EFFECTS OF MODELING AND ANALYSIS PROCEDURES ON SEISMIC DESIGN OF HIGH-RISE RC COUPLED WALL STRUCTURES

Han-Seon LEE[1], Dong-Woo KO[2], and Sung-Wook JUNG[3]

SUMMARY

Seismic codes generally require to take into account the effect of cracks in modeling the structure and to conduct dynamic analysis for high-rise RC building structures. To satisfy these requirements is not straightforward for high-rise RC coupled wall structures because response spectrum analysis cannot account for the change of stiffness due to cracks in members in tension or in compression. The objective of this study is to quantify the effect of the variation of reduced section properties due to cracking on the estimation of member forces and drifts by using ACI, CSA, and NZS codes, and the difference between the analytical results, obtained through response spectrum analysis which uses the constant effective stiffness regardless of the state of tension or compression, and those obtained through equivalent lateral force procedure, which considers the effect of tension or compression forces on the wall stiffness. A sixty-story RC building structure was taken as an example.

Based on the observations on the analytical results, the followings are concluded; 1) The maximum values of story drift, axial force in shear wall, and shear force in coupling beam predicted by equivalent lateral force procedures are about twicer larger than those obtained by response spectrum analysis with the same amount of base shear. 2) The impacts of effective stiffness in coupling beams are largest in the interstory drift ratio and the base flexural moment in the wall while the difference in the other member forces are relatively small. 3) The effective stiffness in walls in response spectrum analysis has the significant influence on the lateral drift ratios, but the minimal effect on the member forces.

Keywords*: coupled wall structures, response spectrum analysis, effective stiffness*

INTRODUCTION

Many high-rise reinforced concrete(RC) building structures up to the height of sixty stories have been recently constructed in Korea. In this type of structures, RC shear walls resist most of earthquake and wind loads. They include in general elevator shafts and stair cases, thereby openings over which the short beams connect the shear walls. This type of walls are called coupled shear walls(CSW). The structural systems which include CSW are classified as dual system(DS) or building frame system(BFS) depending on the role of the peripheral frames in resisting lateral forces induced by earthquake ground motions.

Korean Building Code (KBC) 2005 (AIK, 2005) which was inacted by the Ministry of Construction and Transportation, in April, 2005, was established considering Korean practice and environment in design and construction by modifying International Building Code (IBC) 2000 (ICC, 2000). When compared with IBC 2000, KBC 2005 does not have the limitations on the use of structural systems and the height of building structures. But this code contains the requirements such as the consideration of the effect of cracks in determining the section properties especially for RC structures, the use of dynamic analysis, and the consideration of deformation compatibility of non-seismic-resisting frames such as in building frame systems. However, it is not straightforward to satisfy all the above requirements simultaneously. The reasons are as follows:

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LATERAL FORCE RESISTING MECHANISM OF A MULTI-STORY SHEAR WALL AND PERIPHERAL MEMBERS

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SUMMARY

One 1/4 scale specimen was designed and constructed as basic structural assemblage model extracted from a practical fourteen-story shear wall system. This specimen consisted of a bottom three-story part of the shear wall system, a foundation beam, slabs, and two piles. The yielding of the foundation beam in flexure was designed to precede the yielding of the shear wall in flexure. Static lateral load was applied with proportionally varying vertical load to simulate loading conditions of the prototype fourteen-story shear wall system under earthquakes.
Contrary to the design, the yielding of the shear wall preceded the yielding of the foundation beam. Flexure-shear cracks of the shear wall penetrated the slabs transversely and developed to the foundation beam. At the ultimate state, the shear wall separated along these cracks involving the parts of the foundation beam, the pile, the transverse foundation beam and the slabs. These clarified the monolithic action between the foundation beam and peripheral members. Transition of shear transfer mechanism at the shear wall base was observed from the strain distributions of longitudinal reinforcement in the foundation beam and those strain distributions were predicted accurately.

Keywords: *shear wall; foundation beam; pile; interaction;*

INTRODUCTION

In current design procedures [1][2], cantilever structural walls are normally assumed to stand on a solid foundation, and foundation beams, slabs and piles are designed separately without considering their interactions. This is because their interactions have not been thoroughly studied for its complexity. Also neglected in the practical design is the fact that shear transfer mechanisms along the wall base vary depending on the crack patters and inelastic deformation levels at the shear wall base. This study aims to experimentally clarify the variation of the lateral load resisting mechanisms considering the interaction between a shear wall, a foundation beam, slabs and piles, and to establish more rational design procedures for each structural component.
The specimen configuration was determined from typical fourteen-story residential buildings in Japan. They normally have multiple spans of a RC moment resisting frame in the longitudinal direction and a single span of shear wall system in the transverse direction. In this study, the assemblage consisting of the lowest three floors of a shear wall with a foundation beam, the first floor slab, and two piles in the transverse direction was scaled to 1/4.

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EFFECT OF REINFORCEMENT ORIENTATION ON SEISMIC PERFORMANCE OF REINFORCED CONCRETE SHEAR WALLS

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SUMMARY

Reinforced concrete shear walls are extensively used as the components of earthquake resistance buildings. However, the conventional shear walls, with vertical and horizontal reinforcements, constantly displayed the highly pinched shape in the hysteresis curves. This pinching effect will reduce the energy dissipation capability of shear walls. Former RC panel tests performed at the University of Houston show that controlling the orientation of the steel bars and the percentage of steel in a panel is possible to design shear-dominant structures more ductile. This paper compared the test results of 6 large-scale shear walls under reversed cyclic loading, including two walls contained conventional horizontal and vertical web reinforcement, two walls contained 45° web reinforcement, and two walls contained fan-shaped web reinforcement. The effect of web reinforcement orientation did not have a significant influence on crack pattern of all tested specimens. The compare results show that the pinching effect is remarkably improved in the shear walls with 45° web reinforcement. The larger steel ratio in the shear walls with incline web reinforcements also induces less pinching effect. The effect of web reinforcement orientation did not have a significant influence on the maximum lateral load, but energy dissipation capacity and pinching effect of the walls with 45° reinforcements are higher than the conventional ones. These ductile improvements can be referred to the ductility of the 45° reinforcements which are perpendicular the concrete crack orientation. To choice proper reinforcement orientation of the shear walls brings high potential to improve the seismic performance of buildings.

Keywords*: reinforced concrete; reinforcement; wall; ductility; large-scale test; seismic performance.*

Introduction

Shear walls have been recognized as efficient earthquake resistance elements (Fintel 1991). Framed shear walls are extensively used as the components of earthquake resistance buildings. However, the conventional shear walls, which the reinforcements are in vertical and horizontal directions, frequently possess pinching effect in the load-displacement curves. The pinching effect will reduce the energy dissipation capability of wall. The improvement of conventional shear wall to reduce the pinching effect sounds an essential research.

In the past 20 years, a lot of reversed cyclic loading tests were given to investigate the seismic behavior of reinforced concrete shear walls (Piliakoutas and Elnashai 1995; Mo and Kuo 1998; Salonikios et al. 1999; Zhang and Wang 2000; Sittipunt et al. 2001; Palermo and Vecchio 2002; Hidalgo et al. 20002; Greifenhagen and Lestuzzi 2005). Aspect ratio, strength, ductility, failure mode and energy dissipation capacity of shear walls under seismic loading have been studied. Previous results have demonstrated that mid-rise and low-rise walls behaved higher strength, less ductility and lower energy dissipation capacity than high-rise walls. The hysteretic loops of mid-rise and low-rise walls were also characterized by pinched shapes obviously, which is the

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STUDIES OF HIGH SEISMIC PERFORMANCE SHEAR WALLS – TESTS AND SIMULATION

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SUMMARY

Past RC panel tests of reinforced concrete membrane elements under reversed cyclic loading have much greater ductility when steel bars are provided in the direction of principal tensile stress. In order to improve the ductility of shear walls under earthquake loading, high seismic performance shear walls have been proposed to have steel bars in the same direction as the principal direction of applied stresses in the critical regions of shear walls. This paper presents the test results and model-based simulation of two shear walls under shake table excitation and two shear walls under reversed cyclic loading. In the specimens under shake table tests, steel bars were provided at angles of either 90 degrees or 45 degrees to the horizontal. In the reversed cyclic tests, one-half of the steel bars were placed at an angle of 45 degrees to the horizontal in the low-rise shear wall and at an angle of 65 degrees to the horizontal in the bottom portion of the mid-rise shear wall. Based on the experimental results, the tested shear walls with reinforcement oriented close to the principal tensile direction of applied stresses have greater ductility than that of the conventional shear wall. The model-based simulation based on the Cyclic Softened Member Model (CSMM) is performed and compared with the test results. The comparison shows that CSMM can well predict the seismic behaviors of the shear walls.

Keywords: *shear wall; shake table test; Cyclic Softened Member Model; reversed cyclic loading.*

INTRODUCTION

Structural walls can be divided into three groups based on the ratio of height to length. When the height to length ratio is greater than 2.0, they are called high-rise structural walls; when the height to length ratio is less than 1.0, they are called low-rise structural walls; when the height to length ratio is between 1.0 and 2.0, they are called mid-rise structural walls. For high-rise shear walls, the failure is mainly governed by flexure. In contrast, for low-rise shear walls, the failure is mainly governed by shear. For mid-rise shear walls, the failure is governed by both flexure and shear. For the low-rise and mid-rise structural walls, which are dominated by shear, they are also called shear walls.

The stiffness characteristics and shear strength of shear walls was investigated during the 1990s; Elnashai et al. (1990); Farrar and Baker (1992); Cheng et al. (1993); Colotti (1993); Eberhard and Sozen (1993). In the past 20 years, attention was given to the seismic behavior of reinforced concrete shear walls. Strength, ductility characteristic and energy dissipation capacity of shear walls under earthquake loading have also been studied ; Pilakoutas and Elnashai (1995), Mo and Lee (2000); Tasnimi (2000); Lopes (2001). Test results show that conventional low-rise and mid-rise shear walls have less ductility and lower energy dissipation capacity, which can be observed as a “pinching effect” in the hysteretic response of shear walls.

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BEHAVIOR OF HOLLOW STRUCTURAL SECTION BRACING MEMBERS WITH DIFFERENT W/T RATIOS

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SUMMARY

In this study hysteretic behaviors of Hollow Structural Section (HSS) bracing members having different width-thickness ratios (w/t ratio) are investigated. For this purpose, an experimental study was carried out using 7 specimens under quasi static symmetric reversed cyclic loading in tension and compression. Loading pattern was determined based on the results of a nonlinear dynamic time history analysis of a 3 story concentric braced frame under earthquake ground motions. This study observed that HSS specimens having a small w/t ratio experienced early fracture at the slotted end. It is noted that those specimens satisfied the requirement of w/t ratio for special concentric braced frames. To avoid this problem, this study proposed a design equation. The design equation was verified by additional experimental tests of two specimens designed according to the proposed design equation.

Keywords: *.Bracing members, Hollow sections, Experimentation, Fractures, Buckling*

INTRODUCTION

The most influential factors on the seismic behavior of bracing members are the *w/t* ratio and the effective slenderness ratio ($\lambda = KL/r$), where K is the effective length factor, L is the length of the bracing member, and r is the radius of gyration. Seismic behavior can be measured using fracture life, energy dissipation capacity, maximum drift and ductility capacity, etc. Most of studies on bracing members adopted un-symmetric displacement loading histories having compression oriented loading cycles in which specimens experience a larger displacement under compression rather than under tension during each loading cycle.

This study investigated the seismic behavior of bracing members with respect to the w/t ratio. For this purpose, eleven test specimens having different w/t ratios were made using cold formed hollow structural sections (HSS) having a square sectional shape. The structural details and design of the specimens followed AISC LRFD Manual (AISC 2001). Quasi-static reversed cyclic loading having symmetric tension and compression displacements was applied to each specimen to simulate seismic loading. The loading pattern was determined based on the results of nonlinear response history analysis of 3 story special concentric braced frame (SCBF) under earthquake ground motions.

EXPERIMENTAL PROGRAM

Test Specimens

Eleven cold formed HSS bracing members were tested, which had different w/t ratio. This study used

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STRENGTH DETERIORATION OF REINFORCED CONCRETE BEAMS SUBJECTED TO SEISMIC LOADING

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SUMMARY

Repeated or cyclic loading produces a progressive deterioration of bond that may lead to failure at cyclic bond stress levels lower than the ultimate stress under monotonic loading. Accumulation of bond damage is supposed to be caused by the propagation of micro-cracks and progressive crushing of concrete in front of the lugs. Degradation of bond primarily depends on the peak slip in either direction reached previously. Other significant parameters are rib pattern, concrete strength, confining effects, number of load cycles, and peak value of slip between which the bar is cyclically loaded. The research reported in this paper presents a method to predict the ductility of reinforced concrete members with short-span-to-depth-ratios failing in bond after flexural yielding. The proposed method takes into account bond deterioration due to the degradation of the concrete in the post yield range.

Keywords*: bond strength deterioration; shear strength; ductile capacity; truss model; reinforced concrete members.*

INTRODUCTION

The usual earthquake resistant design philosophy of ductile frame buildings allows the beams to form plastic hinges adjacent to beam-column connections. In order to carry out this design philosophy, the ultimate bond or shear strength of the beam should be greater than the flexural yielding force and should not degrade before reaching its required ductility. The behavior of RC members dominated by bond or shear action reveals a dramatic reduction of energy dissipation in the hysteretic response due to the severe pinching effects. After flexural yielding, plastic hinges develop near both ends of these beams and reversed cyclic loading produces a progressive deterioration of bond that may lead to failure at cyclic bond stress levels lower than the ultimate stress under monotonic loading. In addition, flexural bond stress increases with the increasing of plastic hinge length induced during positive and negative loadings.

Figure 1(a) shows a typical behavior of RC beams subjected to reversed cyclic shear and moment failing in shear or bond. Both behaviors of the beams show a reduction of ductility and energy dissipation in the hysteretic response after flexural yielding. The strength of the beam failing in shear suddenly dropped, while that of the beam failing in bond gradually decreased. After flexural yielding, the shear cracks developed diagonally to the member axis, while the bond cracks developed along the longitudinal reinforcing bars as shown in Fig. 1(b). In case of structures that deform primarily in the flexural mode, the response is governed by well-rounded hysteretic load-deformation curves because the response of such elements is governed mainly by the properties of the reinforcing steel bars. By comparison, RC members that deform primarily in the bond or shear mode show significant pinching around zero load, and severe strength deterioration in their hysteretic loops as shown in Fig. 1(a).

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SEIESMIC PERFORMANCE OF PRECAST/POST-TENSIONED REINFORCED CONCRETE BEAM TO COLUMN CONNECTIONS

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SUMMARY

The precast and post-tensioned structures have the advantage of self-centering and thereby eliminate permanent drift and reduce the possibility of being demolished after an earthquake. Literatures proved that the precast/post-tensioned building systems have steady responses under cyclic loading. In this research, four precast/post-tensioned RC beam-column connections have been constructed and tested. The investigated parameters include beam to column interfaces (steel on steel or plastic on plastic), energy dissipating devices (steel bars or angles) as well as the spacing of hoops in the panel zone. Test results showed that self-centering can be achieved for the post-tensioned reinforced concrete structures. With insufficient transverse reinforcement in the panel zone, two specimens failed due to panel shear with diagonal cracks from corner to corner. As rocking interface, steel plates performed better than the polyethylene plates but still not strong enough to protect the cover concrete under larger drift of loading. Energy-dissipating capacity of the bucking restrained steel bars and steel angles is satisfactory, however, steel angles are easier to install or replace. The force-gap opening relations and shear strength in the panel zone are evaluated by the theory.

Keywords: *beam-column connections; precast; prestressed; self-centering; testing.*

INTRODUCTION

Conventional designed reinforced concrete structures dissipated seismic energy through the damage in the plastic hinge zone. Even though, collapse of structures is suppressed, however, permanent plastic deformations make structures difficult to repair. Therefore, the precast and post-tensioned structures have the advantage of self-centering and thereby eliminate permanent drift and reduce the possibility of being demolished after an earthquake. The subject of self-centering is new, however, the technique of using prestress to enhance the structural performance is familiar. It can be dates back to the study of Park and Thompson in 1977 using post-tensioned tendons to improve the seismic performance of RC structures. Nishiyama et al. (1991) in Japan and Stone et al. (1995) in US tested several precast post-tensioned RC beam-column joints trying to simulate the seismic behavior of monolithic reinforced concrete structures, in an effort to improve the seismic performance of precast concrete connections in high seismic zone. Obviously, self-centering of structures was not the objective of above researchers. The first research to study self-centering effect in structures was done by Priestley and Tao in 1993. Seismic response of precast and post-tensioned concrete structures under earthquakes was analytically investigated.

In 1997, Mander and Cheng tested a precast and post-tensioned concrete column with self-centering feature. The column and foundation were precast and connected together with post-tensioned tendons. Test results showed that the column behaved in a bi-linear elastic fashion without inducing any damage or residual deformations after at least 20 cycles of 5% drift lateral loads. Even though the structure was not damaged after all, however, energy dissipation is limited due to the elastic manner of hysteretic loop. In 2002, the energy

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PERFORMANCE EVALUATION OF EXTERIOR RC BEAM-COLUMN JOINTS WITH ECCENTRICITY

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SUMMARY

Cyclic loading response of five reinforced concrete corner beam-column connections with one concentric or eccentric beam framing into a rectangular column in strong or weak direction is reported. The specimen variables are the direction of shear acting on the joint and the eccentricity between the beam and column centerlines. Experimental results show that two joints connecting a beam in strong direction were capable of supporting adjacent beam plastic mechanisms. The other three joints connecting a beam in weak direction, however, exhibited significant damage and loss of strength after beam flexural yielding. Eccentricity between beam and column centerlines had detrimental effects on the strength degradation, energy dissipation capacity, and displacement ductility of the specimens. It was concluded that ACI design procedures for estimating nominal joint shear strength were unconservative for the tested corner connections under lateral loading in weak direction of the column. Valuable information is provided to help further improve the design requirements for eccentric corner beam-column connections.

***Keywords**: beam-column connection; corner; eccentric; joint; shear strength.*

INTRODUCTION

Shear failure in beam-column joints leading to collapse of reinforced concrete (RC) buildings has been observed in the past earthquakes. The cause of collapse has been attributed to lack of joint confinement, especially for the exterior and corner beam-column joints without beams framing into all four sides. Since the late-1960s, amounts of experimental investigations on the seismic performance of RC beam-column joints have been extensively studied. The majority of the experimental programs has concentric beam-column connections isolated from a lateral-force-resisting frame at the nearest inflection points in the beams and columns framing into the joint. Since 1976, ACI-ASCE Committee 352 has issued design recommendations for RC beam-column joints. (ACI 352R 1976, 1985) Throughout the years these guidelines evolved into state-of-art reports(ACI 352R 1991, 2002) by integrating results of new experimental programs. Finally, a number of these design recommendations for beam-column connections have been adopted in Chapter 21 of ACI 318 Building Code (2005) for seismic design. It should be noted that the current ACI design provisions are primarily developed from test results of concentric beam-column connections, however, eccentric beam-column connections are common in practice. Relatively few RC eccentric beam-column connections have been tested and reported in the literature to date. (e.g., Joh et al. 1991, Lawrance et al. 1991, Raffaelle and Wight 1995, Chen and Chen 1995, Vollum and Newman 1999, Teng and Zhou 2003, Burak and Wight 2002, Shin and LaFave 2004). To clarify the effect of eccentric beams on the behavior of connections, ACI-ASCE Committee 352 has called for additional research on this topic in the past two decades.

In the early-1990s, Joh et al., (1991) Lawrance et al., (1991) as well as Raffaelle and Wight (1995) totally tested six cruciform eccentric beam-column connections with square columns. Early deterioration of connection strength and ductility was observed in these eccentric specimens. The measured stains in joint hoop

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TENSILE FORCE EVALUATION OF UNBONDED PT BAR IN PRECAST CONCRETE COLUMNS

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SUMMARY

This paper describes a parametric study on precast concrete columns focusing on tensile force evaluation of unbonded post-tensioning (PT) bar. The analytical method used in the study is based on a layer-element method proposed in the past. The parameters are axial load level, prestressing force level, and compressive strength of concrete. A major objective of this study is to investigate the effect of these parameters on tensile force variation in PT bars, which is necessary for flexural strength evaluation of prestressed concrete columns. The analytical results showed that tensile force in PT bars at ultimate strength decreased as the axial force level (axial load level + prestressing force level) increased.

A simple macro model was proposed and employed to obtain tensile force in PT bars and column rotation angle at the ultimate strength more easily than the layer-element analyses mentioned above. Tensile force in PT bars at the ultimate strength obtained by the macro model agreed well with the results from the analytical method. In addition, a simpler section analysis using the ACI concrete stress block and plain section assumption was carried out, and it was found that it gave 15~20% smaller ultimate strengths than the above analyses.

Keywords*: Precast concrete column; post-tension; unbond; layer-element method; macro model.*

INTRODUCTION

For evaluation of flexural strengths of prestressed concrete members, it is necessary to evaluate tensile force of post-tensioning (PT) bars as accurately as possible. In the past, an analytical study on cantilever precast concrete columns post-tensioned by unbonded tendons[1] was reported. If unbonded tendons are straight and the member is subjected to anti-symmetrical loading, variation of tensile force in an unbonded PT bar is small because the bar in compression at one end is in tension at the other end of the member as shown in Fig. 1. The behavior is considered to differ from that of a cantilever member. This paper describes a parametric study on precast concrete columns focusing on tensile force evaluation of unbonded PT bars by using the analytical method reported in the references [1] and [2]. The columns are assumed to be subjected to the anti-symmetrical loading. The parameters are axial load level, prestressing force level and compressive strength of concrete.

SUMMERY OF ANALYSIS

The analysis is based on the layer-element method in which the stiffness matrix of a member is constructed in terms of layers in the section and longitudinal elements as shown in Fig. 2. The reference [2] dealt with cantilever members, while this paper is intended for the members under anti-symmetrical loading, which simulates earthquake loading against a member restrained at the ends.

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SEISMIC BEHAVIOR OF PRECAST PRESTRESSED CONCRETE MEMBERS WITH GRADED COMPOSITE STRANDS

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SUMMARY

Two types of graded composite strands, GCS-U and GCS-H, have been proposed and precast prestressed cantilever beams with these strands were tested to see the hysteretic characteristics. GCS-U has one unbonded high strength strand surrounded by twelve bonded mild steel wires, and GCS-H consists of four ordinary strength wires and three ultra high (1.2 times in yielding) strength wires. Damage of concrete concentrated at the fixed end of beams. Increase of energy dissipation was not as large as expected, but there was possibility of improvement by considering the arrangement of strands in section. Ultimate flexural moment capacity can be simulated accurately by considering deterioration of strand bonding.

***Keywords**: Graded Composite Strand; precast prestressed structure, residual deformation, high energy dissipation.*

INTRODUCTION

Precast prestressed structures provide small amount of energy dissipation and tend to deform more than ordinary reinforced concrete structures, although they show very small residual deformations. 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 confirmed that they were able to increase energy dissipations with keep small residual deformations. However, there were some cases that strands fractured by concentration of strain. In this study, new GCS's were proposed and cantilever specimens with them were tested to confirm their efficiency and safety.

GRADED COMPOSITE STRAND

Figure 1 shows GCS which consists of four low strength wires and three high strength wires. Under seismic force, low strength wires become yielding and provide energy dissipation, while high strength wires are elastic and restrain residual deformation, shown in figure 2.

In 1993, Niwa et al tested precast cantilever specimens and confirmed that members with GCS were able to provide sufficient energy dissipation and keep residual deformation small like in figure 3. However some specimens that have large prestressing force showed strand fracture.

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Direct Inelastic Earthquake Design Using Secant Stiffness (Local Deformation Control Design)

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SUMMARY

A new earthquake design method, Direct Inelastic Design (DID), was developed. Existing inelastic design methods use complicated step-by-step analytical technique, and control overall deformation of a structure, which is not directly related to the safety of each member. On the other hand, the DID directly determines the inelastic strength and deformation of structural members using linear analysis for secant stiffness, and controls local deformations of potential plastic hinges of the members. The inelastic strength and deformation of the potential plastic hinge are controlled to belong to an admissible zone which is determined by the earthquake design strategy intended by the engineer, such as the concept of strong column-weak beam. For a design example, a typical reinforced concrete structure was designed by the DID, using the design concept of strong column - weak beam. The result showed that flexural damage of the columns was prevented and plastic hinges of the beams were distributed along the height of the building. The design results of a beam and a column and the overall response of the structure were presented and discussed in detail.

***Keywords**: SEEBUS; seismic design; earthquake engineering; secant stiffness; reinforced concrete*

INTRODUCTION

The response of building structures subject to strong earthquake is inelastic in nature. However the equivalent static design method using linear elastic analysis is still popular for practice engineers because of its convenience in analysis and design. However, safety of the structures designed by the equivalent static method cannot be secured because the inelastic response of the structures cannot be accurately estimated.

Recently, to overcome such disadvantages of the equivalent static method, a variety of earthquake analysis/design methods using nonlinear static analysis were developed: the Capacity Spectrum Method (CSM, ATC-40 1996) and the Direct Displacement-Based Design Method (DDBD, Priestley 2000). Unlike the conventional equivalent static method, the nonlinear static methods can estimate the inelastic seismic performance of structures. As a result, the structural safety against earthquake can be secured. However, even these nonlinear static methods still have several disadvantages in application.

The CSM performs conventional step-by-step inelastic analysis for preliminarily designed structures. Therefore, the engineers must have sufficient knowledge on the complicated inelastic analysis. To secure the structural safety and to achieve an economical design, evaluation and redesign must be repeated. As a direct design method, the secant stiffness methods such as the DDBD can be used. The existing secant stiffness methods simplify an actual complex structure to a substitute structure of single degree-of-freedom, and can determine the strength and ductility demands of the substitute structure. However, it cannot determine the inelastic strength and deformation

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SEISMIC RETROFIT USING PRECAST PRESTRESSED CONCRETE BRACES

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SUMMARY

A new seismic retrofit scheme of reinforced concrete frame using the precast prestressed concrete braces is introduced so that the retrofit scheme satisfies a) no re-bar connection or bolt anchorage between the brace and existing frames, b) short construction period and, c) low construction cost including the out-of-service loss. The lateral load resisting mechanism of braced frame is explained followed by the issues to be solved in design. Then an experiment on two half-scale portal frames is explained. The specimens were constructed based on the pre-1980 Japanese Building Standard and strengthened with the proposed reinforced concrete brace with potentially different failure modes. Both frames showed more than 70% increase in lateral load carrying capacity without excessive frame deformation and demonstrated the effectiveness of the system.

Keywords: Seismic retrofit; Precast prestressed concrete brace; No bolt anchorage.

INTRODUCTION

Upgrading of seismic performance of existing buildings can be achieved by several construction methods such as construction of new in-filled walls and diagonal steel bracings. These upgrading methods enhance strength and/or ductility. However, seismic upgrading makes little progress due to several reasons such as suspension of building services for long construction period, noisy and troublesome construction works, high construction cost, and etc. The purpose of this research is to develop a simple seismic strengthening method which satisfies a) no re-bar connection or bolt anchorage between the brace and existing frames, b) short construction period and, c) low construction cost including the out-of-service loss.

Concept

An X-shape precast prestressed concrete brace consists of multiple precast units. Figure 1 shows four-unit assemblage and it is also possible to have assemblage with four legs and one central unit. These units are assembled at a construction site and prestressing force is introduced to two diagonal directions as indicated in Fig. 1(a) and gaps between brace ends and frame corners are filled with high strength no-shrinkage mortar as shown in Fig. 2(a). Then, the prestressing force is released after grout mortar hardens so that the X-shape brace extends by itself to fix to the existing frame.

When the frame with the brace is subjected to lateral seismic load, only one of diagonal members works effectively in compression. If there is no prestressing introduced, the remaining diagonal member becomes free of stress and may come out of the frame because concrete does not carry tension force. To avoid this, a device with flat springs and steel pipe (FSSP) in Fig. 1(b) is installed at the bottom end of each diagonal member. This

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FIELD TESTS ON RC BUILDING OF HSIN-TSENG JUNIOR HIGH SCHOOL IN TAIWAN FOR SEISMIC RESISTANCE

Yi-Hsuan TU[1], Shyh-Jiann HWANG [2], and Wen-Ching JAUNG[3]

SUMMARY

The 1999 Chi-Chi earthquake revealed the poor performance of the RC school buildings in Taiwan. It also indicates an urgent need of seismic evaluation and retrofit for the remaining schools. In order to realize the behavior of a typical school building subjected to lateral load, an in-situ test for an existing 2-story school building was carried out. Two experiments were conducted: static push over test to identify the strength, stiffness and toughness of the building, and vertical load test to study the vertical load carrying mechanism after part of members failed. In the static push over test, a 6-classroom building constructed in 1964 was cut in the middle where jack was set for monotonic loading alone the longer axis of building. One half of this building was reinforced by steel bracings to provide reaction support, while the other half was pushed to failure. The vertical load test employed only one of the classrooms. Inner columns of 1F of the classroom were cut off in the middle to simulate that they are failed prior due to the short-column effect. The remaining frames with thick brick in-filled walls as partitions were expected to carry the weight of and prevent collapse. Water was added into two tanks set at the 2F and RF slabs as vertical loading. Results of these tests are reported, analyzed and interpreted in this paper.

Keywords: *Field Test; School building; Reinforced Concrete; Seismic Resistance.*

INTRODUCTION

In Taiwan, many typical school buildings suffered severe damage by the Chi-Chi earthquake, 1999. Most of old school buildings were designed according to a standard plan that is functional for getting natural light and ventilation. The typical plan has all the openings and a corridor in the longitudinal direction and many partition walls in the transverse direction. Some common failure patterns were found because of the typical type of school buildings, 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. For preventing possible damage in the future, it is urgent to develop the seismic assessment and retrofit technology for the existing schools. Although there are already some assessment methods developed by international researchers, usually they are verified by small -scale or partial structural assemblages but not full-scale structure. It is still questionable that if test results in the laboratory can represent the true behavior of actual buildings. Therefore, an in-situ push over test of an existing school building is carried out for realizing the real structural behavior.

Indebted to the Hualien County Government and Hsin-Cheng junior high school, the research team composed of crews of the National Center for Research on Earthquake Engineering (NCREE), the National Taiwan University of Science and Technology (NTUST), the Dahan Institute of Technology (DHIT) and the National Taiwan

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THEORETICAL AND EXPERIMENTAL STUDIES ON THE AS-BUILT AND REPAIRED REHABILITATED RC FRAMES

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

SUMMARY

In this paper, an effective repair-rehabilitation working method is proposed for moderately damaged reinforced concrete (RC) building structures after major earthquakes. Three RC frames with nil, half-height and full-height brick walls are designed and tested at the National Center for Research on Earthquake Engineering (NCREE). After the columns of these non-ductile RC frames are damaged, steel wire cables with non-shrinkage motar and carbon fiber reinforced plastics (CFRP) are used in the proposed method to confine reinforced concrete columns. The stress-strain relationship of the confined concrete, proposed by Li et al. (2003), is used in the theoretical sectional analysis. The columns are confined by steel wires and CFRP and the Response 2000 program (Bents, 2001) is used to obtain the moment-curvature relationship of these confined columns. The "equivalent column model" is proposed in this paper and is used to analyze the brick panel inside the RC frames. Finally, the frame and the equivalent column are engaged and then analyzed following a non-linear pushover analysis to obtain the lateral strength-displacement envelope of each frame. The analytical results can reasonably predict the lateral force-displacement relationships of these RC frames.

***Keywords**: non-ductile frame; carbon fiber reinforced plastics; steel wire cable*

INTRODUCTION

In 1999, a major earthquake measuring 7.3 on the Richter scale hit Central Taiwan. After the so-called Chi-Chi Earthquake, 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. If the non-ductile frames or columns sustained shear-failure damage could be repaired and then rehabilitated to meet the current seismic code, it should be more economical and feasible than demolishing and reconstructing the whole building. As seen from the buildings after Chi-Chi Earthquake, most of the damaged buildings have non-ductile frames that fail because of their poor design or construction. It is necessary to develop an effective and efficient repair-rehabilitation working method to prevent the buildings from collapsing during the earthquakes and aftershocks.

In this paper, an effective and efficient repair-rehabilitation working method is proposed. This proposed repair-rehabilitation working method combines the steel wire cables with non-shrinkage motar for repairing and carbon fiber reinforced plastics for rehabilitation. Research on using the steel wire cable to repair moderately damaged column was proposed by Li et al. (2004). CFRP composite material has been widely used in the retrofit and rehabilitation of buildings and bridges due to its merits of anti-corrosion, lightweight, easy cutting and construction, as well as high strength-to-weight ratio, high elastic modulus, and high resistance to environmental degradation. The advantages of this repair-rehabilitation working method are: (1) it can easily be applied by less-experienced workers; (2) the materials are available and easy to obtain from the warehouses; (3) it does not call for any heavy equipments; and (4) it is faster than other traditional working methods, such as

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EXPERIMENTAL STUDY
ON THE JOINTS OF SEISMIC RETROFITTING BRACES
FOR STEEL STRUCTURES
USING A SHEAR-KEY PLATE ADHERED TO A CONCRETE SLAB

Yasushi ICHIKAWA[1], Hiroshi NAKAMURA[2], Satoshi YAMADA[3], and Akira WADA[4]

SUMMARY

Fitting additional braces is a typical method of seismic retrofitting of existing steel structures. The welding of steel and chipping of floor concrete involved in the fitting work of the braces cause noise and vibration, and this poses a problem for the seismic retrofitting of buildings in service. In order to solve this problem, this paper proposes a new joint method for seismic retrofit braces. In this method, the connecting steel plate for a brace is jointed to column-to-beam connection by high strength bolts and a shear-key plate. The high strength bolts are arranged to the column and the shear-key plate is bonded to the concrete slab by epoxy resin and chemical anchors. The portion adhered by epoxy resin is brittle, so it is important that this connection has sufficient strength compared with a brace strength and that the upper limit of yield stress of the brace is controlled. Therefore buckling restrained braces with a low yield steel of its core are applied to this joint. The yield stress of the low yield steel is restricted. By using these items, it is not necessary to chip the concrete slab and weld on the site. This paper reports about the loading tests of the joint portion assumed a real size and the cyclic loading tests of frames assumed a half size to evaluate the dynamic characteristics of this joint method, and refers to the strength of this joint.

Keywords: *seismic retrofitting, epoxy resin, chemical anchors, buckling restrained brace*

INTRODUCTION

The need for seismic diagnosis and seismic retrofitting of existing buildings not sufficiently resistant to earthquakes has been strongly felt since the South Hyogo Earthquake, commonly known as the Hanshin-Awaji Earthquake, in January 1995. The technology of building reinforcement has drawn the interest of many researchers ever since, and various papers have been presented on new methods of building reinforcement (Kukita et al. 2000). In the meantime, a variety of dampers excellent in energy-absorbing capacity have been actively developed; such dampers include steel dampers that transform seismic energy into plastic work and friction dampers that transform the energy into frictional heat. Many papers have reported that retrofitting of existing buildings by the appropriate application of the dampers is effective in reducing the response displacement of building stories and the response shear force by seismic force.

When braces and dampers are used for seismic retrofitting of an existing steel structure by conventional work methods, however, noise, vibration and dust are likely to occur because of site welding and chipping of concrete slabs. Therefore, it is often difficult to perform such retrofitting while the building is in service.

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COMPRESSIVE STRENGTH ENHANCEMENT OF CIRCULAR CONCRETE COLUMNS CONFINED BY CARBON SHEET TUBE

Hee-Cheul KIM[1], Kyung-Hoon LEE[2], and Won-Kee HONG[3]

SUMMARY

An experimental research has been accomplished to investigate the capacity of concrete filled carbon sheet tube columns. In general, two types of column sections, rectangular and circular shapes, are used. If a rectangular shape carbon tube is fabricated with carbon sheet, the confinement effect would be decreased due to the stress concentrations at the four corners of a rectangular section. Therefore, a circular shape that has constant curvature was chosen for this study. Total 18 concrete column specimens confined by carbon sheet tube were tested using 10,000*kN* universal testing machine(UTM).

The maximum compressive strength of a concrete filled carbon sheet tube column is 72.93 *MPa* for the 90-5T specimen and the maximum strain is 0.0247 for the 75-5T specimen. An incremental ratio of the compressive strength and the strain are about 4.2 and 12.4 times greater than those of a plain concrete column, respectively. Compressive strength prediction equations of a concrete column confined by carbon sheet tube is proposed through the regression of experimental results which can consider the arrangement angles of a carbon with respect to the longitudinal direction of a column.

Keywords: *carbon composites, glass fibers, strength, carbon sheet tube*

INTRODUCTION

Carbon sheet has many structural advantages such as high tensile strength, corrosion resistance capacity, light self-weight, and so on. Especially, the tensile strength of a carbon sheet is about 10 times higher than that of steel. Therefore, the carbon sheet can be a suitable material for the repair and reinforcement of structural element of buildings and bridges. In this study, increment of compressive strength of CFCST (Concrete Filled Carbon Sheet Tube) column was observed thorough an experimental investigation.

Early research of strength increment by lateral confinement has been studied in 1988 by Mander(1988)[(1)]. He proposed a prediction equation for compressive strength of rectangular and circular concrete columns confined by hoop steels. Saafi(1999)[(2)] proposed a regression equation of confined columns for relationship with the axial compressive strength and the lateral confinement pressure confined by carbon and glass fibers. An experimental equation of compressive strength for concrete columns confined by carbon fibers and hoop steels was proposed by Han and Hong(2005)[(3)] in Korea.

Generally, lateral reinforcement means that the reinforcing material is confined by 90 degree with respect to the longitudinal direction of a column. However, Hong and Kim(2003)[(4)] proposed an experimental equation with regression analysis for variable angles of carbon fibers and layers of carbon sheets. Lam and Teng(2003)[(5)] also proposed a new theoretical equation by analyzing lots of data obtained from several other researcher's test results.

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OVERSTRENGTH AND RESPONSE MODIFICATION FACTOR IN LOW SEISMICITY REGIONS

*Dong-Guen Lee[1], So-Hoon Cho[2] and Hyun Ko[3]

SUMMARY

Seismic loads are reduced in current design codes for building structures using response modification factors which depend on the ductility capacity and overstrength of a structural system. Seismic design codes are mainly based on the research results for high seismicity regions. However, the overstrength of structures may be larger in low seismicity regions since the ratio of gravity or wind loads to seismic loads may depend on the level of seismicity of a region. Therefore, it is necessary to verify if the response modification factor based on high seismicity would be applicable to low seismicity regions. In this study, the adequacy of the response modification factor is verified based on the ductility and overstrength of building structures estimated from the force-displacement relationship. For the same response modification factor, the ductility demand in low seismicity regions may be smaller than that for high seismicity regions because the overstrength of structures may be larger in low seismicity regions. The ductility demands in example structures designed to UBC97 for high, moderate and low seismicity regions were compared. Demands of plastic rotation in connections were much lower in low seismicity regions compared to those of high seismicity regions when the structures are designed with the same response modification factor. Therefore, in low seismicity regions, it would be not required to use connection details with large ductility capacity even for structures designed with a large response modification factor.

Keywords: *response modification factor, overstrength, system ductility.*

INTRODUCTION

Earthquake resistance design codes such as UBC97 and IBC 2000 classifies moment resisting frames into three structural systems such as ordinary moment resisting frame(OMRF), intermediate moment resisting frame(IMRF) and special moment resisting frame(SMRF) according to their ductility capacity and corresponding response modification factors are used to reduce seismic loads. Since larger elastic deformations are expected, OMRF and IMRF are prohibited in high seismicity regions while OMRF is not allowed in moderate seismicity regions.

The response modification factor mainly depends on overstrength and ductility capacity of structures. Overstrength of structures designed to smaller seismic loads would be larger because structures are designed to resist dead loads and wind loads as well as seismic loads.

Jain performed extensive study on example structures and concluded that overstregth of frames in lower seismicity regions is larger than those for high seismicity regions. Meli showed that the available overstrength varies widely depending on the type of structure and characteristics of ground motion. In UBC97, it is recommended to use larger response modification factor for structures in low seismic regions. Footnote 6 of Table 16-N reads “Ordinary moment resisting frames in Seismic Zone 1 meeting the requirements of Section

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EVALUATION OF HIGHER MODE COMPONENTS IN STORY SHEAR RESPONSE OF BUILDINGS UNDER EARTHQUAKE MOTIONS

Hiroshi KURAMOTO [1]

SUMMARY

This paper shows two methods of reducing a multi-story building to an equivalent single degree of freedom system (ESDOF) in order to improve the capacity spectrum method used for Calculation of Response and Limit Strength provided in the Building Standard Law of Japan. The first is a modal adaptive pushover analysis method, which uses a stiffness-dependent lateral force distribution at each loading step without the need for eigenvalue modal analysis, and the second method uses a modal decomposition procedure together with earthquake response analysis. Applying both methods on 4- and 12-story RC buildings, the validity of the two methods and the earthquake response characteristics of ESDOF systems are examined. An evaluation procedure for higher mode effects in story shear responses is also proposed based on the latter method.

Keywords*: Calculation of response and limit strength, Equivalent single degree of freedom system, Higher mode response, Modal adaptive static pushover analysis, Earthquake response analysis.*

INTRODUCTION

In Calculation of Response and Limit Strength (CRLS) implemented in June 2000 in accordance with the Building Standard Law of Japan (June 1998), Capacity Spectrum Method (CSM; Freeman 1978), founded on the concept of equivalent linearization (Shibata 1976 and 1981) of building models, has been adopted for calculating a building's earthquake response. As commonly known, this method evaluates the peak response values during an earthquake at each floor, or each structural member, of a multi-story building, based on the peak response values of the equivalent single degree of freedom (ESDOF) system taken to represent its structural properties. Thus, a key concern when using this method is on how to suitably reduce a building model into an SDOF system (Kuramoto 2002). At the same time, since a response evaluation based on an ESDOF system does not take into account the higher modes and their amplifying effects on member stresses and deformations, the means to evaluate these effects is another critical consideration related to an ESDOF system reduction method.

With the objective of improving the response evaluation method for superstructures under CRLS, this paper proposes extensions of the methods described by Kuramoto et al. (2001), developed for a more rational ESDOF system reduction method and evaluation procedure for amplifications in response arising from higher modes. More specifically, two ESDOF system reduction methods are described herein: one using nonlinear modal adaptive pushover analysis (Kuramoto 2004) which can take into account the changes in mode shapes associated with yielding of the structure (hereafter abbreviated to *static reduction method*), and another based on modal decomposition utilizing time history earthquake response analysis results of multiple degree of freedom (MDOF) systems (hereafter abbreviated to *dynamic reduction method*). Both methods are applied on a 4-story and a 12-story reinforced concrete (RC) structural frame. Then, together with examining the earthquake response characteristics of their ESDOF systems by comparing the results from the static and the dynamic reduction, the validity of ESDOF system reduction for CRLS is demonstrated. An evaluation procedure for higher mode components in the story shear force time histories, using the dynamic reduction results, is also presented.

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Equivalent Single-Mass and Tri-Degree of Freedom System in consideration of Eccentricity in Strong Column Type Steel Moment Frames

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SUMMARY

This research is concerned with a seismic design procedure to estimate ductility demand of beams in strong column-&-weak beam type steel moment frames with eccentricity. This paper deals with equivalent single-mass and tri-degree of freedom system, which is based on equilibrium between input energy by an earthquake and absorption energy of a frame. The main objectives to use this system are as follows: (1) to characterize some parameters that control inelastic earthquake responses of these frames, and (2) to obtain approximate earthquake responses through time history analysis, where earthquake responses are maximum plastic rotation and cumulative plastic rotation of beams. In this paper, firstly, the single-mass and tri-degree of freedom system (TDOF), which is based on some assumptions in consideration of dynamic property of steel moment frames with eccentricity, is established. The point to be specially mentioned is the application of two kinds of yield surfaces in order to obtain maximum plastic rotation and cumulative plastic rotation of beams. One of the yield surfaces means initial yield of the frames and another means mechanisms of the frames by the effect of lateral-torsional coupling. Next, it discusses whether TDOF is workable by comparison of earthquake responses between TDOF and space frame model using fishbone-shaped frames. The parameters of numerical analysis are amount of eccentricity, type of eccentricity, number of story, aspect ratio of floor plan, and so on. As a result, it is found that utilization of TDOF for two objectives mentioned above is valid.

Keywords: *Eccentricity; Steel moment frame; Plastic rotation; TDOF; Numerical analysis*

INTRODUCTION

When a lateral force caused by an earthquake acts on a structure with eccentricity, lateral-torsional coupling deformation occurs. By torsional deformation, plastic deformation of members in outer plane frame is larger than a structure without eccentricity. The purpose of this research is development of a method to estimate plastic deformation of members in strong column-&-weak beam type steel moment frames in consideration of eccentricity. Single-degree of freedom (SDOF) based on the equivalent linear method is one of the methods to make a rough estimate of maximum earthquake responses. In the case of steel structures, however, this method isn't suitable for the purpose because it is important to obtain not only maximum plastic deformation but also cumulative plastic deformation. Single-degree of freedom based on equilibrium between input energy by an earthquake and absorption energy of a frame has been proposed, and a method to estimate plastic deformation of members in a frame without eccentricity has been developed (Ogawa et al. 2000 [1)]).

In this paper, a system to consider three dimensional freedom into single-degree of freedom based on the equilibrium of energy is proposed. The system is called equivalent single-mass and tri-degree of freedom system (TDOF), and the main objectives to use this system are as follows: (1) to characterize some parameters that control inelastic

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EQUIVALENT LINEAR SYSTEM FOR ESTIMATING MAXIMUM INELASTIC DISPLACEMENT DEMANDS

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SUMMARY

The displacement demand is of primary importance in displacement-based earthquake resistant design. Equivalent linear system (ELS) approach is commonly used to evaluate the maximum displacement of inelastic systems. Formulas for equivalent period and damping ratio of ELS are proposed for the bilinear hysteretic system in this study. These formulas are calibrated using 225 strong earthquake motions by making use of optimization techniques. Let *r* be the ratio of maximum displacements between the equivalent linear and inelastic systems. The objective is to minimize the standard deviation of *r* subjected to the mean value of *r* equals 1. The effective period and damping are expressed using analytical functions of the post-yield stiffness ratio and ductility ratio. The accuracy of the obtained formulas is verified using another 225 strong earthquake motions. It is found that the mean error is in the range of –5% to –5% except for period less than 0.5 sec. The standard deviation depends on the post-yield stiffness ratio α. The largest value is in the range of 25% to 35% when α=0. The value decreases when α increases. For example, the value is in the range of 15% to 20% when $\alpha = 0.2$. Finally, the proposed formulas are accurate than those in FEMA 440 in most cases. Also, they are much general since the fitting coefficients in FEMA 440 are given only for $\alpha = 0$, 0.02, 0.05, 0.1, and 0.2.

***Keywords**: Equivalent linear system, equivalent period, equivalent damping ratio, inelastic system.*

INTRODUCTION

Recently, displacement-based design approaches have been proposed to replace force-based design approaches for seismic design. It is very important to calculate the maximum displacement since the displacement demand is of primary importance in displacement-based earthquake resistant design and also in damage control of structures. Most structures will behave inelastically when subjected to strong earthquake motions. Therefore, how to estimate accurately the maximum displacement for an inelastic system is very important.

It is not difficult to carry out an inelastic analysis for a SDOF or MDOF system using available commercial packages. However, this kind of analysis is still time-consuming and also needs to be carried out for each input ground motions that are of interest. Moreover, how to define the interested input ground motions may not be an easy task. To overcome the above drawbacks, an equivalent linearization method has been proposed [e.g. Miranda and Ruiz-Garcia 2002]. The main idea of the method is to replace an inelastic SDOF system with an equivalent SDOF linear system. To define the equivalent linear system, formulas for calculating the equivalent damping ratio and period are needed. These formulas are often calibrated from analyses of many strong earthquake motions. For example, the secant stiffness method adopts the secant stiffness at maximum displacement to calculate the equivalent period and the damping ratio proposed by Jacobsen. The equivalent period and damping ratio can be derived analytically with the results of

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An Overall Structural Analysis of Steel Frame Collaborating Composite Beam Analysis Program and Exposed Column Base Analysis Program

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SUMMARY

A collaborative structural analysis (CSA) system is proposed which utilizes beneficial features of existing individual structural analysis programs to perform highly sophisticated structural analyses. In the CSA system, the equilibrium equation of the overall structural system is constructed in the host program running on a host computer, while critical parts of the structure are analyzed separately with enhanced detail and accuracy in station programs running on station computers. As the overall analysis progresses step by step, the necessary information, such as the displacement vectors, restoring force vectors, and stiffness matrices, are exchanged between the different programs via the internet.
In order to study the effectiveness of the CSA system, pushover analyses and seismic time history analyses of a steel moment-resisting frame were performed. The overall frame was analyzed by a host program named NETLYS, the composite beams were simulated by a station program named COMPO, and the exposed column bases were simulated by another station program named NETBASE. The proposed CSA system contributes to obtaining detailed and accurate behavior of critical parts of the structure while simultaneously incorporating that local behavior to the overall structural behavior.

Keywords: Internet, Collaboration, Numerical Analysis, Pushover Analysis, Earthquake Response Time History Analysis

INTRODUCTION

Tada et al. have proposed a collaborative structural analysis (CSA) system which utilizes beneficial features of existing individual structural analysis programs to perform highly sophisticated structural analyses (Tada et al. 2003, 2004a, 2004b, 2004c, Ohgami et al. 2004, Tamai et al. 2004). In the CSA system, the equilibrium equation of the overall structural system is constructed in the host program running on a host computer, while critical parts of the structure are analyzed separately with enhanced detail and accuracy in station programs running on station computers. As the overall analysis progresses step by step, the necessary information, such as the displacement vectors, restoring force vectors, and stiffness matrices, are exchanged between the different programs via the internet.

The proposed CSA system has the following characteristics:
1) The system enables collaboration among researchers by linking their programs through the internet. Participants to each project can be easily assembled according to the nature of the project.
2) Since the program developers need not share the source code or libraries to participate in a CSA project, copyrights owned by the program developers are securely protected.
3) It is easy to join a CSA project, since only minor modifications associated with input and output routines are needed.
4) Data exchange via the internet is easily accomplished because the CSA system uses the file and folder sharing protocol implemented in the operating system and a few FORTRAN77 commands.
5) Since a program operator cannot see analysis conducted in other stations, collaboration between all participating operators is essential in interpreting analysis results. On the other hand, if such collaboration is done, the programs are not used as black boxes.
6) It is preferable that the developers of each participating program join the CSA project to correctly modify the I/O routines and to study the analysis results. As stated above, interpretation of the analysis results rely on collaboration between participants. In other words, participation of commercial FEM programs is generally difficult because their source codes are closed, modification of the I/O routines is difficult, and they are often

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DEFORMATION DEPENDENT STRUT-AND-TIE MODELS FOR REINFORCED CONCRETE COUPLING BEAMS

Sung-Gul HONG[1] Sang-Ki JANG[2]

SUMMARY

Mostly the ductility of reinforced concrete structures is controlled by the ultimate strain of concrete in compression. Strength and deformability of disturbed regions are explained by proposed deformation based strut-and-tie models. Indeterminate STM are interpreted by direct and indirect force transfers. Direct transfer corresponds to arch action indirect force transfer by truss action relying on bond. The role of bond force along ties is investigated for transformation of an initial STM into another one as the deformation increases. This paper presents a deformable strut-and-tie model and a mechanism-based model to determine the shear strengths and ultimate deformations of reinforced concrete coupling beams. The proposed models based on strut-and-tie model concepts satisfies equilibrium, compatibility, constitutive laws, and the geometric conditions of shear deformation. Experimental programs for deep coupled beams explain how the deformation controls primary force transfer mechanisms and thereby selection of appropriate STM.

Keywords*: coupling beams; shear-dominant members; strut-and-tie models; mechanism-based model; strength degradation; yielding deformation; ultimate formation.*

INTRODUCTION

The ductility of reinforced concrete members mostly depends on ultimate strain of concrete in compression. Current design practices rely on formation of plastic hinge of flexural members for ductile behavior of systems [Park and Paulay 1972]. To guarantee and increase deformability of concrete in compression reinforcement details for confinement have been focused on. Along this tendency most design provisions suppress premature shear failure before flexural failure as much as possible [ACI 2002]. However, inevitable shear strength controlled members and shear related behavior cannot be avoided such as deep beams and joints which play important role to transfer forces. Since most ductility of such members and components show much smaller than those of flexural members so called limited ductility concepts have been considered for seismic design practices [Paulay and Preistley 1992].

Strut-and-tie models (STMs) for disturbed regions have served rational design tools for dimensioning and detailing of reinforced concrete members. Truss-like behavior in disturbed regions at ultimate has been explained by STMs. Current strut-and-tie models focus on their strength only because stress fields of a STM is based on the lower bound theorem. To extend their application STMs for deformation have been proposed by postulating force-deformation relationships for ties and struts[CEB 1993; Hwang and Lee 2000; Lee and Hong 2003]. However, there have been still questions why struts in flexural compression show larger ductile behavior than diagonal struts for shear. Deformation limits of disturbed region or shear strength controlled members need rational models for explanation.

This paper discusses transformation of STM depending on the degree of deformation and interpretation of indeterminate STM based on compatibility approach. For comparison with the proposed models a series of results from experimental program of deep coupled beams are investigated.

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PROGRESS ON MANAGEMENT OF EARTHQUAKE ENGINEERING EXPERIMENTAL DATA AT NCREE

Shang-Hsien HSIEH[1], Wen-Hsiang TU[2], Yuan-Sen YANG[3], and Wei-Chung CHENG[4]

SUMMARY

This paper reports the progress of an effort at National Center for Research on Earthquake Engineering (NCREE) on management of earthquake engineering experimental data. In this research effort, a draft data model designed for modeling typical experiments conducted in NCREE is proposed. The relational database technology is then used to realize the proposed data model, while the XML technology is employed to facilitate data sharing and exchange. A set of web-based user interfaces is designed and implemented for inputting and browsing the experimental data into and from the database. In addition, experimental data of both a shake table experiment and a pseudo-dynamic experiment are selected and being employed to verify the sufficiency and effectiveness of the proposed data model and the user interfaces.

Keywords: *Earthquake engineering; Experimental data; Data model; Relational database; XML*

INTRODUCTION

It usually takes a lot of time, money, and manpower to complete a structural experiment in an earthquake engineering laboratory such as National Center for Research on Earthquake Engineering (NCREE). Therefore, there should be no doubt that the experimental data resulted from structural experiments are valuable assets of the earthquake engineering laboratory. Although all of the experiments conducted by researchers and the data obtained from them are documented and stored in some way, it is often difficult for other researchers or even the original researchers after some period of time to be able to retrieve and reuse the experimental data. Moreover, rapid advancement of network technology and increasing opportunities for international collaboration have stimulated more and more demands on sharing experimental data in the earthquake engineering community.

To address the issues about the archive, reuse, and sharing of experimental data, an effort is currently in progress at NCREE toward the development of an experimental data management system. The present focus of the effort is on the development of a good data model that can capture sufficient information for future reuse of experimental data and is effective enough to support data management for the experimental data management system. Figure 1 shows the development process of the NCREE experimental data model. The process is an incremental and iterative one. It starts from the Requirement Analysis phase, and then goes into an iterative process that consists of Data Model Design, Test Environment Implementation, and Sufficiency & Effectiveness Test phases, finally completes the development of the NCREE experimental data model. The rest of this paper describe the tasks involved in each phase of the development process and reports the progress that has been made.

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DESIGN AND ANALYTICAL ASSESSMENT OF A BRBF SPECIMEN FOR FULL-SCALE 3D SUB-STRUCTURAL PSEUDO-DYNAMIC TESTING

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SUMMARY

In this paper, the analytical studies of a 2-story buckling-restrained-bracing frame (BRBF) to be tested using sub-structural pseudo-dynamic testing (PDT) procedures are described. The seismic ground accelerations considered in this study was recorded in the 1999 Chi-Chi earthquake. The ground accelerations are scaled up to represent 50%, 10%, and 2% probability of exceedance in 50 years and bi-directionally applied in the transverse and longitudinal directions simultaneously. The displacement-based seismic design procedures used in the design of the BRBF is presented. A systematic approach to determine the earthquake scenario for the test is developed. For the first phase of the test, analytical results suggest that the peak story drift is likely to reach 0.025 radian in the transverse direction under the 2/50 exceeding-probability earthquake. It also shows that the allocation of actuators in the BRBF direction might be adequate. For the second phase of the test, the analytical model is adjusted to form three planar asymmetric structures. These three asymmetric structures are coupled in translational and rotational motions with different extent. One of the objectives of the Phase II test is to validate the MPA procedures in estimating the seismic demands of asymmetric structures. The analytical results obtained by MPA procedures and response history analysis (RHA) are compared.

***Keywords**: PDT, earthquake scenario, BRBF, displacement-based seismic design, RHA*

INTRODUCTION

In March 2005, sub-structural pseudo-dynamic tests of a full-scale steel buckling restrained braced frame (BRBF) with internet testing techniques were conducted at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. The prototype of the 2-story building configuration and the BRBF specimen is shown in Fig. 1. In the context of the pseudo dynamic testing, only one BRB frame specimen was tested, the remaining structure was simulated analytically. The prototype structure is located at seismic zone I, which has strong seismic intensity according to Taiwan Building Regulations. The bay width and story height of the BRBF are 8m and 4m, respectively. This experimental program consists of two phases. In Phase I, the seismic ground motion records, which was recorded in the 1999 Chi-Chi earthquake, are scaled up to represent 50%, 10%, and 2% probability of exceedance in 50 years (denoted as 50/50, 10/50 and 2/50 events, respectively). The ground motions were applied bi-directionally in the transverse (Y-axis) and longitudinal (X-axis) directions. One of the test objectives in Phase I is to observe the performance of various BRB to gusset plate connections (Fig. 2) under the bi-directional seismic load effects (Tsai et al., 2005). In Phase II, the prototype structure is to be adjusted to form three asymmetric structures, which have the common experimental BRB frame on Frame Line B. The objective in Phase II is to verify the validity of the η-MPA procedure to estimate the seismic demands of asymmetric structures. This experiment also provides great opportunities to further enhance the networked pseudo-dynamic test and data archiving techniques envisioned for the Internet-based Simulations for Earthquake

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PROPOSED SITE CLASS DEFINITIONS FOR THE DEVELPING SEISMIC DESIGN CODE CONSIDERING SI UNIT SYSTEM AND INTERPOLATION OF SITE COEFFICIENTS

Yong-Seok KIM[1]

SUMMARY

UBC and IBC adopted the soil class definitions for the seismic analysis originally developed by NEHRP. However, this site class definitions are based on the USCS(ft-kips) unit system and is not friendly for the interpolation of the site coefficients due to the unclear relationship between a site class and site coefficients. A site class is defined by the range of the soil properties, but not by a single site coefficient for each site category. The site class definitions of IBC have also too wide range of soil properties for each site class. It makes the structural engineers difficult to estimate the site coefficients for the diverse soil layers.
In this study, the site class definitions were modified based on the SI unit system considering the 30m soil layer below the foundation for the shallow embedded foundation, and the interpolation of the site coefficients was investigated using the original equations to calculate the site coefficients and comparing them with those of IBC. Also equations to evaluate the site coefficients were modified adjusting the reference shear wave velocity of site class B and curve-fitting them by the least squares method.
With the study results, new site class definitions in SI unit system were suggested to make the interpolation of the site coefficients easy for the diverse site shear wave velocities and the different level of seismic shaking intensities, modifying IBC site class definitions and calculating the site coefficients of Fa and Fv with the same shear wave velocity. This proposed site class definitions can be used to prepare the next generation of seismic design codes.

Keywords: *IBC, site class definitions, site coefficients, interpolation, SI unit system, next generation of seismic design code*

INTRODUCTION

Soil class definitions adopted for the seismic analysis in UBC and IBC were originally developed by NEHRP, and evolved for the last two decades. UBC-84 introduced the soil classes defined in 3 soil conditions considering the soil depth up to 60m. After 1985 Mexico City earthquake and 1989 Loma Prieta earthquake, UBC-94 modified the soil classification redefining the soil classes into 4 soil conditions adding the soft soil condition. However, the new soil class definitions were not well-defined for the practical point of view. The soil class can be classified differently depending on the judgment of an engineer.
In 1994, NEHRP updated the site classification more clearly and redefined the soil profile type as 6 soil conditions of A through F as shown in Table 1, characterizing the site soil by shear wave velocity, N-value and undrained shear strength. The soil properties were calculated averaging those of the upper 30m soil layers. And UBC-97 and IBC2000 adopted the site class definitions of NEHRP-94.

But, the updated site class definitions introduced in UBC and IBC have still some problems. The site class definitions based on the ft-kips unit system is not friendly to and familiar with the SI system users making it difficult to correlate the soil properties with those in SI unit system. And, the interpolation of site coefficients of

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SHAKING TABLE TESTING METHOD CONSIDERING DYNAMIC SOIL-STRUCTURE INTERACTION

Sung-Kyung LEE[1], Masato MOTOSAKA[2], and Kyung-Won MIN[3]

SUMMARY

This paper proposes the shaking table testing method, without any soil specimen only using building model as an experimental part, considering dynamic soil-structure interaction based on the substructure method. Two-layered soil is assumed as a soil model of the entire soil-structure interaction (SSI) system in this paper. The proposed acceleration and velocity feedback methods are those that the shaking table is driven by the motion corresponding to the acceleration and velocity at foundation of the total SSI system for the acceleration and velocity feedback methods, respectively. For the experimental implementation of the proposed methodologies, analytical frequency-dependent dynamic soil stiffness is approximately considered for the case of both acceleration and velocity feedback, respectively, and then they are incorporated in control computer. In the experimental verification of the proposed methods, only building specimen is mounted on shaking table, and then it is excited by shaking table with the motion of the acceleration or velocity, required to replicate the behavior of total soil-structure interaction system, which is produced by indirectly measuring the acceleration- or velocity-formulated interaction force from the fed-back accelerations of superstructure and by passing it through the filters corresponding to the approximated dynamic soil stiffness. Experimental results show that the proposed methodologies can be successfully applied to the shaking table test considering dynamic soil-structure interaction with good accuracy.

***Keywords**: shaking table testing method; soil-structure interaction; substructure method; acceleration feedback; velocity feedback.*

INTRODUCTION

It has widely been recognized for structural engineers that the grasping of the dynamic vibration characteristics of soil-structure interaction(SSI) system subjected to seismic loading is required to adequately evaluate the structural response under earthquake loads.

For this purpose, the application of substructure method to the numerical evaluation of dynamic soil-structure interaction system gives less calculating time than Finite Element Method. With the endeavor to calculate the non-linear superstructure on unbounded linear soil media, one of authors has applied this method to the numerical evaluation of large nuclear power plant, which contains many vibration-sensible devices[Motosaka et al, 1992, 1993].

The concept of substructure method has recently applied to the development of testing techniques. Iemura's group has applied this method to the shaking table test including the vibration control device, to verify its control efficacy. In his method, the experimental part is the vibration control device and the computing part is the building or bridge structure[Iemura et al, 2002], [Igarashi et al, 2002]. Meanwhile, Konagai has carried out the shaking table test on the SSI system as an application of substructure method[Konagai et al, 1998]. In his method, the experimental part is the superstructure and the computing part is the soil model. A force sensor, installed between the bottom of superstructure's specimen and the shaking table, was used for observation of

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EXPERIMENTAL EVALUATION OF POST-TENSIONED STEEL CONNECTIONS WITH STEEL BARS AND A DISCONTINUOUS SLAB

Chung-Che CHOU[1], Wen-Chia Yang[2], and Keh-Chyuan Tsai[3]

SUMMARY

Cyclic tests were conducted on three full-scale subassemblies to investigate the behavior of post-tensioned steel beam-to-column connections. Strands were placed along each side of the steel beam web, passing through the steel column to provide a precompression between the beams and column. The connection used top and bottom steel bars to increase the moment capacity and energy dissipation capacity. One of the subassemblies had a concrete slab with discontinuity at the centerline of the column. Test results showed that (1) the steel beam could reach 4% interstory drift without strength degradation, (2) the buckling-restrained steel bars were effective in dissipating energy after a gap opening at the interface between the beam and column, (3) the residual forces in the bars gradually reduced the decompression moment of the connection, and (4) discontinuity of a slab at the centerline of the column ensured recentering capability of the connection. A simplified analysis procedure was used to perform correlation studies, indicating that the force-deformation relationships of the subassemblies could be reliably predicted when both the effects of the steel bars and post-tensioned strands were considered in the model.

Keywords: *post-tensioned steel beam-to-column connection, strands, energy-dissipating bar, concrete slab*

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. (2001, 2002), Christopoulos et al. (2002), Garlock et al. (2005), and Chou et al. (2004, 2005). The connections in these studies incorporated seat angles, round bars, or reduced flange plates to dissipate energy, and high strength strands or post-tensioned bars to provide a recentering capability. These connections exhibited similar force-deformation behavior and recentering capability. While the use of bolted seat angles is convenient for erection and replacement, modeling of these connections under inelastic cyclic loading may be complex due to geometric considerations (Shen and Astaneh-Asl 2000). The reduced flange plate, subjected to in-plane force, is easy for modeling, but manufacturing steel plates with arc cut-out may be expensive. Results of a cyclic test on an exterior beam-to-column connection with energy-dissipating (ED) bars have showed that this connection is capable of achieving stiffness and strength comparable to a traditional welded moment connection (Christopoulos et al. 2003). No tests on interior beam-to-column connections with ED bars have been performed. Therefore, this paper investigates further the cyclic responses of the post-tensioned interior connections with ED bars and evaluates whether the post-tensioned connection with a discontinuous concrete slab can also recenter in cyclic loading.

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EFFECT OF RESTRAINT AT WELDED FLANGE JOINTS

Masaru TERAOKA[1]

SUMMARY

In this paper, we discussed the effects of geometrical restraints on the elongation capacity of a beam flange based on the results of cruciform tests subjected to static monotonic tensile loading that were conducted for a tensile beam flange-to-column connection, and also on the results of static cyclic loading tests conducted for a beam model of the simple supported type. The elongation capacity of the flange at the beam-end in the beam-to-column connection was investigated by the model tests. The following results were obtained:

(1) The results of static monotonic tensile loading test on a cruciform model of the beam-to-column connection showed that the average strain ($\bar{\varepsilon}_m$) in the beam-end flange at the maximum load ranged 8.4～10.2% (average for all test specimens = 9.5%). The ratios of $\bar{\varepsilon}_m$ to the uniform elongation of the raw material (ε_m =17.9%) were $\bar{\varepsilon}_m / \varepsilon_m$ =0.47～0.57 (average for all test specimens = 0.53). Thus, it was confirmed that elongation capacity of the flange at the beam end significantly decreases due to the restraint effect.

(2) The results of static cyclic loading test on a beam model of the simple supported type showed that the average cumulative tensile strains ($\bar{\varepsilon}_m$) on the fractured beam flange at the maximum load were 4.1～5.6% (average for all test specimens = 4.9%). The ratios of $\bar{\varepsilon}_m$ to the uniform elongation of the raw material (ε_m =18.6%) were $\bar{\varepsilon}_m / \varepsilon_m$ =0.22～0.30 (average for all test specimens = 0.26). Thus, it was confirmed that elongation capacity of the flange at the beam-end decreases to approximately 1/2 that of monotonic tensile loading due to a combined effect of restraint and cyclic loading.

***Keywords**: effect of restraint; welded flange; beam-to-column connection; elongation capacity*

AIM OF EXPERIMENT

The flange of a beam-to-column connection is subject to geometrical restraints at the beam end such as restraint of a plate in the transverse direction, caused by a diaphragm, or restraint of a member in the axis direction, caused by a weld access hole. Due to these restraints, strain is concentrated in the restrained part, which is likely to cause early failure of the beam member, and reduce the deformation ability of the member.

In this paper, we discuss the effects of geometrical restraints on the elongation capacity of a beam flange based on the results of cruciform tests subjected to static monotonic tensile loading that were conducted for a tensile beam flange-to-column connection, and also on the results of static cyclic loading tests conducted for a beam model of the simple supported type.

INVESTIGATION OF ELONGATION CAPACITY OF THE FLANGE AT THE BEAM-END BY CRUCIFORM TENSILE TEST

Outline of test

The test specimens were a cruciform model of the beam flange-to-column connection subjected to tension. A total of 13 test specimens were made for this experiment. Figure 1 shows the shape of the test specimen, and details of the test

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STRENGTH OF RHS-COLUMN TO BEAM CONNECTIONS WITH EXTERIOR DIAPHRAGM

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SUMMARY

This paper addresses the derivation of design formula of RHS-column to beam connections with exterior diaphragm based on the method of limit analysis. In order to verify the validity of the collapse load, tests of RHS-column to beam connections with exterior diaphragm were performed. Major findings from the tests are summarized as follows: (1) the yield strength obtained from the tests was about 70~80% of the collapse loads, (2) there were 13~37% differences between the experimental maximum strength and the calculated maximum strength, and (3) the assumed collapse mechanisms agreed well with the ultimate states of the connections.

Keywords *: exterior diaphragm ; RHS-column ; limit analysis ; collapse mechanism ; fillet weld ; tensile test*

INTRODUCTION

In Japan, rectangular hollow sections (RHS) or box-sections are mainly used as columns of steel buildings.Three typical types of beam-to-column connections, namely the through-diaphragm connection, the interior diaphragm connection, and the exterior diaphragm connection, are adopted in order to transmit the stress of beam flanges to columns. Among these connections, the exterior diaphragm has its advantages because both manufacturing and welding processes can be reduced.

Tabuchi et al. examined the local failure of welded RHS-column to beam connections with exterior diaphragm (hereafter, denoted as connections with exterior diaphragm) shown in Fig.1(b), and the empirical formula predicting the local strength of connections with exterior diaphragm are derived by the dimensional analysis and the regression analysis (Ref.1). The empirical formula is described as the design formula in the present Japanese design guideline (Ref.2). However, the applicable scope of the design formula is restricted with regard to shapes in connections with exterior diaphragm. The design method of weld zone between RHS-column and exterior diaphragm is not described in the guideline. It is necessary to investigate these points. Under the consideration described above, we have proposed four types of collapse mechanisms and derived collapse loads with regard to connections with exterior diaphragm shown in Fig.1(a) (Ref.3,4). When diaphragm depth (h_d) is large or beam flange width (B_f) is smaller than diaphragm width (B_d), effects of these dimensions on the internal work of column tube wall can be estimated, but not exactly.

In this paper, we propose new collapse mechanism and derive the design formula of connections with exterior

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Summarizing Force Transfer Mechanism of Welded Steel Moment Connections

Cheol-Ho LEE[1]

SUMMARY

Employing the classical beam theory for the design of welded steel moment connections has been brought into question since the 1994 Northridge earthquake. In this study, the force transfer mechanism in various welded steel moment connections is comprehensively reviewed based mainly on the author's recent studies. Available analytical and experimental results showed that the load path in almost all the welded steel moment connections is completely different from that predicted by the classical beam theory. Vertical plates near the connection such as the beam web, the web of the straight haunch, and the rib act as a strut to some extent. The force transfer mechanism in the RBS connection is essentially the same as that in the pre-Northridge type connections. Some simplified analytical models that can be used as the basis of a practical design procedure are also briefly presented.
Keywords*: Connections; Steel moment frames; Seismic design; Load path; Strut action.*

INTRODUCTION

The 1994 Northridge and the 1995 Kobe earthquakes caused widespread damages in the connections of steel moment-resisting frames. This type of framing system was once thought to be one of the premium systems for seismic resistance. In response to the unexpected damage, extensive researches were conducted to find ways to repair damaged steel moment frames as well as to strengthen existing and new steel construction. A variety of new ideas have been proposed in the US (SAC 2000). Steel moment connections have been commonly designed by employing the classical beam theory which is based on the Bernoulli-Euler hypothesis, or the assumption that cross sections of a beam in bending remain plane. However, the application of the classical beam theory in the design of steel moment connections has been brought into question after the Northridge earthquake. For example, the author's previous work on the triangular haunch reinforced connection (Lee and Uang 1997) found that the neutral axis does not shift in spite of the addition of a haunch at the bottom of the beam. The validity of the universally-used shear formula in connection design was seriously questioned (Popov et al. 1997). Goel et al. (1997) promoted the use of truss analogy for welded steel moment connection design.
The primary objective of this study was to review the force transfer mechanism of a variety of welded steel moment connections from the available analytical and experimental results and to show that the load path in almost all the welded steel moment connections is completely different from that predicted by the classical beam theory.

SHEAR TRANSFER FROM CLASSICAL BEAM THEORY

Two basic formulas for bending and shear design derived from the classical beam theory are given in Eqs. (1) and (2) (Gere and Timoshenko 1984). As shown in Fig. 1, the shear stress distribution in the web of a H-shaped section can be obtained by Eq. (2). In practical design, the maximum shear stress is approximately computed by dividing the total shear force by the area of the web (see Eq. 3). For H-shaped beams of typical proportions, the average value of shear stress as computed by Eq. (3) is within 10% of the actual maximum value that occurs at the neutral axis location.

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INELASTIC ANALYSIS OF REDUCED BEAM SECTION STEEL MOMENT CONNECTIONS TO BUILT-UP BOX COLUMNS

Taejin KIM[1], Jun-Hee PARK[2], and Cheol-Ho LEE[3]

SUMMARY

This paper presents results of analytical study on the seismic responses of RBS steel moment connections between am H-shaped beam and a built-up box column. Two analysis models of the connections were selected from the test specimens used in previous research. These two analysis models used one beam size (H-700×300×13×24) and two columns sizes, which were a box column (B-400×400×20) and an H-shaped column (H-428×407×20×35), respectively. The box column had a square cross section fabricated from four column plates and interior horizontal diaphragms at the levels of beam flanges. The beam flanges were partially removed at some distance from the column face, so that the beam plastic hinge develops at the location far from the weld joints. Inelastic analyses of the connection models were conducted using general finite element program, ABAQUS. It was verified the effectiveness of the RBS details on the performance of box column connections by investigating the seismic responses of both analysis models.

***Keywords**: Built-up box columns; steel moment connections; seismic behavior; reduced beam section; finite element analysis.*

INTRODUCTION

The fully restrained steel moment connection joining beams and a column is the important structural component in steel moment frames. Moments and shears developed in the beam can be transferred to the column by the rigid action of the connection when the steel moment frame moves laterally. Although H-shaped sections are usually preferred to the column of the connection, box sections are also often used for the column. The box column offers two principal advantages over the H-shaped column. First, box columns can be designed to have no significantly weaker axis to avoid problems in designing corner columns in moment frame structures. Second, cross sections of box columns are closed, making their torsional stiffness and resistance much higher than those of H-shaped columns.

After 1994 Northridge Earthquake in United States and 1995 Kobe Earthquake in Japan, many questions have arisen on the reliability of steel moment connections for the seismic application. For a decade extensive researches have been conducted to develop strategies that can achieve the performance level required in the seismic code. Practical design guidelines, published in a series of FEMA documents, gave designers new tools to design special steel moment frames and provided a portfolio of new connection solutions (FEMA 2000). Satisfactory seismic behavior of the new connections was proven in a comprehensive series of pre-qualification tests. However, such connections were prequalified only for the connections between H-shaped beams and H-shaped columns, but the box column connections were not considered.

The box column connection also can be suffered from brittle damage in the joint as the connection to an

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