantilever beams with new GCS’s were tested. A specimen with GCS-H, which consists of ultra high strength wires and low strength wires, had increased equivalent damping factor with keeping residual deformations small. Low strength wires of GCS-H dissipated energy by yielding, and ultra high strength wires remained elastic and provided restoring force. All specimens had small damage and structures with GCS’s would need almost no repair. The ultimate flexural moment capacity was calculated accurately by considering bond deterioration of strand.

***Keywords**: Graded composite strand; Precast prestressed structure; High energy dissipation; Residual deformation; Flag shape hysteresis loop.*

INTRODUCTION

Precast prestressed structures show nonlinear elastic hysteresis and very small residual deformation. However, they dissipate small amount of energy and tend to deform more than ordinary reinforced concrete structures under seismic loading. In order to solve this problem, the Graded Composite Strand (GCS) was proposed by Muguruma et al. in 1993. Cantilever specimens with GCS's were tested by Niwa et al. and it was confirmed that they increased the amount of energy dissipation by yielding of low strength wires and kept residual deformation small by restoring force of high strength wires. However, when the prestressing force was high, some wires fractured.

In this study, to prevent the fracture and enhance the energy dissipating capacity of GCS, three kinds of new GCS's were proposed. They were GCS-U, GCS-H3L4, and GCS-H1L6. GCS-U has one ordinary strength seven-wire strand surrounded by twelve low strength wires. GCS-H3L4 and GCS-H1L6 have the same shapes as ordinary seven-wire strand. GCS-H3L4 consists of three high strength wires and four low strength wires, and GCS-H1L6 consists of one high strength wire and six low strength wires. Precast prestressed cantilever beams with these strands were tested to confirm that they improved energy dissipating capacity without increasing residual deformation and wires didn’t fracture.

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EXPERIMENTAL AND ANALYTICAL VERIFICATION ON DESIGN APPROACHES OF RC TRANSFER GIRDERS HAVING SHORT END SHEAR SPAN

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SUMMARY

The transfer girder which should accommodate the upper wall and lower columns may have very short end shear span between the wall boundary and the face of the lower column. Two available design procedures for this type of transfer girders, conventional and strut-and-tie procedures, are performed, and the details based on these design methods are compared with regard to the dimensions and amount of reinforcements. This paper presents the results of tests and analyses performed on two specimens of transfer girders designed by different design approaches.

The conclusions are as follows: (1) The approach of the strut-and-tie method is valid for the type of transfer girders rather than conventional beam approach. (2) Since the upper load was carried over directly to the supporting columns through the stiff concrete strut without any assistance from the adjacent beam up to the point of the yielding of the bottom ties, it is reasonable to model the continuous deep beam as a group of independent simple deep beams up to yielding of bottom tie. (3) The shear capacity of deep transfer girders are mainly governed by the bottom tie yielding forces. The additional shear resistance comes from the continuity with the adjacent beams or walls. Shear and top reinforcements in the continuity region can be designed using the strut-and-tie model for the additional margin of safety regarding strength and ductility. (4) Results simulated from nonlinear 2-dimensional analyses using DIANA showed the behaviors similar to experimental results in the global sense, but could not simulate exactly the strain distribution and detailed crack development.

***Keywords**: Reinforced Concrete, Transfer girder, Strut-and-tie method, DIANA*

INTRODUCTION

The most common structural system for multi-purpose buildings in Korea has been a moment-resisting frame for the lower stories and a bearing-wall system for the upper stories. The lower stories usually accommodate parking areas, commercial spaces, gardens, or open spaces for architectural reasons, while the higher stories are generally used for apartments. In this kind of building structures, the transfer girders transfer the load from the upper bearing wall to the lower frame. Transfer girders are generally deep in depth and the principle of Bernoulli (a plane remains a plane after deformation) does not apply. Nevertheless, the practicing structural engineers model this transfer girder as a beam and design the girder using the conventional procedure for beam design. This convention not only causes uneconomy and difficulty in construction due to excessive depth and congested reinforcement, but also the resulting design does not necessarily ensure the safety and efficiency in structural behavior. Recently, the method of strut-and-tie model appears to be an alternative, but still lacks a sound logic in the approach though ACI 318-05 (ACI 2005) proposed a recommendation.

A 17-story reinforced concrete structure was selected as the prototype based on an inventory study of multi-purpose buildings in Korea(Lee et al. 1999) A transfer girder in this prototype was designed using the conventional beam method and the strut-and-tie method. The design details are compared. Experiments were conducted to investigate the behavior of specimens designed according to the two methods. Two design

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SHEAR STRENGTH AND DEFORMATION CAPACITY OF REINFORCED CONCRETE BEAMS

Hong-Gun PARK[1], Kyung-Kyu CHOI[2], and Chong-chul PARK[3]

SUMMARY

A theoretical model was developed to predict the shear strength and deformation capacity of slender reinforced concrete beams. The shear force applied to a cross-section of the beam was assumed to be resisted primarily by the compression zone of intact concrete rather than by the tension zone. The shear capacity of the cross-section was defined based on the material failure criteria of concrete. In the evaluation of the shear capacity, interaction with the normal stresses developed by the flexural moment in the cross-section was considered. Because the magnitude and distribution of the normal stresses vary due to the flexural deformation of the beam, the shear capacity of the beam was defined as a function of the flexural deformation. The shear strength of the beam was determined at the intersection between the shear capacity and the shear demand curves. The proposed strength model was verified by comparisons to prior test results.

***Keywords**: shear strength; slender beam; deformation; failure criteria*

INTRODUCTION

The shear strength of slender reinforced concrete beams ($a/d > 2.5$) is affected by various design parameters. According to existing test results, the primary parameters that affect the shear strengths of concrete beams are the compressive and the tensile strengths of concrete, the ratio of flexural reinforcement, shear span to depth ratio, and the size of the beam. Based on the test results, current design codes of ACI 318-02 (2002) and BS 8110 (1997), and many researchers including Zsutty (1971) and Okamura and Higai (1980), proposed various design equations. ACI 318-02 overestimates the shear strengths of beams made of high strength concrete. On the other hand, Okamura and Higai's equation accurately evaluates the shear strengths of beams. However, since these design equations were developed empirically, based on the test results for simply supported beams, their applicability to beams with different loading and boundary conditions needs to be examined.

As for theoretical models, Bažant and Sun (1987) developed a design equation based on fracture mechanics. Nielsen (1998), and Zararis and Papadakis (2001) developed strength models based on the theories of plasticity and elasticity, respectively. Al-Nahlawi and Wight (1992) proposed an improved strut-and-tie model, introducing concrete-tension ties. Unlike other empirical models, these models are based on firm theoretical backgrounds. Most of these theoretical strength models used the distribution and magnitude of the internal forces and stresses based on the force-equilibrium or the energy principle at failure. However, the distribution and magnitude of the internal forces and stresses vary significantly according to the flexural deformation of the beam. Therefore, to accurately evaluate the shear strength of a beam, the effect of the deformation on force-distribution and -magnitude must be addressed.

In the present study, a shear strength model that addresses the effect of flexural deformation of a beam was developed. The proposed model was used to evaluate the shear strength and deformation capacity of slender beams ($a/d > 2.5$).

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FLEXURAL SHEAR DEFORMABILITY OF STRUCTURAL CONCRETE

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SUMMARY

Deformability of structural concrete is mostly controlled by governing failure modes and ultimate strain of concrete in compression. This paper investigates flexural shear deformability of shear critical structural concrete members showing limited ductility. Stress fields describing the potential maximum shear strength relying arch action and truss action need to be refined to consider the lower shear force determined by flexural yielding. Decomposition of total shear strength into shear via direct transfer by diagonal strut and indirect transfer by transverse reinforcement is necessary to determine finite dimensions of struts in the stress fields for shear force after flexural failure. Appropriate Failure mechanisms compatible with stress fields at ultimate are investigated to estimate deformability controlled by crushing of struts of finite dimensions. Proposed procedures for deformability are capable of explaining of limited ductility of other structural concrete in D-regions dominated by shear.

Keywords: *shear-dominant members; stress-fields; strut-and-tie models; ultimate deformation; deep beams; coupling beams.*

INTRODUCTION

Ductility of reinforced concrete members mostly depends on ultimate strain of concrete in compression. For ductile behavior of systems seismic design practices rely on formation of plastic hinge at an intended region of flexural members. To guarantee and increase deformability of concrete in compression appropriate reinforcement details for confinement have been focused on. Most design provisions are supposed to suppress premature shear failure before ductile flexural failure as much as possible. However, decrease in shear strength dependent on deformation cannot be avoided such as in deep coupling beams and beam-column joints even after flexural yielding for stable energy dissipation. Since most of ductility of such members and components show much smaller than those of flexural members so called limited ductility concepts have been introduced for flexural shear failure of columns and shear strength of walls of low aspect ratios.

Current strut-and-tie models have served rational design tools for dimensioning and detailing of reinforced concrete members in disturbed regions. Truss-like behavior of RC in disturbed regions at ultimate has advocated practical use of STM. Conventional strut-and-tie models have served tools of estimation of strength only. To extend the application of STM to the estimation of deformation some of force-deformation relationships for ties and struts have been suggested. STM consisting of such components is capable of estimating strength and deformations. However, there have been still task to determine realistic dimension of struts with nodal zones at different load levels. Also we need to set up criteria to limit the ultimate deformation of STM. There have been still question why ductility by flexural failure show larger ductile behavior than shear relying on diagonal struts in case of shear failure even though ultimate deformation in both failure modes are controlled by ultimate strain of concrete in compression. Deformation limits of disturbed region or limited ductility for shear strength controlled members need rational models for explanation.

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TORSIONAL STRENGTH OF REINFORCED CONCRETE BEAMS CONSIDERING TENSION STIFFENING EFFECT

Jung-Yoon LEE[1] Ji-Hyun Kim[2] and Hyun-Bok Hwang[2]

SUMMARY

The current ACI design code (ACI 318-05) for the torsional design of reinforced concrete (RC) members is based on the space truss analogy and the thin walled tube theory. This design code simplified the previous design code (ACI 318-71) and it is not directly take into account the contribution of concrete. Though the torsional equation of the ACI 318-05 predicted the test results of 180 RC members with reasonable accuracy, some test results indicated that the current code was not successful in evaluating the interactions between concrete and torsional reinforcement in the torsional resistance of the RC members. The research reported in this paper provides an evaluation equation to calculate the torsional moments of the RC beams subjected to pure torsion. The proposed equation was derived from the equilibrium as well as compatibility equations of the truss model for cracked RC beams. Comparisons between the observed and calculated torsional moments of the 66 tested beams showed reasonable agreement.

***Keywords**: torsional strength; tension stiffening effect; space truss model; reinforced concrete members.*

INTRODUCTION

The torsional strength of reinforced concrete(RC) members as specified by the ACI 318-71 Code consists of two parts. The first part is the contribution of concrete (T_c) which is based on the Portland Cement Association (PCA) tests. The second part is the contribution of torsional reinforcement (T_s) using a well-known space truss analogy. The torsional design provisions in the 1971 Code were changed in the 1995 ACI Building Code. The new torsional design provisions are based on a thin-walled tube and space truss analogy using the equilibrium of forces. Comparing the 1995 provisions to the previous ones, there are three main differences:

The first, the provision of the contribution of concrete (T_c) was not directly considered,
The second, the angle (α) of inclination between the diagonal compressive stresses and the longitudinal axis of a member varies according to the ratio of the content of the longitudinal reinforcement to the content of the transverse reinforcement, while α was assumed as 45° in the 1971 Code, and
The third, the gross area (A_o) enclosed by the centerline of the shear flow path is taken as 0.85 times the area (A_{oh}) enclosed by the outmost closed stirrups in the section.

Though the revised torsional design equation in the 1995 Code is a simple equation in calculating the torsional moment because it is proportional to the amount of torsional reinforcement, some test results (Fang and Shiau, 2004) indicated that this equation under-evaluated or over-evaluated the torsional moments of RC beams depending on the ratio of the compressive strength of concrete to the amount of torsional reinforcement. Figure 1 compares the experimental torsional strengths of the RC beams tested by Fang and Shiau to the torsional

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MECHANICAL BEHAVIOR OF BOLTED STUD-TYPE DAMPER WITH STEEL SHEAR PANELS

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SUMMARY

This research is concerned with a new type of stud-type damper with steel shear panels. The advantages expected from the damper are as follows. 1) Because all joints of the damper adopt high-strength bolts, construction of the damper is much easier than that of conventional stud-type dampers whose joints are welded. 2) The damper is available as seismic retrofit of existing steel buildings. 3) Replacement of energy dissipation devices after strong earthquakes is very easy. The mechanical model for estimating lateral stiffness and lateral strength of the stud-type damper is proposed and full-scale loading tests were performed for verification of the mechanical model. The results of the tests clearly reveal that the hysteresis loops of the specimens are stable. It is also verified that the design method of bolted joints are valid and theoretical results of lateral stiffness and lateral strength are agree well with experimental values.

Keywords: *stud-type damper, bolted connections, steel shear panel, full-scale loading test, mechanical model, seismic retrofit of existing steel buildings*

INTRODUCTION

Since the 1995 Hyogoken-Nanbu Earthquake occurred, it has been recognized that functional maintenance of buildings and asset protection against strong earthquakes are very important. Under such condition, it is thought that passive damping structures which can control damage of buildings play an socially important role. Various types of support of dampers have been developed so far, i.e. brace-type, wall-type, stud-type, and so on. In this research, the stud-type damper is chosen as the subject to the study. In placing stud-type dampers having steel shear panels for buildings, steel shear panels are usually connected at the middle of support members and support members are welded to wide flange beams at both ends as shown in Fig.1. Because adoption of damping elements as seismic retrofit of existing steel buildings is a final target, a new type of stud-type damper whose joints are all connected by high-strength bolts is proposed in this research (Fig.2).

The stud-type damper absorbs seismic input energy by repeated shear yielding of energy dissipation devices. Shear deformation of energy dissipation devices occurs by relative displacement toward vertical direction between two wide flange shapes (hereafter called H-stud) as shown in Fig.2. According to the recent researches about dampers, there have been developed some types similar to the stud-type damper proposed in this paper (Sakata et al. 2005, Torii et al. 2004, Sakino et al. 2004). Sakata et al. have proposed a wood frame with shear link passive control mechanism involving plywood panel, and Torii et al. proposed damping elements composed

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EXPERIMENTAL VERIFICATION OF DESIGN CRITERIA OF KNEE BRACE DAMPER

Yuji Koetaka [1], Yasuki Byakuno [2], and Kazuo Inoue [3]

SUMMARY

An innovative structural system, named weld-free system, has been developed to overcome the difficulty in quality assurance encountered in construction of steel moment resisting frames with conventional welded connections. Wide flange columns and beams are connected by means of double angles and high strength bolts. And buckling-restrained braces, referred to as knee brace dampers, are implemented to improve lateral stiffness of the frame as well as to dissipate seismic input energy. In this paper, design criteria of the knee brace damper is presented, and two kinds of experimental verifications that were conducted on sub-assembled test specimens are described. The primary objective of the first stage test is verification of design criteria of the restraining member, and the objective of the second stage test is verification of seismic performance of knee brace dampers. The test results clearly reveal that knee brace dampers can have large and stable hysteresis loop if they satisfy the proposed design criteria. It is also verified that there is sufficient plastic deformation capacity of several times required against a strong earthquake in knee brace damper.

Keywords : *Buckling-Restrained Brace, Bolted Connection, Design Criteria, Loading Test, Plastic Deformation Capacity*

INTRODUCTION

In order to assure sufficient plastic deformation capacity of welded beam-to-column connections for steel structures, various suggestions have been made in the United States and Japan after the 1994 Northridge earthquake and the 1995 Hyogoken-Nanbu earthquake. For example, the reduced beam section design (FEMA, 2000) has been widely accepted in the United States as an effective and economic solution. On the contrary, based on the observation that cracks often initiated at the toe of the weld access hole, Japanese researchers placed more emphasis on connection details to mitigate stress concentrations at welds and finally adopted the connection without a weld access hole as an alternative for building construction (Architectural Institute of Japan, 1996). Although these modified connections have shown satisfactory performance in the laboratories, it is realized that the quality of welds is difficult to control in practice so long as the structural fabrication relies on workmanship. The defects as well as insufficient deposition are often of concern regardless of the connection details adopted. As compared with welded connections in the United States, the Japanese practice generally requires larger volume of weld, implying that the Japanese connections are more relevant to the quality assurance problems (Nakashima, 2000).

To overcome the difficulty in the weld quality assurance as well as stringent post-Kobe requirements for welding practice, an idea to mainly utilize high strength bolts in beam-to-column connections with the number of welds minimized is appealing. In this regard, an innovative structural system, the "weld-free" system has

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CONTROL OF FLOOR VIBRATIONS USING TUNED MASS DAMPER

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SUMMARY

The Floors of Structures are often subjected to the periodic force which is induced by vibrating machines or human activity (walking, jumping, running etc). These periodic forces cause excessive oscillation. Such a vibration can be reduced efficiently by using tuned mass dampers.
The tuned mass damper, one of the simplest and most reliable control device applied to the vibration problems of the floors, has many advantages compared with other damping devices. But TMD is very sensitive even to a small offset in the tuning frequency ratio. This reduced the effectiveness of TMD the vibration control. In this paper, to decrease the magnitude of the floor motion, two TMD's per one bay were installed. Each damper consisted of four spring and two viscous damper which were used to adjust the TMD frequency and damping. After Installation of TMD's, the floor response is reduced about 40~50%

***Keywords**: Human Activity; vibration control; floor vibration; Tuned Mass Damper;*

INTRODUCTION

The recent improvements in performance of architectural materials and expanded use of steel provided more long-span and light-weight floor structures, resulting in increased possibilities for vibration due to dynamic loads such as mechanical operation or human activity. This vibration is affected by a reciprocal action between the characteristics of accelerated dynamic loads and the structure. The interaction between the loads and frequency of the structure results in unusual dynamic movements such as resonance and beating, generating excessive vibration that may lead to negative effects on people. Also, LCD monitors, widely used, are much lighter and have higher the center of gravity than the traditional CRT monitors, providing increased chances of witnessing visual vibration.
The natural frequency of the floor structure chosen for this research was 5.5 Hz with human activity. In order to decrease discomforted vibration for people, we measured natural frequency and inertance values of the floor slab using impact hammer test and exciter. We selected major location for the vibration control based on the results.
The previous studies have been reported that the tuned mass dampers (TMD) can efficiently reduce the vibration. The TMDs are applied to the floor slabs for reducing the vibration generated by the human activity in this study. The TMDs are specially designed as the slim types to install between floor slabs and OA floors. The developed TMDs in this study performed to reduce more than 40~50 percent of control improvements after installed.

THE EVALUATION OF STRUCTURAL VIBRATION

The vibration problems are currently being used as a research facility has six story with OA floors. The fifth floor has been the most problematic. Figure 1 shows the plan of the fifth floor.
The fifth floor plan presents that all researchers are to use only one entrance with no exception, and the main corridor is located towards area A from the center columns. The most of passengers go through area A so this

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SHAKE TABLE TESTS OF RC FRAME WITH SHAPE MEMORY ALLOY BRACING BARS

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SUMMARY

Many researchers in recent years have widely applied various sorts of passive control devices to enhance the structural performance on resisting and mitigating the external seismic excitation. One reliable passive control device is the shape memory alloy (SMA) which provides large recovery strain and effectively additional damping for structures. In this study, high seismic performance RC frames have been proposed to have SMA bars acting as a kind of structural bracing system at both sides of a frame to increase the energy dissipation capacity of the RC frame. The type of SMA bar used in the study is the Superelastic SMA bar. The force-displacement hysteretic loops of the RC frame with SMA bars under seismic loading are presented and compared with the test results of the bare RC frame. Test results show that the SMA bars can effectively reduce the maximum story drift of the tested frame. It was found that the reduction of story drift and base shear was depending on the characteristic of the input ground motion. Finally, mathematical model for frame with SMA has also been proposed in this study, the related coefficients were regressed in time domain by the least-squared method considering all nonlinear elements.

Keywords: *Shape memory alloys, Shake table, RC frame.*

INTRODUCTION

Shape Memory Alloys (SMAs) are the type of alloys which have the ability to dissipate energy through repeated cycling without significant degradation or permanent deformation. Superelastic SMA possesses the stable hysteretic behavior over a certain range of temperature, where its shape is recoverable upon removal of load. SMA shows a particular promise in civil infrastructural applications, especially in seismic resistant design and retrofit of structures. Recent years have observed increased research efforts in using shape memory alloy materials, in particular Nitinol, for civil structures to survive extreme events such as earthquakes. Nitinol, a special alloy of Nickel and Titanium, has very good electrical and mechanical properties, long fatigue life, and high corrosion resistance. Nitinol is the most popularly used shape memory alloy. The following summarizes recent advances in using Nitinol elements for civil structural vibration damping and energy dissipation.

- 10mm-diameter SMA bolt anchorages used for concrete column (Tamai et al., 2003).
- SMA wire-strand based devices to retrofit a historical bell tower in Italy (Indirli et al., 2001).
- SMA bar dampers added to the laminated rubber isolation system for elevated highway bridges (Wilde et al., 2000).
- Afull-scale SMA restrainer used in the simply supported bridge for seismic retrofit (DesRoches and Delemont, 2002).
- 30mm-diameter SMA tendons to enhance steel beam-column connections (Leon et al., 2001).

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INVESTIGATION ON THE SEISMIC CHARACTERISTICS OF THE BEDROCK EARTHQUAKE MOTIONS

Yong-Seok KIM[1]

SUMMARY

Lots of seismic analyses were performed with the earthquake motions recorded at the soft soil site without taking into account the effects of the soft soil amplification. However, it is important to utilize the bedrock seismic motions for rational seismic analyses of a structure considering the site soil conditions. In this study, the 26 bedrock earthquake records were selected from publicly available 1557 seismic records provided by the Pacific Earthquake Engineering Research Center (PEER) for the study, and the characteristics of them were investigated finding that it is not reasonable to estimate the earthquake acceleration intensity from the magnitude of an earthquake without considering the site soil conditions and it is hard to draw any relationship between earthquake acceleration intensity and epicenter distance with a poor bedrock earthquake record database.

Keywords: *earthquake motions, soft soil amplification, bedrock earthquake, PEER, acceleration intensity, magnitude, epicenter distance*

INTRODUCTION

Quite for a long time, lots of seismic analyses were performed with the earthquake records acquired from the soft soil ground without recognizing the importance of the effects of soil amplification and structure-soil interaction. Even though it was unconscious to use the ground motions of the soft soil sites mainly due to the poor seismic records, it should be realized that the erroneous research results can lead to the misunderstanding of the true seismic behaviors of the structures.
The seismic records of the 1940 El Centro Earthquake are the most famous ones could be placed in this category. According to the study results, the response spectra of the 1940 El Centro Earthquake have a suspicious clue of the soft soil ground motions as shown in Fig. 1, showing big peaks which are typical at the soft soil site lying on the bedrock due to the soil amplification. The response spectra of 1940 El Centro Earthquake are compared with that of a bedrock earthquake of 1994 Northridge Earthquake in Fig. 1.

During the last three decades, studies on the soil amplification effects indicate that it is necessary to use the bedrock or outcrop seismic motions for the seismic analyses of structures taking into account the site soil conditions. It is important and proper to understand the site soil effects on the structural response for the reasonable seismic analyses of a structure. However, it is not simple and easy for the practical engineers to acquire and discriminate the bedrock or outcrop earthquake records.

In this study, a research was performed to select the bedrock earthquake records from the earthquake records data base provided at the Pacific Earthquake Engineering Research Center (PEER) in the University of California at Berkeley. The PEER Strong Motion Database has publicly available 1557 records from 143 worldwide earthquakes including the 1935 Helena Earthquake and the 1999 Turkey Duzce Earthquake.
The characteristics of the 50 earthquake records classified as bedrock earthquakes listed in Table 1 are investigated to find out the effects of the site soil conditions. Also the relationships between earthquake intensity and magnitude or epicenter distance are studied to investigate the characteristics of the bedrock earthquakes.

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CRACK WIDTH OF REINFORCED CONCRETE COLUMNS SUBJECTED TO SEISMIC FORCE

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SUMMARY

In the new type earthquake-resistant design method, it will be required to define detail index to estimate the damage level for structural members. Because it make us easy to judge whether the building struck by earthquake motion is reworkable and the cost of repairing the damaged part in it. As a damage index for structures, the story drift have been used generally and it is useful to estimate the condition of damaged buildings roughly. But now it is indispensable to classify the damage levels of the structures to define and fix the damage level index to each member, namely, beams and columns. As a index to indicate the damage levels of concrete members, the width of crack developed in the member is quite suitable and useful. In Guidelines for Performance Evaluate of Earthquake Resistant Reinforced Concrete Buildings(Draft) published in 2004 by AIJ (Architecture Institute of JAPN)[1], acceptable values of residual crack width in plastic hinge area of concrete member in various damage level are recommended. But as reported in the guideline, the values recommended is provisional because we do not have enough knowledge to decide the dependable value. In this paper we study on the relationship between the bending crack width developed in the plastic hinge area and the drift angle of the member. Five RC columns were tested , varying axial compression, diameters of longitudinal reinforcement and so on. To investigate the behavior of the plastic hinge area and crack width, we used a high performance digital cameras. Based on the results of the experimental test, the way to estimate a crack width performed in plastic hinge area from drift angle of the members were discussed.

***Keywords**: earthquake engineering; reinforced concrete column; crack width; plastic hinge area digital camera.*

EXPERIMENATAL PROGRAM

Five RC columns measuring 1000mm in length and having typical 250mm square section were tested to investigate the behavior of plastic hinge area and crack width developed in the end of concrete column member(Figure 1, Table 1). Each specimen had the following feature. No.1 was a standard specimen. No.2 was a specimen applied low axial compression (axial force ratio is 0.11 which calculated by N/(Ac*sc) : N is the axial compression, Ac is the gross area of section, sc is the compression strength of concrete). No.3 was a specimen arranged by large diameter longitudinal bars on the condition amount of reinforcement was same as that of other specimen. No.4 was a specimen applied high axial compression (axial force ratio is 0.35). No.5 is a specimen applied high axial compression (axial force ratio is 0.35) and had much transverse reinforcement in the plastic hinge area(clearance of transverse reinforcement is narrowed by one-third in comparison with the standard specimen). In all specimens the shear strength was designed higher[2] than the bending moment[3]. Details of specimens are summarized in Table 1 and mechanical properties of reinforcement and concrete are shown in Table 2. The column specimens were loaded laterally through a parallel translation device in addition to constant axial load as shown in Figure 2 Lateral loads were reversed cyclic loads with 1 cycles at a rotation angle of members of 1/200, 1/100, 1/66, 1/50, 1/40 and 1/33 in No.1-No.3 specimen and 3 cycles at an angle of

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MOMENT CAPACITY ESTIMATION OF PRECAST CONCRETE COLUMNS ASSEMBLED BY POST-TENSIONING

Masanori Tani[1] and Minehiro Nishiyama[2]

SUMMARY

Use of unbonded tendons enables us to save troublesome grouting, to re-tension prestressing tendons and to disassemble jointed members. These advantages over conventional assembling methods have been attracting attention because environmentally-friendly structural systems can be realized; re-usable concrete members. In Japan, use of prestressed concrete members with unbonded tendons has been restricted to secondary structural members such as floor beams and slabs. After a code revision, which will be soon enforced, they can be used as primary earthquake-resistant structural members. Experimental and analytical research on post-tensioned beam-column assemblages has been carried out in the past. However, research on column-foundation assemblages post-tensioned by unbonded or bonded tendons is limited. Loading tests on several precast column specimens assembled by post-tensioning were conducted to investigate seismic behavior, such as failure modes, flexural strengths and stress of post-tensioning (PT) bars. Different failure modes were observed between the specimens with small and large axial load levels. An evaluation method for flexural strength and tensile stress of PT bars of unbonded specimens was proposed and proved to be successful.

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

INTRODUCTION

Use of unbonded tendons enables us to save troublesome grouting, to re-tension prestressing tendons and to disassemble jointed members. These advantages over conventional assembling methods have been attracting attention because environmentally-friendly structural systems can be realized; re-usable concrete members.

In Japan, use of prestressed concrete members with unbonded tendons has been restricted to secondary structural members such as floor beams and slabs. It is because it has been said that the stress variation in unbonded tendons is larger than that in bonded tendons, and the tendon low-cycle fatigue failure at the anchorage has been of great concern. Experimental results obtained by Muguruma et al.[1)], however, revealed that the stress variation in bonded tendons was larger than that in unbonded tendons because the bond between tendon and grout in the beam-column joint, in which or outside of which the anchorage is located, deteriorated under cyclic loading and the stress fluctuation at the critical section was transferred to the anchorage. Based on the research by Muguruma et al. and other researchers on prestressed concrete members with unbonded tendons, a code revision will soon be made, which allows use of unbonded tendons in primary earthquake-resistant structural members. Experimental and analytical research on post-tensioned beam-column assemblages[2), 3)] have been carried out in the past and the authors now have some amount of knowledge on their seismic behaviors.

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LATERAL LOAD-DISPLACEMENT ANALYSIS OF COMPOSITE *EWECS* COLUMNS

FAUZAN[1] and Hiroshi KURAMOTO[2]

SUMMARY

This paper presents an analytical study to describe the seismic behavior of composite EWECS columns which were compared with some experimental data. A moment-curvature analysis based on hysteresis models of materials was conducted to produce the response of the columns subjected to both constant axial load and lateral load reversals. The analytical results showed a good agreement with the test data. The results also confirmed the test data for the contributions of woody shell to flexural capacity by around 12 % in maximum. In addition, the results demonstrated the contribution of woody shell to the shear force and axial load until maximum R of 0.05 rad. Furthermore, a parametric study using the same numerical model was carried to examine the effects of axial load on behavior of EWECS columns.

Keywords*: Column; composite construction, fiber section analysis, seismic test, parametric study, effect of axial load.*

INTRODUCTION

Engineering Wood Encased Concrete-Steel (EWECS) column is a new composite structural system. The columns consist of concrete encased steel (CES) core and an exterior woody shell, as shown in Fig. 1. The use of woody shell as column cover have some advantages both economically and structurally, because the shell can be used directly as formwork for concrete placement and can provide the core confinement and resistance to bending moment, shear force and column buckling. Some experimental studies have been carried out to investigate seismic behavior of EWECS columns (Kuramoto et al. 2005, Fauzan et al. 2005 and Fauzan et al. 2006). Furthermore, an analytical study was performed to describe the behavior of EWECS columns under simulated seismic loading.

This paper presents an analytical study of seismic behavior of EWECS columns. A moment-curvature analysis based on hysteresis models of materials was conducted to produce the response of the columns subjected to both constant axial load and lateral load reversals. This paper also briefly outlines the experimental study of the columns. The results of the analytical study were compared with some experimental data. The effects of axial load on behavior of the columns examined by a parametric study using the same model were also described in this paper.

SUMMARY OF TEST RESULTS

Test Specimen and Method

A total of five column specimens of which the scale is about two-fifth, were tested in two Phases. The dimensions and details of the specimens are shown in Fig. 1 and Table 1. All specimens had a column with 1,600 mm height. In Phase 1, two specimens (Specimens WCS-1 and CS) were tested to investigate the seismic

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DAMAGE EVALUATION OF MULTI-STORY SHEAR WALL WITH AN ADJACENT FRAME CONSIDERING VERTICAL MOVEMENT OF FOUNDATIONS

Masanobu SAKASHITA[1], Kyohei MURAKAMI[2], Susumu KONO[3],
Hitoshi TANAKA[4]and Fumio WATANABE[5]

This paper will be submitted to The Second NEES-EDEFENSE Workshop on Collapse Simulation of Reinforced Concrete Building Structures, October 30 – November 1, 2006, E-Defense, Kobe, Japan

SUMMARY

Static loading test was performed with a 40% scale specimen, in order to check the basic performance of uplifting behavior, such as restoring force characteristic and energy dissipation capacity, and especially evaluate the damage in detail. The specimen consisted of a multi-story shear wall and an adjacent frame. At the first loading stage, the foundation was free to uplift and shear force was transferred by friction between the specimen and concrete blocks fixed to the reaction floor. The strength of specimen was considered to be reached, and then the first loading stage was terminated. After the first loading stage, the foundation was fixed to the concrete blocks and the second loading stage was continued.

From experimental results, the effect of uplift on the restoring force characteristic and energy dissipation capacity of the structures was clarified. The envelope curves of the experimental lateral load-displacement relations were simulated well using the nonlinear SAP2000 program and Tsuda et al’s shear wall model.

Keywords: foundation; vertical movement; multi-story shear wall; adjacent frame; restoring force characteristic;

INTRODUCTION

Upper structures are often designed assuming that foundations are fixed to the ground rigidly. The locking mechanism of foundations shouldn’t be neglected, if strength and stiffness of the subgrade aren’t enough. There is a possibility that foundations supporting rigid superstructures, such as shear walls, lift up. Some studies [2] indicate that foundation uplift may reduce the damage of the superstructures because shear force resisted by the superstructures decreases. On the contrary, if adjacent frames are connected to the uplifting shear walls, foundation uplift may cause large deformation leading the significant damage of the adjacent frames [3].

Recently performance evaluation procedures have come to be practiced regularly. In these procedures, it is important to connect the response of structures, such as maximum response or residual response, with the specific damage of structures. But the experimental studies dealing with foundation uplift are few, so enough information about the damage to establish the performance evaluation procedures has not been obtained. More detailed information, such as cracked region of members or crack widths, is also necessary.

In this study, static loading test, with a 3-story shear wall with an adjacent frame on one side, was conducted

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EXPERIMENTAL INVESTIGATIONS ON THE LATERAL RESISTANCE CAPACITY ENHANCEMENT OF A SHEAR WALL STRUCTURAL SYSTEM

Kyoung-Hun Lee[1] Jin-Young Park[2] Young-Hak Lee[3] Won-Kee Hong[4] Hee-Cheul Kim[5]

ABSTRACT

Most of apartment buildings in Korea were built with shear wall type. For old shear wall type structures, many structural problems were found at the lintel beam which is connected with shear walls. One of the most important structural members in shear wall type apartment building is a lintel beam under the lateral load. The failure mode of shear wall system can be occurred by differences of resistance capacity for moment between lintel beam and shear wall therefore structural capacity of coupled shear wall is depends on that of lintel beam.
While the high DOC(Degree of Coupling) exists on structure the ability of energy dissipation is relatively decreased, when the structural capacity of a certain structure is controlled by failure mode of shear wall. In this study, flexural capacity estimation test of coupled shear walls retrofitted by carbon plate was observed under the lateral load. All specimens were manufactured by 1/5 scale coupled shear walls with openings. Test results -such as maximum force, ductility level and energy dissipation area of coupled shear walls- were comparatively estimated with those of non-retrofit specimens.

Keywords: *Coupled shear wall; Carbon plate; Lintel beam*

INTRODUCTION

Most linear type residential apartment buildings in Korea have been built with shear wall system which mainly consists of RC shear walls and RC slabs to resist both vertical and horizontal loads. And several columns and beams are used as minor structural members in this structural system. However, the resistance capacity of this system against lateral load is relatively lower than the one against the vertical load because its geometric aspect ratio of horizontal axis to vertical axis in the plan is usually quite big (quite small vice versa)
Resistance capacity of coupled shear wall system under the dynamic lateral load such as seismic and wind load is governed by original structural capacity of lintel beams and shear walls with DOC(Degree of Coupling) of the coupled shear wall system. One of the effective and economical methods to increase structural capacity of the coupled shear wall system is reinforcing the lintel beams. In this case, ductility and energy dissipation capacity could be decreased due to cracks occurred in the shear wall.
In this study, lateral load experiments of the coupled shear wall specimens reinforced with carbon plate in order to investigate the lateral behavior of the reinforced coupled shear wall systems were performed. Since the shear walls were reinforced, the lateral resistance capacity of the coupled shear walls increased and cracks occurred in the lintel beam. According to the test results, the ultimate tensile strength of the carbon composite material was about 10 times higher than that of steel plate. Also, it is notable that the carbon plate is a light weight material along with good corrosion resistance capacity. The 1/5 scale shear wall specimens with 4 openings were tested

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Prediction of ultimate strength and deflection of double-curvature low rise shear walls

Ika Bali[1] and Shyh-Jiann Hwang[2]

Abstract

This study elucidates the prediction of ultimate strength and corresponding deflection of low rise shear walls subjected to lateral load. The scope of the study is limited to low-rise shear walls with height to length ratios not exceeding two, and walls due to lateral load are deformed in a shape of double-curvature. This study is based on limited knowledge of the shear behavior of low-rise shear walls subjected to double-curvature bending. In this study, the wall shear behavior as the ultimate shear strength and corresponding deflection of walls is predicted according to the softened strut-and-tie model. Moreover, the corresponding lateral deflection of walls is estimated by superposition of its flexibility sources such as flexural, shear and slip. Theoretical analyses reveal that the calculated results of the proposed procedure correlate reasonably with previously reported experimental results.

Keywords: reinforced concrete; double-curvature; low-rise; shear wall; strength; deflection; strut; tie.

1. Introduction

The seismic-resistant structural systems of reinforced concrete structural buildings generally use either moment resisting space frames, shear walls or a combination of both. However, shear wall systems exhibit better performance than space frame systems do (Fintel 1991).

Shear walls, which are used in lateral force resisting systems, can exhibit either ductile or non-ductile behavior. The ductile shear walls develop a flexural-ductile mode of failure if a severe earthquake occurs, while the non-ductile shear walls exhibit a shear mode of failure.

The non-ductile behavior of shear walls makes them appropriate for low-rise buildings due to their efficiency and economy. Shear walls are thus extensively applied for low-rise buildings in the form of reinforced concrete squat walls, which have height to length ratios not exceeding 2. The predominant action of such walls is shear, and the flexural yielding is practically limited.

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THE ANALYSIS OF THE RC FRAME WITH RC WALL BY USING THE EQUIVALENT COLUMN MODEL

Yeou-Fong LI[1], and Cheng-Wei CHEN[2]

SUMMARY

In this paper, an effective model was proposed to analyze the RC frame with RC walls. From many experimental results, we have found that the RC wall can resist part of the moment, the shear force and the axial load. The behavior of the RC wall is similar to the RC column. Therefore, the RC wall can be assumed to be an equivalent column. Using the equivalent column model, the RC wall combined with the pure RC frame is engaged and then analyzed following a non-linear pushover analysis to obtain the lateral force-displacement envelope of each frame. The analytical results can reasonably predict the lateral force-displacement relationships of these RC frames.

Keywords: *RC frame; RC wall; steel wire cable*

INTRODUCTION

After the Chi-Chi Earthquake in 1999, some columns of existing buildings suffered shear-failure damage but the building did not collapse. As already known, the brittle shear failure in RC columns is identified as one of the most dangerous failure modes because it may cause the collapse of buildings. In engineering practice, engineers did not quite understand the contributions of the RC walls under seismic loading in structural analysis. For simplicity, they usually ignore the existence of the RC walls in structural analysis. But, the plastic behaviors of the columns and walls are very important in the nonlinear structural analysis, especially for evaluating the seismic capacity of RC buildings. In the past, some researchers use soften trusses to simulate the RC walls, and these RC walls only exhibit the behavior of multi-linear two-force member. When the structure proceeds into non-linear behavior, the behavior of the RC wall can not be represented by the soften truss model perfectly. Therefore, the equivalent column model is proposed in this paper and is used to analyze the RC walls inside the RC frames. The analysis results are compared with the experimental results of the RC walls inside the RC frames.

THE CONSTITUTIVE MODEL OF CONFINED CONCRETE

The confined concrete constitutive model proposed by Li et al. (2003) was originally developed for concrete confined by CFRP. In this paper, we modify the L-L model (M.L.L. model) and extend the application of this model to concrete cylinders confined, respectively, by steel reinforcement only, by CFRP only, and by steel reinforcement and CFRP together. In this section, the ascending branch stress-strain curve of the Modified L-L model for concrete confined by both steel reinforcement and CFRP will be introduced in details.

Because the mechanism of confined concrete is similar to the mechanism of soil under tri-axial loading, the stress relation of confined concrete can be derived from tri-axial stress relation. According to the

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PREDICTION OF SEISMIC DAMAGE FOR BEAMS AND JOINT PANELS IN STEEL MOMENT FRAMES

Seiji MUKAIDE[1], Susumu KUWAHARA[2], and Motohide TADA[3]

SUMMARY

This paper considers a prediction method for seismic damage of individual members in a steel moment frame that forms a sway mechanism. In the proposed prediction method, seismic damage is estimated without using seismic response analysis, based on seismic input energy on the frame and restoring force characteristics of the members. The energy is distributed to cruciform subassemblages, which are employed as unit elements in the method. Maximum deformation and cumulative plastic deformation of the members (beams and joint panels) are calculated as seismic damage by assuming cruciform subassemblages deform due to the dissipation of the energy in specified situations. The applicability of the proposed method is confirmed through comparison with results of numerical analysis.

Keywords*: Seismic damage prediction; Maximum deformation; Cumulative plastic deformation; Earthquake response analysis; Joint panel; Cruciform subassemblages.*

INTRODUCTION

Members of structures must be designed with larger ductility capacity than the demand (based on the predicted seismic damage), in seismic design that permits plastic deformation. Many researchers study design problems from this point of view. For example, Ogawa (Nov. 2000) presented a seismic design procedure to estimate the ductility demand of beams in strong column - weak beam steel frames. The theoretical solution is obtained with equivalent single-degree-of-freedom systems. However, Ogawa does not present the respective deformations of beams and joint panels. It is quite possible that not only beams but also joint panels yield under severe earthquake loads. In this case, the damage of the joint panel influences the beam connected to it. Therefore, it is necessary to evaluate the damage of beams taking into account the damage of joint panels.

The purpose of the present study is to propose a prediction method for seismic damage of members (both beams and joint panels) in a steel moment frame that forms a sway mechanism. To be specific, the maximum deformation and cumulative plastic deformation of members are calculated by using seismic input energy on the frame and restoring force characteristics of the members, without using earthquake response analysis. The seismic responses are varied considerably, because of the varied characteristics of individual earthquakes, even if the structure is the same. The calculated deformation approximates the average earthquake response in an attempt to represent the general behavior of the members during a severe earthquake. This paper demonstrates the accuracy of the estimated deformations through the comparison with numerical results. It is our expectation that the method will provide a fundamental understanding of the ductility demand of beams and joint panels.

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COLLAPSE-RESISTANT DEMAND ANALYSIS OF STEEL MOMENT FRAMES WITH POST-NORTHRIDGE CONNECTIONS

Taewan KIM[1], Taejin KIM[2], and Jinkoo KIM[3]

SUMMARY

The purpose of this study was to investigate the collapse-resistant performance of post-Northridge steel moment frame connections proposed by FEMA/SAC project, which are WUF-W, RBS, WCPF connections. Among these connections, performance of the WCPF was a major interest herein so it was compared with that of other post-Northridge connection types. A three-story SAC building was selected as an example building. Not only the performance against collapse but also the seismic performance was investigated for comparison. Beam drift demand was used for an engineering demand parameter. For the collapse analysis, both linear static and nonlinear dynamic analyses were carried out. The result showed that the WCPF connection may have a merit of low demands in the connection region when a sudden removal of columns results in progressive collapse.

***Keywords**: SMRF, Collapse resistance, Seismic performance, Cover Plate, RBS.*

INTRODUCTION

Steel moment resisting frames (SMRFs) are being widely used for a major lateral force resisting structural system in high seismic regions. SMRF 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 SMRFs after the 1994 Northridge earthquake in the U. S. and 1995 Kobe earthquake in Japan. Since then, reliability analyses for the SMRF have been exceedingly executed. As a result, SAC Joint Venture was launched funded by the Federal Emergency Management Agency (FEMA), 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 SMRF (e.g., FEMA-350 2000 and FEMA-351 2000). 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 separated from the critical region). These seismic connections (PQ) have been prequalified by full scale tests so they are generally being used in SMRFs 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 SMRFs are of high interest for researchers. Of the connection types, seismic connections like the PQ are the very first option for the connection element in progressive collapse resisting structural systems. The Side Plate™ originally developed for a seismic connection is an example of having also a good progressive collapse resisting behavior, and it is used for the construction of public offices in the U. S. or embassies in abroad (Houghton and

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EFFECTS OF GROUND MOTION PARAMETERS ON PERFORMANCE EVALUATION OF STEEL WELDED BEAM-TO-COLUMN CONNECTIONS

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

SUMMARY

The deformation capacity of steel welded beam-to-column connections is usually evaluated by cyclic tests of specific amplitude and time history (i.e. code-prescribed loading protocol). However, this kind of engineering practice was found unable to ensure the connections having enough deformation capacity against great earthquakes like the 1994 Northridge Earthquake in U. S. and the 1995 Hyogoken-Nanbu Earthquake (Kobe Earthquake) in Japan. The necessity has arisen to investigate the effects of seismic loadings (or ground motion parameters) on the performance evaluation of steel welded beam-to-column connections. To address the above demand, a series of non-linear time history analysis was carried out here with the selected 21 sets of M6.5-8.0 strong ground motions. Analysis results show that the deformation demands may be very sensitive to the variations in ground motion parameters, and detailed attention should be given to the effects of seismic loadings in the performance evaluation of steel welded beam-to-column connections.

Keywords: *performance evaluation; seismic loadings; steel buildings; beam-to-column connections; deformation capacity.*

INTRODUCTION

Plastic deformation of beam-to-column connections is the main energy-dissipation mechanism of steel moment-resisting frames (SMRFs). Laboratory tests show that the performance of steel welded beam-to-column connections is sensitive to the details in design and construction. To confirm the capacity, newly developed steel welded beam-to-column connections are usually required to perform full-scale cyclic tests of specific amplitude and time history (i.e. code-prescribed loading protocol). However, this kind of practice was found unable to ensure the connections having enough capacity against the great earthquakes like the 1994 Northridge Earthquake in U. S. (Earthquake Engineering Research Institute (EERI) 1994) and the 1995 Hyogoken- Nanbu Earthquake (Kobe Earthquake) in Japan (Architectural Institute of Japan (AIJ) 1995 and 1997).

During the last several years following these two earthquakes, considerable efforts have been made to find the causes of brittle fracture of steel welded beam-to-column connections, and to help improve the details in design and construction (e.g. Skiles, and Campbell 1994; Bonowitz and Youssef 1995; Miller, D; Popov et al 1998; Kuwamura 1998, 2003 and 2004). There are many factors affecting the brittle behavior of steel structures, such as material and welding as well as seismic loading with a high degree of uncertainty. At present, it is still difficult to thoroughly resolve the brittle fracture of steel structures caused by severe earthquake shaking. Despite that, some tentative design methods have been proposed to prevent brittle fracture of steel structures (e.g. American Institute of Steel Construction (AISC) 2005). It appears necessary to investigate what kind of earthquake shaking might lead to steel fracture before adopting these design methods. A simple but reliable analytical study is therefore conducted here to investigate the influence of ground motion parameters on the

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VECTOR FORM INTRINSIC FINITE ELEMENT METHOD OF THE FAILURE AND COLLAPSE OF SPACE FRAME UNDER SEISMIC

Ren-Zuo Wang[1], Chung-Yue Wang[2], Keh-Chyuan Tsai[3]

ABSTRACT

In this paper, the explicit vector form intrinsic finite element (VFIFE) method is used to simulate the nonlinear behavior of the building system under seismic and blasting loadings. This VFIFE method can do the motion analysis of structure with large deformation and rotation from continuous states to discontinuous states. Three dimensional frame elements of the VFIFE method are used to model the structure system. The progressive failure of structure components was investigated in detail by considering the material nonlinearity and associated failure criteria into the analysis. In order to simulate the progressive collapse behavior of the structure, the mechanisms of contact detection and contact force calculations among frame elements are also developed.

Keywords*: VFIFE, large deformation, progressive collapse.*

INTRODUCTION

To prevent the immeasurable losses of human lives and social properties due to earthquakes, the seismic resistance evaluation and retrofitting of civil infrastructures becomes an important issue of many countries in the world. Besides experimental and theoretical studies, the numerical simulation is another way to assist engineers to understand the nonlinear dynamic failure behavior of structure under the earthquake excitation. Large deformation, progressive failure and collapse are the most critical deformation states causing structure damage and the threatening to lives. However, the mechanical behaviors inherited in these failure modes are extremely complicated and some physical quantities are difficult to be measured in the test. Therefore, the numerical techniques for the motion analysis of a given structure system under earthquake loading are essential in the evaluation of the capability of seismic resistance of structure.

Nonlinear analysis methods developed since last century are used to study the behavior of structures with material and geometrical nonlinearities. Gallagher and Padlog (1963) first introduce the geometrical stiffness matrix into the nonlinear analysis of structure by considering the nonlinear strain terms in the formulation. The rigid body motion of the structure will change the directions of internal forces and causes fictitious strains (Jagannathan et al., 1975a, b). Later Kohnke (1978) proposed the concept of fictitious force to remedy this phenomenon. Yang and Chiou (1987b) developed a rigid body test method to evaluate the geometrical stiffness matrix. Leu and Yang (1990) ; Leu and Yang(1991) stated that the structure has to satisfy the equilibrium equation during the rigid body motion to get rid of the error in the numerical analysis due to the fictitious forces. Argyris et al. (1978) and Elias (1986) have tried to modify the definition of bending moment to derive a modified geometrical stiffness matrix to satisfy the equilibrium requirement at each deformed state. Many researchers used the iterative method to solve the system equilibrium equations of incremental form (Mallett and Marcal (1968); Rajasekaran and Murray (1973); Leu and Yang (1990, 1991)). Leu and Yang (1990, 1991) proposed a predictor-corrector mechanism in the nonlinear analysis to accurately predict the elastic buckling behavior of space truss structure. Yang and Kuo (1994) proposed a method to decompose the displacement of structural element into rigid body displacement and natural deformation displacement in each incremental step of the computation and this kind of decomposition can lead the geometrical stiffness matrix pass the rigid body motion test. It is well known that the core idea of the nonlinera analysis of structure is how to clearly identify the rigid body component and the deformation component in the motion.

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DATA MODEL AND SOFTWARE TOOLS FOR MANAGING EARTHQUAKE ENGINEERING EXPERIMENTAL DATA

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SUMMARY

This paper presents a data model and a set of software tools developed at National Center for Research on Earthquake Engineering (NCREE) for management of earthquake engineering experimental data. The data model, called SAMCO Data Model, is designed to serve as a generalized reference model for modeling experimental data in earthquake engineering. It can be further extended and refined to address the needs of specific earthquake engineering experiments. A suite of data management software tools based on the SAMCO data model is also developed. The suite includes a desktop metadata editor with 3D information processor and a data management server. The desktop editor software adopts Extensible Markup Language (XML) technology to implement the data model and provide an interface for editing and sharing experimental data. The 3D information processor takes advantage of X3D (eXtensible 3D) to help handling the spatial information of an experiment. The data management server adopts relational database to implement the data model and provides a centralized environment to store, arrange, and manage all experimental data.

***Keywords**: Management of Experimental Data, Data Model, XML, X3D, Earthquake Engineering*

INTRODUCTION

Experimental data resulted from earthquake engineering experiments are very important resources for advancing our understanding on the behavior of structural systems under seismic loads. Analysis on these data has contributed many outstanding achievements in the domain of earthquake engineering. These experimental data are also very valuable resources because it typically takes considerable money, time, and manpower to carry out earthquake engineering experiments at a laboratory such as the National Center for Research on Earthquake Engineering (NCREE). However, reusing experimental data are often very difficult unless they are well managed and documented. At NCREE, for example, many researchers manage their experimental data by storing them into a large number of digital data files that are organized in computer folders. It has been found that other researchers or even the original researchers after some period of time cannot easily reuse former experimental data if the meanings of and the relationships among data files were not well described and documented. In addition, the lack of a standard scheme for management of experimental data makes the storage, retrieval, and reuse of the data more difficult and less efficient.

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SEISMIC PERFORMANCE OF RC STRUCTURES STRENGTHENED WITH PRECAST PRESTRESSED CONCRETE BRACES

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SUMMARY

This research aims to propose a simple seismic strengthening method which satisfies a) no wet concrete work, b) no rebar or bolt anchorage, c) short construction period, and d) low construction cost. For this purpose, a precast prestressed concrete brace system has been developed to demonstrate its efficiency experimentally and analytically. In an experimental phase, two identical half scale portal frames were constructed based on the old Japanese building standard and strengthened with two kinds of brace. Both frames showed more than 70% increase in lateral load carrying capacity and the efficiency of the bracing system was proved. Additional experiment was carried out on four 1/3 scale beam-column-brace joint models to evaluate the shear and bearing strength. In an analytical program, the nonlinear and dynamic frame analyses have been conducted to evaluate the effect of end fixity and out-of-plane deformation of braces on the performance of braced frames. It was shown that the failure mode of braced frames involved the tension failure of the first story column due the cantilever action, buckling or crushing of concrete brace, axial tensile failure of beams, and the bearing failure at joints. Based on the experimental and analytical results, economical and simple strengthening procedure using concrete brace and its design method is proposed. The proposed system is also shown to be applied to strengthen new buildings instead of using shear walls. A few retrofitted and newly strengthened buildings using the proposed method are introduced.

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

INTRODUCTION

Many buildings designed using the pre-1981 Japanese building standards experienced serious damage during the Kobe Earthquake in 1995 and it was recognized that such buildings were in need of prompt seismic retrofits. During the same year, the national organization promoting the evaluation of seismic susceptibility of buildings was founded and a law was enacted to promote seismic upgrades to deficient buildings. Since that time, a few public schools and government offices started upgrading their buildings, but the Ministry of Land Infrastructure and Transport recently announced that only 6% of 90,000 public schools and government offices had finished seismic upgrading and 1.2 million private buildings need seismic upgrading.
Many excellent seismic upgrading schemes enhancing strength and/or ductility already are available. However, seismic upgrading has made little progress due to reasons such as: suspension of building services during long construction periods, noisy and taxing construction process, and high cost. The probability of major earthquakes in the Japanese urban area between Tokyo and Osaka is reported as 80% in next thirty years. Before the existing old buildings are damaged by earthquakes, some measures need to be taken promptly. The purpose of this research is to develop a simple seismic retrofit method satisfying the following criteria.

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Forced Vibration Test for Simulating the Earthquake Response of a Real-Scaled Building Structure

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ABSTRACT

Forced vibration testing is important for correlating the mathematical model of a structure with the real one and for evaluating the performance of the real structure. There exist various techniques available for evaluating the seismic performance using dynamic and static measurements. In this paper, full scale forced vibration tests simulating earthquake response are implemented by using a hybrid mass damper. The finite element(FE) model of the structure was analytically constructed using ANSYS and the model was updated using the results experimentally measured by the forced vibration test. Pseudo-earthquake excitation tests showed that HMD induced floor responses coincided with the earthquake induced ones which was numerically calculated based on the updated FE model.

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IN-SITE PUSH OVER TEST OF EXISTING AND RETROFITTED SCHOOL BUILDINGS IN TAIWAN

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SUMMARY

In-site test is the most direct way to realize the actual behavior of structures. School buildings were found to have typical failure behavior because their typical architectural pattern. Based on the experience of the last earthquake disaster, school buildings usually fail in the longitudinal direction that has no walls. Therefore seismic retrofits for school buildings are usually centered on the longitudinal direction. In-site push over tests of three specimens were conducted to study the effects of different retrofitting measures. A 2-floor school building consists of 8 classrooms was cut into 5 parts, includes 3 specimens, each has 2 classrooms, and the rest 2 classrooms reinforced by steel bracing to provide reacting support. Original brick wing walls in the longitudinal direction were reserved in the first specimen. In the second one, all brick wing walls were demolished and RC wing walls were constructed instead. The last specimen was a pure frame with brick wing walls in the longitudinal direction removed. 6 hydraulic actuators were placed at the top of each floor to provide lateral loading along the longitudinal direction of specimens. While being lateral loaded, the specimens were also subjected to extra vertical loads by added weights on the slabs. Results of these tests were reported and analyzed in this paper.

***Keywords**: In-Site Test; Seismic Retrofit; School Building*

INTRODUCTION

In Taiwan, many typical school buildings suffered severe damage by the Chi-Chi earthquake, 1999. These school buildings usually have a standard plan that with all the openings in the longitudinal direction and partition walls in the transverse direction. Therefore common failure patterns were also found in them, such as failure in the longitudinal direction due to lack of walls, short-column effect due to constrain by windowsills, and strong-beam-weak-column effect due to non-ductile reinforcement and slabs that connect with the beams (Loh & Sheu 1999). It is urgent to develop the seismic assessment and retrofit technology for the existing schools for preparing for the future earthquakes. Usually small-scale or partial structural assemblages but not full-scale structure test result is employed to verify assessment method and retrofit measures. However, it is still questionable that if test results in the laboratory can represent the true behavior of actual buildings. Therefore, in-site test of existing school buildings were carried out for realizing the real structural behavior.

After an in-site push over test of Hsin-Cheng junior high school, the research team composed of crews of NCREE,

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CYCLIC RESPONSE OF ECCENTRIC BEAM-COLUMN CORNER CONNECTIONS WITH TRADITIONAL AND HEADED REINFORCEMENT

Hung-Jen LEE[1], Si-Ying YU[2], and Jen-Wen KO[3]

SUMMARY

This paper presents cyclic responses of six reinforced concrete exterior beam-to-column connections, namely, three pairs of concentric or eccentric connections. Specimen variables are joint eccentricity and anchorage of the beam bars terminating within the joint. One pair of connections used the traditional 90-degree hooks anchored within the joint. The other two pairs of connections used headed reinforcement consisting of screw-deformed bars and mechanical anchorage devices. All six specimens exhibited beam yielding in the 1.5% drift cycle, developed an anticipated beam plastic hinge in subsequent cycles, and eventually reached the joint shear capacity at a drift level of 4% or more. Each eccentric connection performed worse than each companion concentric connection. Using headed bars with single mechanical device provided anchorage as good as using standard 90-degree hooks. For the connections with single mechanical device, pushout spalling of the concrete covers behind the heads appeared after 5% drift level, which is considered to be acceptable in a building. Using double mechanical devices, two connections avoid pushout spalling and perform excellent cyclic response up to 8% drift.

Keywords: *beam-column; headed reinforcement; eccentric; joint; seismic design; shear; strut*

INTRODUCTION

The current ACI design methods for beam-column connections are given in ACI 318-05 Building Code Sec. 21.5 and its companion report of Joint ACI-ASCE Committee 352 (2002). In these procedures, the nominal joint shear strength is calculated on the effective cross-sectional area within a joint computed from joint depth multiplied to effective joint width. The effects of the column's aspect ratio and joint eccentricity are considered by limiting or reducing the effective joint width. Joint eccentricities between the beam and column centerlines are common in building frames for architectural reasons. Because relatively few experimental programs of eccentric beam-column connections have been verified to date, more experimental studies in this area are needed.

For the beam bars terminated in the exterior or corner joints, the use of standard hooks usually results in congestion with column lateral reinforcement (Fig. 1). The use of headed reinforcement in place of standard hooks in joints is a viable option (Wallace 1997). Wallace et al. (1998) have shown that the application of headed reinforcement within exterior or corner beam-column joints is appropriate. Headed reinforcement refers to the process of reinforcing a bar terminated with a head or end anchor plate. Figure 2 shows one type of headed reinforcement consisting of screw-deformed bars and mechanical anchorage devices. The mechanical device is a cast iron forming an anchor plate with a screw nut. The screw-deformed bar is a reinforcing bar with rolled-on deformations forming a screw for mechanical connection and anchorage. Hence, the mechanical device can be screwed onto the bar to provide a head. Because the rolled-on screws are quite loose, a nonshrink, high-strength

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CYCLIC BEHAVIOR OF STEEL COLUMN-TREE MOMENT CONNECTIONS WITH TAPERED BEAM FLANGES

Cheng-Chih CHEN[1] and Chun-Chou LIN[2]

SUMMARY

This paper presents test results of a proposed moment connection that improves and enhances the seismic behavior of column-tree connections between a steel beam and a welded box column. By enlarging beam flange in the beam-to-column joint and tapering a portion of the beam flange along the seismic moment gradient, a tapered beam flange connection is proposed to not only diminish potential for brittle fracture at the beam flange groove weld but also provide a large yielding zone in the beam. Full-scale specimens were designed and tested to clarify their cyclic performance. The test results demonstrate that the tapered beam flange connections are able to develop significant yielding and plastification in the beam away from the column face, which achieves reliable hysteresis performance with satisfactory plastic rotations.

Keywords: *column-tree; moment connection; hysteresis; plastic rotation.*

INTRODUCTION

Steel special moment frames (SMFs) are widely used in middle- and high-rise buildings because the structural systems are highly ductile and able to dissipate energy by developing inelastic deformation during strong ground excitation. Moment connections used in the SMFs in the US are generally connected by groove welding the beam flange and bolting the beam web to the column in the field. This type of the web-bolted flange-welded connection represents a field-welded beam-to-column connection. However, typical type of beam-to-column connections in Japan is connected mostly by welding the stub beam to the column in the shop and bolting the beam splice in the field. This type of shop-welded beam-to-column connection is performed intending to avoid poor field welding practice by constructing an often called “column-tree” while short pieces of stub beams are shop-welded to the column.

Fig. 1 demonstrates the column-tree construction. After the field erection of these tree-like columns, a mid-portion of the link beam is then spliced to the stub beams with bolted connections. Of course, the column-tree construction should be possible to increase some handling costs; however, defects caused by the field welding are intentionally reduced because the critical welding is performed in the shop to have better quality control. Unfortunately, the 1995 Kobe earthquake caused serious failure in the steel structures. After the earthquake, Nakashima et al. (1998) investigated and reported that many fractures were found in the shop-welded beam-to-column connections where they should have been able to achieve better welding quality. Nevertheless, limited yielding and plastic deformation were observed in the connections. The unexpected failures raised many doubts regarding a vantage of the quality control on these shop-welded, field-bolted connections.

This study analytically and experimentally investigates to enhance the seismic performance of the beam-to-column connections, achieving by widening and tapering the beam flange of the stub beams used in the column-tree construction. Nonlinear finite element analysis was employed to demonstrate the effectiveness of

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SEISMIC RESISTANT SELF-CENTERING MOMENT CONNECTIONS WITH BOTTOM FLANGE BUCKLING-RESTRAINED ENERGY DISSIPATORS

Chung-Che CHOU[1] and Yu-Jen LAI[2]

SUMMARY

This paper presents the results of experimental and analytical studies on self-centering moment connections. The connection subassembly consisted of post-tensioned steel beams, a reinforced concrete column, and energy dissipators, which were placed only below the beam bottom flange due to simplicity of construction, replacement, and no interference with the composite slab. Two types of energy dissipators were used: one includes a reduced section plate restrained by two flat plates; the other contains cross-shaped steel plates. Cyclic tests were conducted on three full-scale connection subassemblies and five energy dissipators. Parameters in these tests included the initial post-tensioning force, type and size of energy dissipators. Finite element analysis was also used to investigate the cyclic performance of the energy dissipators. Experimental and analytical results showed that (1) although energy dissipation, moment, and flexural stiffness of the beam in positive bending were larger than those in negative bending, gap opening angles at the beam-to-column interfaces were similar in both bending directions, (2) the energy dissipators were effective in dissipating energy in axial tension and compression until plate fracture, and (3) the radius of the cut in the reduced section plate should be larger than four times the plate thickness in order to avoid plate fracture along the shape transition.

Keywords: *Self-centering moment connection, Strands, Buckling-restrained energy dissipator*

INTRODUCTION

As an alternative to the steel welded moment connection, the post-tensioned technology has been applied to the steel connections by Ricles et al. (2002), Christopoulos et al. (2002), and Chou et al. (2004, 2005, 2006a). The connections in these studies incorporate seat angles, round bars, or reduced flange plates to dissipate energy, and high strength strands or post-tensioned bars to provide a self-centering capability. These connections exhibit similar force-deformation relationship and re-centering capability. The reduced flange plate restrained between the beam flange and the cover plate or T-shaped stiffener (Chou et al. 2006b) excludes the buckling of the plate in compression and is also able to provide stable energy dissipation. This research investigates further the cyclic responses of three post-tensioned connection subassemblies with energy dissipators placed only below the beam bottom flange in order to avoid interference with the composite slab. Two types of energy dissipators are proposed: one is composed of a reduced section plate restrained by two flat plates similar to that proposed by Suita et al. (2004), and the other is made of cross-shaped steel plates, which will buckle in large compressive force. Five cyclic tests and finite element analyses of the energy dissipators are performed to evaluate the hysteretic behavior and potential failure modes.

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Punching Shear Strength and Post-Punching Behavior of CFT Column to RC Flat Plate Connections

Cheol-Ho Lee[1], Jin-Won Kim[2], Jin-Gyu Song[3], and Eunjong Yu[4]

SUMMARY

This paper summarizes full-scale test results on CFT column-to-flat plate connections subjected to gravity loading. CFT construction has gained wide acceptance in a relatively short time among Korean construction companies due to its various structural and construction advantages. However, efficient details for CFT column to flat plate connections have not been proposed yet. Based on the strategies that maximize economical field construction, several connecting schemes were proposed and tested in this study. Test results showed that the proposed connections can exhibit punching shear strength and connection stiffness exceeding those of R/C flat plate counterparts. A semi-analytical procedure is presented to model the behavior of CFT column-to-flat plate connections. The five parameters needed to model elastic to post-punching catenary action range are calibrated based on the limited test data of this study. The application of the proposed modeling procedure to progressive collapse prevention design is also illustrated.

Keywords : *Composite Structure, Connections, Concrete Filled Steel Tube Column, Flat Plate, Punching Shear, Progressive Collapse.*

1. INTRODUCTION

The use of concrete filled steel tube (CFT) columns in tall buildings is rapidly increasing in Korea. The CFT system has a number of, well-known, advantages on structural performance and construction process over conventional reinforced concrete systems. In Korea, basement floors of tall buildings, which are usually used as parking areas, are often designed with reinforced concrete flat slab systems or flat plate systems to minimize construction costs and to enable rapid construction. For basement, flat plate systems are more advantageous than flat slab systems, since drop panels or capitals can be eliminated. Thus, the story height, accordingly the amount of excavation can be reduced. Moreover, total construction time can be effectively reduced since construction of basement floors generally takes up a substantial portion. Therefore, a CFT column system combined with flat plate floor may be the best option for construction of basement floors. However, the issues regarding the connections between CFT columns and reinforced concrete flat plates have not been addressed to date.

In this paper, efficient and robust connection details for concrete filled steel box columns to reinforced concrete flat plate system are proposed. Structural performances of proposed details are verified by full-scale testing programs. In addition, a modeling strategy of post-punching behavior and the design procedure to prevent progressive collapse are also proposed.

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EVALUATION OF BEAM YIELD DEFORMATION IN RC INTERIOR BEAM-AND-COLUMN SUBASSEMBLAGES

Masaru TERAOKA[1], Yoshikazu KANOH[2], and Satoshi SASAKI[3]

SUMMARY

To predict the yield deformation of beams more accurately, it is necessary to consider (a) the flexural and shear deformation of members, (b) additional deformation due to slippage of main rebar in the joint panel, and (c) additional deformation due to strain shifting obtained from the difference of measured and flexural analysis caused by diagonal crack at the beam end.
Using available data on interior beam-and-column subassemblages and considering all the three effects, a new prediction method of yield deformation of beams of presented in this study.
The following statements can be made from this study:
The beam deformations at beam yielding could be estimated approximately by considering (a) the flexural and shear deformation of members according to the beam theory, (b) the additional deformations of beams due to slippage of beam main rebars in joint panels, and (c) the additional deformations of beams due to strain shifting obtained from the difference of measured and the flexural analysis caused by shear cracking at the beam ends. Therefore, it can be said that the prediction method of deformation of beams at beam yielding showed its validity. As found in this study, the average percentages of the three components of deformations were about 50, 30, and 20 respectively.

Keywords:evaluation,beam yield deformation,interior,beam-column-joint,subassembrage

INTRODUCTION

It is one of the important subjects to predict the yield deformation of beams more accurately for the estimation of earthquake resistance performance of reinforced concrete structural frames of the beam yielding type. To predict the yield deformation of beams more accurately, it is necessary to consider (a) the flexural and shear deformation of members according to a beam theory, (b) additional deformation due to slippage of main rebar in thejoint panel, and (c) additional deformation due to strain shifting obtained from the difference of measured and flexural analysis caused by diagonal crack at the beam end.
In order to design a reinforced concrete frame for earthquake resistance, the yield deformation of the beams is usually predicted by using a simple but applicable equation

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