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EXPERIMENTAL INVESTIGATIONS OF THE LATERAL FORCE STRENGTHENING FOR LOW-RISE SOFT STORY

Sang-Hoon SEO[1], Youn-Jong YOO[1], Heecheul KIM[2], Young-Hak LEE[3], Han-Seon LEE[4] and Kihak LEE[5]

SUMMARY

An experimental research has been accomplished to investigate the lateral force strengthening effect of low-rise concrete building having soft story at the first floor. The columns on the first floor were reinforced with aramid fiber wrapping and both aramid fiber and buckling restrained brace(BRB). Three different one-story frames having 300 ㎜×400 ㎜ columns and 350mm x 400mm were tested and the corresponding lateral force capacity was compared.
The maximum lateral load capacity of the aramid fiber reinforced frame was almost twice larger then that of the un-reinforced frame. The frame that was reinforced with both aramid fiber and buckling restrained brace showed better improved lateral force resisting capacity than those of others. However the different crack pattern was observed for the three different frames. The results obtained from full scale experiment would give appropriate reinforcement guidance for an earthquake resistant low-rise building having soft story at the 1st floor.

Keywords*: aramid fiber; Buckling Restrained Brace(BRB); soft story; low-rise.*

INTRODUCTION

The first earthquake resistant building design code in Korea did not specified earthquake resistant design of buildings lower than 6 stories. Therefore, all low-rise buildings that is lower than 6 stories being built without considering any earthquake load during a design process. As every structural engineer recognize that it is very dangerous during earthquake although the building is only five stories if the first floor of a building stiffness is softer than other stories. The special care should be provided for this type of structure. An efficient and economical reinforcing method in improving lateral capacity became an issue for this type of structure, recently.
The lateral force reinforcing method of pilotis type structure can be classified into two types; 1) strengthening of column, and 2) strengthening of beam-column joint. A column reinforcement using fiber sheet is very common due to its easy construction. Also the reinforcement using fiber sheet has economical and environmental advantages due to its decrement of both labor and wastes.
Typical type of fibers used in reinforcing structural member are; 1)Carbon fiber, 2)Glass fiber, and 3)Aramid fiber. The research on structural reinforcing method with aramid fiber polymer was conducted by some scientists and engineers due to its relatively high ductility and low cost in spite of its relatively low tensile strength compared with other two fibers[1),2)]. A typical column reinforcing method using fiber sheet is the increment of compression and flexural strength through the confinement of all or part of a column using resin impregnated fiber polymer sheet.
Steel braces such as diagonal type, K-type, and X-type were the typical earthquake reinforcing method for the frame structure. However, much restriction follows in applying braces to low-rise soft story building for earthquake reinforcement since most of soft story in first floor is designed for a parking lot. Buckling Restrained

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SEISMIC MOTION-DAMAGE RELATIONSHIP BY USING DAMAGE DATA OF SCHOOL BUILDINGS IN TAIWAN

Yi-Hsuan TU[1], Pei-Lin YEH[2], Tzu-Wei LIU[2], and Wen-Yu JEAN[3]

SUMMARY

A databank for a total of 157 typical low-rise RC school buildings damaged in the 1999 Chi-Chi Earthquake in Taiwan is established. Damage and structural information include original architectural blueprints, ground motion intensity, and damage records are collected by literature survey. Different from most of the earthquake damage investigation, the samples are not normally distributed and therefore regarded as independent events. Bias and uncertainties in the databank are discussed. Since the buildings have high similarity in architectural type and structural system, empirical fitting approach with properly chosen samples is employed for attempt on establishing a motion-damage relationship. Peak ground acceleration (PGA), the ratio of column area at ground floor, and a five-level damage index are used to define ground motion, structure, and damage characteristics, respectively. Although the correlation between them is not as clear as expected, a reasonable tendency is observed.

Keywords: *earthquake damage; school buildings; databank; motion-damage relationship.*

INTRODUCTION

Earthquake damage is not only a disaster, but also an examination for the present structure engineering technique and a lesson for the future one. By collecting and investigating damage data, the correlation between ground motion and damage can be established directly and applied to earthquake loss estimation and hazard mitigation. ATC-13 report (ATC 1985) presented a methodology that has been widely used for estimating damage/loss caused by earthquake and collateral hazards. The major procedure includes facility classification, selection of ground motion characterization, and damage estimates for different types of structures under specific ground shaking intensity.

Mostly the motion-damage relationship developed by using investigated damage data during a past earthquake is in the form of vulnerability function (also termed fragility curve). It predicts the probability of reaching or exceeding specific damage state for a category of buildings with given earthquake intensity. The vulnerability functions are feasible for damage loss estimate in a large region, such as a city or a state, yet scatter caused by insufficiency of detail in structure classification exists generally. The applicability of the functions to regions outside the area where it is originally developed is unclear as well. Seismic evaluation that concerns about localized structural behavior and ground motion characteristics is utilized to establish the motion-damage relationship for individual buildings. While detailed information and complicated calculation are required, higher accuracy and applicability may be expected. Vulnerability functions can also be developed by performing seismic evaluation procedure on representative type of model buildings. Kircher et al. (1997) demonstrated the building damage function developed for the FEMA/NIBS earthquake loss estimation methodology (Whitman et al. 1997) by using the capacity-spectrum method proposed in ATC-40 report (ATC 1996).

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ANALYTICAL SIMULATION OF SHAKE-TABLE RESPONSES OF A TORSIONALLY-ECCENTRIC PILOTI-TYPE HIGH-RISE RC BUILDING MODEL

Han-Seon Lee[1], Jeong-Jae Lee[2], and Dong-Wook Jung[3]

SUMMARY

A series of shaking table tests on a 1:12 scale model using scaled Taft N21E earthquake records were conducted to investigate the seismic performance of a 17-story high-rise reinforced concrete building structure with a high degree of torsional eccentricity and soft-story irregularities in the bottom two stories. The main characteristics of behaviors were: (1) The sudden change of the predominant vibration mode from the mode of translation and torsion to the torsional mode after the flexible side underwent large inelastic deformation. (2) The abrupt increase in the torsional stiffness in this change of modes. (3) The warping behavior of wall in the torsional mode. And (4) the unilateral overturning moment in the direction transverse to the table excitations.

In this study, efforts were devoted to simulate the above characteristics using a nonlinear analysis program, Perform*3D*. The advantages and limitations are presented with the nonlinear models available in this software as they are related to the correlation between analysis and test results.

***Keywords**: SEEBUS; torsional irregularity; reinforced concrete building; shaking table test; nonlinear dynamic analysis.*

INTRODUCTION

Performance-based seismic evaluation and design has become practical with remarkable developments in the experimental and analytic seismic engineering research. In performance-based engineering, the estimation of the available capacity of structural components as well as of the whole structure is crucial to its successful implementation. Also, the demand on component forces and deformations by the expected earthquake ground motions should be reasonably predicted. This prediction or estimation is largely enabled by nonlinear dynamic or static analysis. Current techniques of nonlinear time-history analysis or static analysis are well developed. However, nonlinear analysis should be calibrated whenever possible as analysis reliability has a crucial role in performance-based engineering. The objective of this paper is to investigate the correlation between the results of previously conducted earthquake simulation tests of a 1:12 scale 17-story high-rise RC building model (Dong-Woo Ko et al. 2006) and the nonlinear time-history analysis performed by using Perform*3D* (Computer and Structures Inc. 2006). The reason why the authors have chosen this software is that it has good pre- and post-process and a user-friendly environment, particularly with currently available rehabilitation guides for existing building structures such as FEMA 356 (Federal Emergency Management Agency 2000) or ASCE/SEI 41-06 (American Society of Civil Engineers 2007).

EARTHQUAKE SIMULATION TESTS OF A 1:12 SCALE 17-STORY RC BUILDING MODEL

The prototype was selected based on an inventory study of multi-purpose high-rise buildings in Korea (Han-Seon Lee et al. 1999). The structure consists of a lower 2-story 2-bay×2-bay frame and upper 15-story

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IN-SITU TESTS OF RC SCHOOL BUILDINGS IN TAIWAN FOR SEISMIC RESISTANCE

Shyh-Jiann HWANG[1], Lap-Loi CHUNG[2], Yeong-Kae YEH[3], Wen-Yu JEAN[4] and Wen-Cheng SHEN[5]

SUMMARY

The poor performance of the RC school buildings with substandard reinforcing details had been widely observed during the 1999 Chi-Chi earthquake. Seismic evaluation and retrofit of these numerous vulnerable school buildings is an important societal issue to be resolved. Concerns about the seismic upgrading of the existing school buildings grew considerably and resulted in several programs to identify and mitigate seismic risk. This paper reviews a series of in-situ tests of RC school buildings in Taiwan to identify findings that may be immediately useful to the structural engineers in government and private practice who are working on seismic retrofitting of school buildings.

Keywords: *In-situ tests; RC school buildings; seismic evaluation; seismic retrofit.*

1. INTRODUCTION

A number of buildings in the primary and middle schools in Taiwan have suffered damages of various degrees during the Ruei-Li earthquake (July 17, 1998), Chi-Chi earthquake (September 21, 1999) and Chia-Yi earthquake (October 22, 1999) in the past decade. In the disaster area of Ruei-Li earthquake, many school buildings were severely damaged. The damage modes included the spalling of columns' cover concrete, cracking of core concrete and buckling of longitudinal reinforcement. The seismic resistance of these buildings should have been greatly reduced. These school buildings were on the brink of collapsing. Nevertheless many school buildings were collapsed during Chi-Chi earthquake. According to the report published by Ministry of Education, a total of 786 schools (1958 classrooms) were damaged in Chi-Chi earthquake. Even in Taipei City, which is about 150 km far away from the epicenter, there were 67 school buildings damaged. The total loss of life resulted from the Chi-Chi earthquake almost reached 2,500. If Ruei-Li earthquake were not occurred during the summer break or the Chi-Chi earthquake were not taken place in late night, the total casualties could have been much more.

Based on the damage statistics, a lack of seismic resistance appears to be a common problem in the existing primary and secondary school buildings in Taiwan. Significant casualties and property losses could be resulted from the collapse of these school buildings under strong earthquakes. Furthermore, school buildings might have to be assigned as emergency shelters immediately after a severe earthquake. An urgent need to retrofit the existing school buildings is clear.

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 brick partition walls in the transverse direction (Figure 1). 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, as shown in Figure 2. 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

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DISPLACEMENT-BASED SEISMIC DESIGN FOR PLAN-ASYMMETRIC WALL SYSTEMS

Sung-gul HONG[1] and Taehyu HA[2]

SUMMARY

This paper proposes a displacement-based seismic design methodology for plan-asymmetric wall systems with consideration of inelastic torsional behavior and coupled dynamic effects to overcome the limitation of current force-based design approaches for torsion. Review on current seismic design code provisions and research for torsion requires us how to realistically estimate design parameters such as additional shear forces due to torsion together with target displacement and target rotation angles of systems at multi-levels of deformation, particularly, at inelastic levels. The proposed design method for torsion introduces a concept of displacement eccentricity from static equilibrium in the framework of current displacement-based design approaches. The proposed displacement eccentricity as a value of proportionality relates a system rotation angle and system displacement for an asymmetric system from static equilibrium. Furthermore, the addition of amplified rotation angles to the static rotation angle enables us to consider the effect of mode shapes of an asymmetric system. Trial rotation angle with a target displacement at initial design step is calculated by assuming elastic displacement eccentricity in an elastic state, and then a displacement eccentricity determined from natural frequency and mode shapes based on secant stiffness are updated for inelastic torsion according to the proposed design method. The proposed design method finally satisfies the target lateral displacement and target rotation angle by successive modification of the lateral stiffness distribution.

Keywords: *torsional response, seismic design, displacement-based approach, shear wall systems, design eccentricity*

INTRODUCTION

Plan-asymmetric structures are likely to undergo torsional vibrations in addition to lateral oscillations, when subjected to earthquake ground motions. As a torsional response occurs, excessive stress and/or deformation are concentrated on the flexible side members. These excessive seismic demands for lateral resisting components often result in an unexpected collapse of systems. Several failures in relation to the torsional responses have been reported for past earthquakes (Mitchell et al. 1986, 1990, 1996). Hence, almost seismic design codes have proposed a criterion of torsional design to minimize the torsional response and consequent damages to systems.
Force-based design methods for torsion have focused on additional shear in individual walls by shear due to the eccentricity. The design eccentricity in the current code provisions includes static, dynamic and accidental eccentricity of the plan. Torsional design methods stated in the current seismic code provisions have some limitations in their application. It is well appreciated that a new approach for torsional design may improve the concepts of current design methods. Several researches have been carried out to analyze the torsional response and to develop the appropriate design process. This paper proposes a displacement-based design method for plan asymmetric wall systems to overcome the limitations of current seismic design code provisions.

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EXPERIMENT OF ENHANCING SEISMIC ISOLATION SYSTEMS IN NEAR-FAULT AREAS BY USING MAXWELL-TYPE VISCOUS DAMPER

Lyan-Ywan LU[1], Ming-Hsiang SHIH[1], Ging-Long LIN[2], and Shih-Wei YEH[3]

SUMMARY

Recent studies have revealed that when a sliding isolation system is subjected to a near-fault earthquake that usually contains a pulse-like long-period velocity waveform, the system may suffer from a low-frequency resonance problem that can cause an excessive isolator displacement and degrades the isolation efficiency. In order to mitigate this problem, in this study, an isolation system with supplemental viscous damping is investigated experimentally. A hybrid isolation system that consists of a long-stroke fluid damper and a set of FPS (friction pendulum system) isolation bearings was tested in a shaking table test program. An element test performed on the fluid viscous damper shows the hysteretic property of the damper can be well characterized by the Maxwell's viscous model. Acceleration records that contain long-period pulse-like waveform were used as the input ground motions in the shaking table test. The experimental result indicated that adding supplemental viscous damping in the sliding isolation system can effectively suppress the isolator displacement and mitigate the resonant effect due to a near-fault earthquake, without increasing the structural acceleration response.

Keywords: *Seismic isolation, Near-fault earthquake, Viscous damper, Maxwell model, Sliding isolation, Shaking table test.*

INTRODUCTION

The technology of seismic isolation has been developed for protecting civil engineering structures from seismic hazard for decades [1]. Some of isolated structures were even subjected to real-life earthquakes and proved that the isolation technology is effective in mitigating structural seismic responses [2, 3]. Nevertheless, there are very few structures actually subjected to near-fault earthquakes that possess very different wave characteristics from those of usual (far-field) earthquakes. There are relatively few experimental studies on the dynamic responses of near-fault seismic isolation.

The literature has shown that near-fault earthquakes usually have the following features [4, 5]: (1) a high level of peak ground acceleration, (2) a large vertical ground motion, (3) a long-period pulse-like waveform, especially in velocity field. The pulse-wave component, whose pulse period usually ranges from 2 - 5 seconds, can be very destructive to structural systems of long vibration periods, such as high-rise buildings and base isolated structures. Recent studies have revealed that when a sliding isolation system is subjected to a near-fault earthquake that usually contains a pulse-like long-period velocity waveform, the system may suffer from a low-frequency resonance problem that can cause an excessive isolator displacement and degrades the isolation efficiency [6, 7, 8].

In order to prevent the excessive isolation response induced by the long-period pulse velocity waveform in

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CYCLIC LOADING TEST OF FRICTION-TYPE REINFORCING MEMBERS UPGRADING WIND-RESISTANT PERFORMANCE OF TRANSMISSION TOWERS

Ji-Hun Park[1] , Byoung-Wook Moon[2] , Kyung-Won Min[3], Sung-Kyung Lee[4]

SUMMARY

Two types of friction-type reinforcing members (FRM) are proposed for the purpose of upgrading wind resistant performance of a transmission tower and verified through cyclic loading tests. First, a suitable installation scheme of the FRM is investigated through numerical analysis. Tower-leg-reinforcing type and brace type installation schemes are examined, and numerical analysis shows that the latter is more effective due to the vertical cantilever type behavior of the transmission tower. Based on this result, two types of the FRMs, dissipating energy in slotted bolted connections, are proposed. The one utilizes the relative displacement between the FRM and the tower leg, and the other utilizes that between the separated angles consisting of the FRM as a slip deformation of the slotted bolted connection. Proposed FRMs are installed on each tower leg of the 1/2 scale substructure models of an actual transmission tower body. From cyclic loading tests, the latter type of the proposed FRMs dissipates energy more effectively and is less sensitive to the work tolerance and initial deformation.

Keywords: *friction damper, slotted bolted connection, transmission tower*

INTRODUCTION

Many large-scale transmission towers have been constructed, as high electrical transmission capacity is required for effective supply of electrical power. When, typhoon, 'Maemi', swept Korea in 2003, nine transmission towers collapsed, three were damaged severely and, as a result, many industrial plants and other infrastrucrure was paralyzed and enormous economical loss was caused. Also, prior to typhoon, 'Maemi', many existing transmission towers have been retrofitted according to revised design codes reflecting change of environmental condition, such as wind pattern and velocity. Transmission towers located in an open terrain such as shore or plain are exposed to strong winds and those located in mountainous areas undergo a wind speed-up effect over hills or escarpment. Especially, turbulence caused by complex terrain effects in mountainous areas increase the dynamic component in the wind loads. However, to date most retrofitting practices for transmission towers have employed only static approaches such as increasing member section area or shortening effective member length by additional members [1]. But, for wind loads with a lot of dynamic components, enhancing energy dissipation capacity incorporating with static retrofit could improve wind-resistant performance of the transmission tower effectively through the suppression of dynamic response amplification.

Existing studies on retrofitting transmission towers are given as follows. Albermani et al. proposed several retrofitting methods, which work as a diaphragm and constrain out-of-plane deformation of each face on the transmission tower, and verified their performance through experiments and numerical analyses [2]. Battista et al. investigated dynamic characteristics of the integrated structure composed of a tower body and transmission lines and applied a pendulum-type tuned mass damper to reduce dynamic response of the transmission tower [3]. Kilroe installed vibration absorbers to members in transmission tower arms to mitigate fatigue phenomenon under low levels of vibration caused by wind loads and reported their economical advantage over existing method such as changing critical members [4]. Xu et al. and Qu et al. verified vibration control performance of

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USING AN EQUIVALENT FIXED BASE MODEL TO INVESTIGATE THE EFFECTS OF SOIL-STRUCTURE INTERACTION

Cheng-Hsing CHEN[1] and Shang-Yi HSU[2]

SUMMARY

This paper is to investigate the effects of Soil-Structure Interaction (SSI) on the dynamic response of a soil-structure system. A model with a simple structure supported on elastic half space is utilized to derive the factor F_{SSI} that can completely represent the effects of SSI. This factor characterizes the altering of predominant frequency and damping ratio of the system when compared to the conventional rigid-base type structural analysis. Based on that, an Equivalent Fixed-Base (EFB) model, which takes the effects of SSI into account, can be constructed. Field test results are then used to verify the applicability of the proposed EFB model.

***Keywords**: soil-structure interaction; equivalent fixed-base model; Hualien large scale seismic test; forced vibration test.*

INTRODUTION

In engineering practice, a rigid-base model is often used to calculate the response of a structure subjected to earthquake excitations. This kind of analysis is based on an assumption that the deformability of foundation soils can be ignored, so that the interaction between the structure and supporting soils is negligible. It is reasonable and practical for conventional structures such as residential houses. However, for a massive structure founded on soft ground, the effects of soil-structure interaction (abbr. as SSI, thereafter) will significantly affect the response of structures. The inertial interaction resulted from the mass inertia of structure and the kinematical interaction resulted from the foundation scattering are important and will alter not only the response amplitude, but also the response characteristics of the structure. In the SSI analysis, the fundamental is to assemble the system matrix of the entire soil-structure system and solve the final response directly. From the results obtained, it is very difficult to quantify the influence of SSI on the structural responses. Therefore, those kinds of SSI analysis can only be used as a validating or checking analysis, and can not be applied to the design process directly.

To incorporate the effects of SSI into the structural design, the most important issue is to quantify the effects of SSI for a soil-structure system. For design purpose, it is well recognized that the predominant frequency and associated damping ratio are the key parameters for structural analysis. The former is used to locate where the maximum response will be, and the latter controls the magnitude of maximum response. Based on such understandings, this study aims at quantifying the effects of SSI on a soil-structure system. Since the phenomenon of SSI is very complex for a general structure with large number of degrees-of-freedom, this study will use a simple structure founded on elastic half-space as the fundamental model to deduce how the predominant frequency and associated damping ratio are affected by the effects of SSI. Once these two parameters are quantified for a soil-structure system, then an Equivalent Fixed-Base (EFB) model, which includes the effects of SSI, can be constructed. This equivalent model is actually a rigid-base model that can be conveniently applied in a conventional structural analysis for engineering design.

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STRUCTURAL SYSTEM ENABLING PROMPT RECOVERY AFTER EARTHQUAKES

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SUMMARY

After experiencing catastrophic earthquakes in major cities worldwide in 1990's and 2000's, the general public recognized that the economical loss due to the out-of-service period for repairing the damaged building was more significant than the cost to rehabilitate the damaged building itself. The base isolation system is one of the solutions to keep building functions after earthquakes, but the initial and maintenance costs are too high to apply to all kinds of buildings. The authors proposed a new economical structural system which enables prompt recovery after earthquakes using precast prestressed concrete frames with energy dissipating elements. Precast prestressed concrete structures show high self-centering characteristics with negligible damage and enable prompt recovery, if the excessive drifts due to nonlinear elastic behavior are reduced by using some energy dissipating elements. The structural system with precast concrete frames and energy dissipators satisfy the social demand with much less cost compared to that of the base isolation system. This paper introduces two types of the structural system and describes the design concepts. The focal point is the optimization of self-centering and energy dissipating properties. An FEM model was developed to simulate experimental results with high accuracy. From this analysis, a simplified calculation method was proposed to estimate hysteretic characteristics of the structures with each types of the system. Finally, design procedures for optimization of the restoring performance and the amount of energy dissipators were introduced.

Keywords: *Precast prestressed concrete structure; Damage control; Residual deformation; Energy dissipation; Optimization of hysteresis loop.*

INTRODUCTION

When major earthquakes attacked major cities in the world in 1990's and 2000's, even if structures do not collapse, many damaged reinforced concrete (RC) buildings did not function continuously without long term repair period. The economical loss in this period was more serious than the cost to rehabilitate damaged buildings. Though conventional seismic design codes require little or no damage for minor or medium earthquakes and prevent collapse of structures for major earthquakes, the demand of society is shifting to the higher level. The general public started to pursue structures which experience no or minor damage, hence necessitates no repair, and can be used immediately after the earthquake regardless of its intensity. For this reason, the base isolation system has been attracting attention. However, the base isolation system ordinarily needs much higher initial and maintenance cost and it is not very realistic to apply them to all sorts of structures.

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MODAL PARTICIPATION FACTOR ESTIMATION BASED ON VIBRATION TEST

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SUMMARY

The load distribution to each mode of a structure under seismic loading depends on the modal participation factor and thus the exact estimation of modal participation factor is essential to analyze the seismic response of a structure. The modal participation factor is not a unique value but varies depending on the normalization method of mode shape vectors. The modal participation factor of an idealized analytical model, however, is different to the actual one due to modeling and construction error. Therefore, there exist limits on the estimation of actual behavior. In this study, an identification procedure for participation factor based on vibration test is proposed. The modal participation factor is obtained from the relationship between observability matrices realized from system identification. Using the observability matrices, it is possible to transform an arbitrarily identified state space model obtained from the experimental data into a state space model which is defined in a domain with physical meaning. Then, the modal participation factor can be estimated based on the transformation matrix between two state space models. The proposed procedure has an advantage that the modal participation factor of the modes that are normalized to a certain element can be estimated from the response of the corresponding floor without knowing responses of other floors. Further, the mode shape vector can also be estimated directly from the experimentally estimated modal participation factors. The numerical simulation is performed to evaluate the proposed procedure, and the results show that the modal participation factor and mode shape vectors are estimated from the structural responses accurately. The procedure is also applied to the experimental data obtained from the shaking table test of a three story shear building model.

***Keywords**: Modal participation factor, Mode shape, System identification.*

INTRODUCTION

The modal participation factor is a coefficient that represents how the ground acceleration is distributed to each mode. This is because the inertial force, which is generated by the ground acceleration and the mass of each floor, is distributed to each mode through the modal participation factor for typical building structure. For this reason, the method to calculate the modal participation factor for the seismic analysis is presented in many references in the area of dynamics of structures (Clough and Penzien, 1975, Paz and Leigh, 2004).

The exact estimation of the modal participation factor is essential in seismic analysis and design of a structure. This is because the modal participation factor has a significant role both in estimating an accurate response and in design. Kim and Choi (2006) presented a nonlinear static analysis procedure for the design of supplementary dampers that uses the modal participation factor of the fundamental mode to obtain base shear versus roof-story displacement capacity curve of a structure from pushover analysis. Park et al. (2007) proposed a factored modal combination method for accurate prediction of the inelastic earthquake response of a structure by pushover

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SECANT STIFFNESS ANALYSIS METHOD FOR EARTHQUAKE DESIGN OF REINFORMECED CONCRETE STRUCTURES

Hong-Gun PARK[1], Chang-Soo KIM[2], and Tae-Sung EOM[3]

SUMMARY

A linear analysis method using reduced secant stiffness was developed for inelastic earthquake design of reinforced concrete structures. In the proposed method, the beam-column element and plane element, which are the same as used in conventional elastic analysis, are used for structural modeling. Based on the structural plastic mechanism intended by engineer, the distribution of inelastic members is determined. The secant stiffness of the inelastic members is determined based on the target ductility of the structure. Inelastic strengths of the members are calculated by using linear analysis on the structure modeled with secant stiffness. Plastic rotations in the inelastic members are calculated with the nodal rotations resulting from the secant stiffness analysis. For verification, the proposed method was applied to the inelastic earthquake design of a moment-resisting frame and a dual system.

***Keywords**: earthquake design; linear analysis; secant stiffness; inelastic behavior; reinforced concrete.*

INTRODUCTION

Generally, structures are designed to exhibit ductile behavior under strong earthquake. Therefore, in the earthquake design of structures, it is necessary to directly address a sound plastic mechanism intended by engineer. In most of modern design codes, the equivalent static design method using elastic analysis and the response modification factor is used for earthquake design of structures. However, safety of structures under large inelastic deformation cannot be accurately evaluated by using elastic analysis. Further, the response modification factor, which is prescribed according to structural types, does not directly address the effect of actual ductility of structures on the design earthquake load.
For inelastic earthquake design of structures, conventional nonlinear analysis can be used. However, since conventional nonlinear analysis is a pure analysis method, preliminary member design should be prepared for nonlinear analysis, and the plastic mechanism intended by engineer cannot be directly addressed. Therefore, for economical and safe design, nonlinear analysis and redesign should be repeated. Particularly, for 3-dimensional analysis, modeling for nonlinear analysis is very complicated. In most of analysis software, nonlinear analysis cannot be performed for structures with 3-dimensional walls. For this reason, despite its technical disadvantages, the equivalent static method is still popular in design practice because of its convenience in analysis and design.

In order to directly address the effect of the inelastic behavior of structures, secant stiffness analysis methods have been studied. (Priestley 1996, Shibata et al. 1976, Iwan et al. 1979) However, the application of existing secant stiffness methods has been limited because they require the use of an idealized substitute structure with single degree of freedom, or because inelastic deformation of members cannot be evaluated.

In the paper, a secant stiffness analysis method was developed to extend the application of secant stiffness

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APPLICATION OF ON-LINE RECURSIVE LEAST-SQUARES METHOD ON PARAMETER IDENTIFICATION OF STRUCTURES SUBJECTED TO EARTHQUAKE EXCITATIONS

Shih-Yu CHU[1] and Shih-Chieh LO[2]

SUMMARY

An on-line recursive least-squares (RLS) identification technique is applied in this study to identify the dynamic parameters of structures subjected to earthquake loadings. Based on the framework of adaptive control, the observations are obtained sequentially in real time. It is desirable to perform the identification tasks recursively to save computation time and to be able to observe the variations of parameters in real time. Computation of the least-squares estimation can be arranged recursively so that the estimated parameters at previous step can be used to predict the responses at current time. The one-step ahead predicted error between estimated response and measured response is calculated by the RLS method and the dynamic properties of system can be identified as well. The purpose of this study is to apply the RLS method in discrete-time and to perform the identification procedures on a single-degree-of-freedom system and a three-floor benchmark building under earthquake excitations. The identification performance that depends on the initial conditions of the Kalman gain vectors as well as the selection of appropriate exponential forgetting factor is addressed in detail in this study.

Keywords: *system identification; adaptive identification; recursive least-squares identification technique.*

INTRODUCTION

The aim of system identification is to find a mathematical model and to determine its parameters using the input and output signals. Generally speaking, system identification algorithms fall into two categories depending on whether they operate on the data in time domain, or in the frequency domain. Frequency-domain algorithms involve averaging temporal information, thus discarding most of their details. For structural systems whose parameters are expected to degrade with time, this trade-off of temporal information for frequency information is not always justifiable. Over the past decades, a few time domain methods have been transferred to civil engineering applications (Shinozuka et al. 1982; Ghanem et al. 1991). The discrete-time method for system identification based on linear filtering and least-squares estimation was discussed by Loh and Lin (1996). Both recursive least-square method (RLS) and adaptive forgetting through multiple models (AFMM) (Anderson 1985) with respect to ARX model are adapted as on-line identification scheme. Using the seismic response data of the Van Nuys seven-story building, the modal frequency and modal damping ratio of the fundamental mode are estimated by the on-line RLS. It was pointed out that different values of forgetting factor may give different results for the estimation of the modal parameters. Moreover, the identification results of the off-line identification techniques based on the first initial time segment are used as initial values for the on-line identification. Several unreasonable results are observed: (1) the identified natural frequencies from each seismic event to another are not consistent; (2) the time variation of the identified damping ratio during the first 10 sec of the record changes too rapidly. In view of detecting rapid changes of modal parameters, the AFMM method was applied. Although the identified fundamental frequency and damping ratio from event to event are more reasonable than the results obtained using the RLS method. No analytical analyses are performed for both

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AN EFFICIENT ANALYTICAL MODEL FOR DYNAMIC ANALYSIS OF TWO HIGH-RISE BUILDING STRUCTURES CONNECTED BY A SKY-BRIDGE

Dong-Guen LEE[1], Ah-Ram YANG[2], Hyun-Su KIM[3] and Hyun KO[4]

SUMMARY

Recently, a sky-bridge is frequently introduced to adjacent high-rise buildings as an escape route in the event of a fire or other emergency. Since vibration problem in high-rise buildings subjected to lateral forces occurs due to slender structural members, high strength and lightweight materials, a sky-bridge is used as not only a an escape route but also a vibration control device. Repetitive structural analyses are required to evaluate the effects of various connection systems of sky-bridge. Generally, time history analysis is used to predict accurate dynamic behaviors of high-rise buildings connected by a sky-bridge because connection systems usually have high nonlinearity. However, if a finite element model for entire high-rise building structures is used for repetitive time history analyses, analysis time could be significant. In this study, equivalent cantilever model has been proposed for efficient structural analysis of high-rise building structures connected by a sky-bridge in preliminary design. To consider more realistic conditions, the effect of belt-walls is included. Two single DOF model connected by spring and damper is used to investigate vibration control performance of linked structures varying dynamic characteristics. An example building structures are 49- and 42-story buildings constructed for the city of Seoul, Korea. Two buildings are connected by a sky-bridge at 34th story. An equivalent cantilever model with a sky-bridge for this example structure is developed by the proposed method. To verify the accuracy and efficiency of the proposed equivalent model, time history analysis has been performed for the equivalent model and the original model subjected to wind load. Wind load used in this study is obtained by wind tunnel tests. Based on the analytical results, it has been verified that the proposed equivalent model can provide dynamic responses of building structures connected by a sky-bridge with accuracy and efficiency.

Keywords: *Highrise building structures; Equivalent model; Time history analysis; Sky-bridge*

INTRODUCTION

After the World Trade Center tragedy, vertical evacuation systems have been one of the most important research topics for tall buildings. Horizontal evacuation at height is proposed through creating sky-bridge linkages between towers. This is a good choice when vertical evacuation routes out of the tall building cuts off in a fire. A sky-bridge can increase the evacuation efficiency without increasing the number of fire stairs. This is well demonstrated in the Petronas Towers in Kuala Lumpur, Malaysia. These days, there are many big cities in the Asian region having so many high-rise buildings built closely together. And, sky-bridges are frequently used for tall buildings as an evacuation option. Vibration problem in high-rise buildings subjected to lateral forces occurs due to slender structural members, high strength and lightweight materials. Very often, such slender and tall buildings experience excessive wind-induced vibration, which will cause discomfort to occupants or even shatter windows. Vibration control of tall buildings subjected to wind excitation has been studied extensively, and various types of control devices and algorithms have been developed. Viscoelastic dampers are frequently used to reduce lateral vibration of tall buildings subjected to wind or seismic excitation because it is easy to install no

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Response of Steel Moment Frames to Near-fault Horizontal and Vertical Earthquake Ground Motions

Heui-Yung Chang[1], Lin Bo-Cheng[2], Lin Chih-Ho[2] and Ker-Chun Lin[3]

SUMMARY

The recently occurred great earthquakes, such as the 1995 Kobe earthquake and 1999 Chi-Chi earthquake, reveal a fact that a moderate near-fault earthquake can cause severe vertical shaking. During the last few years, a number of studies have been made on the near-fault vertical shaking and its impacts on constructed structures. However, few of them gave detailed attention to the interaction between axial-force and moment in columns. As a result, the effects of near-fault vertical shaking have not been completely clarified yet. Considering that, this study assessed and compared the frame response to near-fault horizontal and vertical earthquake ground motions. The analyzed 6- and 20-story steel moment frames were designed for the seismic conditions of Chai-Yi City in Taiwan. The frame response was assessed using three-dimensional nonlinear time history analyses with near-fault earthquake records from the 1999 Chi-Chi earthquake. An identical scale factor was applied to each set of the ground motion components, and the horizontal shaking was shifted to the level of design earthquake and maximum considered earthquake. Following that, the responses due to simultaneously horizontal and vertical shaking were compared in detail to those due to horizontal shaking only. A simplified approximation was also given to the axial stress of columns subjected to near-fault horizontal and vertical shaking.

Keywords: *deformation demands, steel moment frames, near-fault records, vertical shaking.*

INTRODUCTION

The recently occurred great earthquakes, such as the 1995 Kobe earthquake and 1999 Chi-Chi earthquake, reveal a fact that a near-fault ground motion of moderate-to-large magnitude can cause severe vertical shaking (Ambraseys and Douglas 2003). This has raised serious concerns, because the vertical shaking may lead columns to yield and fail at an earlier stage. During the last few years, a number of studies have been made on the near-fault vertical shaking and its impacts on constructed structures (e.g. Pan and Nakashima 2003; Kunnath et al 2008; Lin et al 2008). However, few of them gave detailed attention to the interaction between axial-force and moment in columns. As a result, the effects of near-fault vertical shaking have not been completely clarified yet.

To address the issue, the presented study assessed and compared the frame response to near-fault horizontal and vertical earthquake ground motions. Three-dimensional nonlinear time history analyses were carried out for a 6- and 20-story steel office buildings with 7 sets of near-fault earthquake records from the 1999 Chi-Chi earthquake. An identical scale factor was applied to each set of the ground motion components, and the horizontal shaking was shifted to the level of design earthquake (DE) and maximum considered earthquake (MCE). Following that, the responses due to simultaneously horizontal and vertical shaking were compared in detail to those due to horizontal shaking only. All that helps gain a better understanding about the effects of near-fault vertical shaking.

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MACRO MODEL SIMULATING THE SEISMIC FORCE RESISTING MECHANISM OF MULTI-STORY SHEARWALLS SUPPORTED BY PILES

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

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SUMMARY

Structural walls are supported by foundation beams and piles that transfer earthquake-induced forces from the structural walls to the soil. In the current design procedure, the structural walls are normally assumed to stand on the solid foundation. This assumption makes it possible to evaluate the seismic behavior of each member independently. However, in case that the foundation beams don't have enough strength and stiffness to resist against seismic lateral forces, unexpected lateral load resisting mechanism can be formed. The resulting stress state may be completely different from that based on the assumption of the solid foundation.
This study aims to analytically clarify the lateral load resisting mechanism of the structural wall system considering the interaction between the structural wall, the foundation beam and the piles. One 25% scale specimen was modeled in two different ways. Difference of the models was with or without the pile foundation of the specimen.
The analytical models made clear the differences of the lateral load resisting mechanism. Experimental results, such as hysteretic curve of the structural wall, deformation and damage, could be simulated well by the model with the pile foundation. It was confirmed that the behavior of the structural wall with the pile foundation could be simulated by the rotational model of the wall-pile assemblage.

Keywords: *Structural wall; Foundation beam; Pile; Macro Model; Interaction*

INTRODUCTION

Typical Japanese RC mid-rise and high-rise residential buildings have multiple bay moment resisting frames in the longitudinal direction and single bay structural wall systems in the transverse direction. The structural walls are widely used for buildings in order to provide high stiffness and strength against seismic lateral forces. The structural walls are also supported by foundation beams and piles that transfer earthquake-induced forces from the structural walls to the soil. In current design procedures, the structural walls are normally assumed to stand on solid foundation. This assumption makes it possible to evaluate the seismic behavior of each member independently. As extensive studies have been conducted based on this assumption, design procedures for these structural members are well established. However, whether the assumption is valid or not is hardly confirmed.
If the foundation beam doesn't have sufficient stiffness and strength to regard it as an infinitely rigid element, it is possible to consider the structural wall and the foundation beam as one structural wall whose base is supported by piles. In this case, the resulting stress state may be completely different from that based on the assumption of solid foundation.

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Cyclic Testing of Bracket and WUF-B Type Weak-Axis Steel Moment Connection

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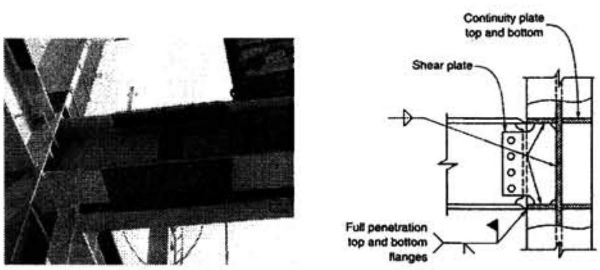
SUMMARY

Foci have been on the strong axis steel moment connections after Northridge earthquake in 1994. However researches on the seismic behavior of weak axis moment connections could be hardly found, even though these connection details have been frequently used as seismic details of MRF in Korea. Therefore, the objective of this research is to provide better knowledge on the seismic behavior of weak-axis steel moment connections that can be broadly applicable to many structures with similar characteristics. For this purpose, experimental program was designed and performed with 2 types of weak-axis steel moment connections, bracket type and WUF-B type, based on the survey of the existing field data and literatures. Using the experimental results obtained from the quasi-static cyclic testing of these specimens, structural performances of the joints such as hysteretic curves, maximum strength capacities and strain of reinforced bars were investigated. From the test results, bracket type connection revealed to have more than 5% story drift capacity and WUF-B type connection, 4%. These specimens also revealed to have higher strength capacities than the nominal design strength. The bracket type connection showed slow strength degradation after maximum strength was researched, however the WUF-B type connection showed rapid strength degradation causing brittle behavior

Keywords: *Momentmoment resisting frame, weak-axis steel moment connection, cyclic testing, seismic evaluation, story drift ratio*

1. INTRODUCTION

Recognizing the needs for the development of MRF connection details, SAC Joint Venture was initiated in USA. With the almost 10 years researches, qualified MRF connection details were proposed in 2000(FEMA-350). These researches were mostly focused on the strong axis MRF connections because the weak axis MRF connections were hardly used in USA. However the weak axis MRF connections are frequently used as seismic details of MRF in Korea. (FIG. 1)



(a) Bracket type (b) WUF-B type

FIG. 1 Representative weak axis steel MRF connection details in Korea

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Experimental Study on Seismic Behavior of Interior CES Beam-Column Joints

Tomoya MATSUI[1] and Hiroshi KURAMOTO[2]

SUMMARY

Development studies on composite Concrete Encased Steel (CES) structures composed of steel and fiber reinforced concrete (FRC) have been continuously conducted by the authors. This paper presents an experimental study on seismic performance of interior CES beam-column joints. In this study, four interior CES beam-column joints were tested under lateral load reversals and constant axial load to grasp their failure mode, cracking damage, lateral load carrying capacity and deformability. Two specimens with different failure mode, the beam yielding type and the joint shear failure type, were prepared. Other two specimens were the beam yielding type and had different thickness of panel web and flange around panel web in this test. The CES beam-column joints with the beam yielding had stable restoring characteristic with little damages in cover concrete until a large story drift angle of 0.05 rad.. The CES beam-column joint with joint panel shear failure showed strength deterioration after reaching maximum strength at a drift angle 0.015 rad.. However, falling of cover concrete was not observed at the joint panel regardless of not having reinforcement bars, it is confirmed that the use of FRC facilitates to control the damage of the joints. In addition, it is shown that the ultimate shear strength of CES beam-column joint could be evaluated using the calculation method in AIJ design standard for SRC structures in comparison experimental maximum shear force with calculated joint panel shear strength.

***Keywords**: Composite structure; CES beam-column joints; Static loading test; Fiber reinforced concrete; Shear capacity magnification factors; failure mode.*

INTRODUCTION

Steel Reinforced Concrete (SRC) Structures developed in Japan have good structural performance for resisting lateral forces imposed by wind and earthquakes, and have been adopted for medium-rise, high-rise, and super high-rise buildings. However, the number SRC of structures constructed has decreased since the 1990s. The decrease in the number of constructions has been caused by the development of a new structural engineering system called the High-strength concrete structure or Concrete-Filled Steel Tube (CFT) structure, but the main reason thought for the causes of the decrease is the construction problems that increase construction costs and lengthen construction schedules. Even so, it could be important that SRC structures provide better seismic performance in comparison to other structural systems. So, the authors aim to develop a structural system with as good seismic performance as SRC structures and good workability, and have conducted a continuing development study on composite Concrete Encased Steel (CES) structures composed of steel and fiber reinforced concrete (FRC) as shown in Figure 1.

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

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BASIC EXPERIMENTS FOR DEVELOPING TUBULAR DAMPERS APPLIED TO BOLT JOINTS

Seiji MUKAIDE[1], Takuya ASAMITSU[2], Motohide TADA[3], and Isao KOHZU[4]

SUMMARY

In order to adapt to anchor bolts of exposed bases in steel moment frames, we devise a hysteretic damper with tubular shaped steel in a bolt joint. The damper is expected to deform stably to large plastic deformation range after local buckling occurs in the tubular part. In this paper, basic experiments are described for the development of the devise. Firstly, monotonic loading test is conducted in order to investigate compressive behaviors of commercially available steel tubes until the extreme condition. The test result shows that the tubes have high deformation capacity while elephant-foot buckling events occur sequentially like an accordion. Secondary, cyclic loading test of the damper is conducted to investigate deformation capacity of the damper whose ends are welded. The test results show that the damper fractures before the damper reaches a large enough deformation range as a hysteretic damper.

Keywords: *hysteretic damper; local buckling; anchor bolts; diameter-to-thickness ratio; deformation capacity; tubular shaped steel.*

INTRODUCTION

A variety of seismic dampers are utilized to reduce structural response to seismic loading. It is possible for viscoelastic or hysteretic dampers to reduce structural and nonstructural damage from seismic events. However, it is difficult to design a steel moment frame which suffers no damage at column bottoms of the first story under a sever earthquake. In order to adapt to anchor bolts of exposed-type column bases in steel moment frames, we devise a hysteretic damper with tubular shaped steel in a bolt joint as shown in Fig.1. In the proposed devise, one end of the damper is combined with the nut, and the other is combined with the base plate. When the column bottom is subjected to flexural deformation, the damper is expected to keep the other parts elastic by absorbing seismic input energy. Therefore, the damper must deform stably to large plastic deformation range after local buckling occurs in the tubular part.

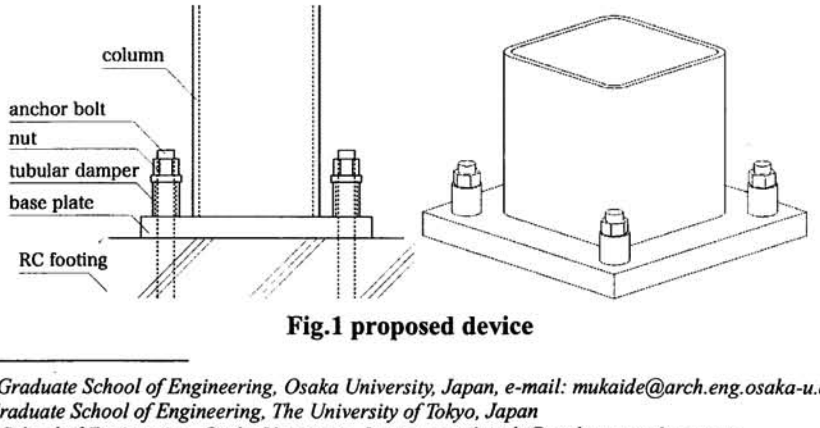

Fig.1 proposed device

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SEISMIC DESIGN OF THE VERTICAL BOUNDARY ELEMENT IN STEEL PLATE SHEAR WALLS

Chao-Hsien LI[1], Chih-Han LIN[2], Pei-Ching CHEN[3] and Keh-Chyuan TSAI [4]

SUMMARY

This paper proposes a capacity design method for the first story vertical boundary element (1F VBE) which is the first story column adjacent to the infill steel panel of a steel plate shear wall (SPSW). This design requirement is to ensure that the plastic hinges forming at the bottom ends of the 1F VBEs when an SPSW develops its plastic mechanism. Several undesirable inelastic responses resulted from weak first story columns were observed in the past experiments. It is presented in this paper, including: (1) a plastic hinge forming within the column height; (2) soft-story mechanism taking place at the first story; and (3) out-of-plane flexural buckling of the column. Analytical studies on the inelastic responses of the weak columns were conducted in this research. The inelastic column responses and its effects in SPSWs with hinging forming in the mid-height of the first-story column are presented in this paper. The paper confirms the effectiveness of the proposed capacity design method by evaluating a number of past experimental test results.

Keywords: *steel plate shear wall; SPSW; capacity design; vertical boundary element.*

INTRODUCTION

A steel plate shear wall (SPSW) consists of infill panels and boundary elements. The beams and columns surrounding the infill panels are called horizontal boundary elements (HBEs) and vertical boundary elements (VBEs), respectively. An SPSW can effectively resist horizontal earthquake forces by allowing the development of diagonal tension field action after the infill plate buckle in shear thereby dissipating earthquake energy through the cyclic yielding of the infill plate in tension. The concept of using the post-buckling strength of the steel plate to resist the earthquake forces was first proposed by Thorburn *et al.* (1983). The method of calculating the angle of the tension field action α has been developed by Timler and Kulak (1983). The boundary beam and column elements are responsible to anchor the tension field action of the fully yielding infill plates. Except for the plastic hinges forming at the boundary beam ends and the bottom ends of the first story boundary columns, the remaining parts of the boundary elements are required to remain elastic under the forces generated by the fully yielded plates. There have been a number of studies focusing on the capacity design of the boundary elements. Berman and Bruneau (2003) employed plastic analysis methods for SPSWs. The capacity design method for the top beam has been studied by Vian and Bruneau (2005). The design guide for SPSW was then introduced by Sabelli and Bruneau (2006). Park *et al.* (2007) studied the capacity design method for the columns. Many experimental studies on the multi-story steel plate shear walls have been conducted (caccese *et al.* 1993; Driver *et al.* 1998; Lubell *et al.* 2000; Behbahanifard *et al.* 2003; Tsai *et al.* 2006; Park *et al.* 2007; Tsai and Li, 2008). In some tests, the failure modes of the SPSW specimen were governed by the excessive yielding or buckling of the first story vertical boundary elements (1F VBEs). This paper proposes a capacity design method for the 1F VBE to ensure the plastic hinges forming at the bottom ends. In this paper, analytical evaluations of

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A PLASTIC HINGE MODEL FOR NONLINEAR DYNAMIC PROGRESSIVE COLLAPSE ANALYSIS OF WELDED STEEL MOMENT FRAMES

Cheol-Ho Lee[1], Seonwoong Kim[2] , and Kyungkoo Lee[3]

SUMMARY

In this study, a parallel axial-flexural hinge model capable of representing the interaction effects of axial force and moment is proposed for a simplified but accurate nonlinear dynamic progressive collapse analysis of welded steel moment frames. To this end, the behavior of double-span beams was first investigated based on the experimentally validated material and geometric nonlinear parametric finite element analysis. A multi-linear point hinge model which captures the moment-axial tension interaction was then proposed. The emphasis was to develop a reliable and computationally efficient macro-model for practical progressive collapse analysis. The application of the proposed model to nonlinear dynamic analysis was illustrated by using OpenSEES program. The accuracy as well as the efficiency of the proposed model was verified based on inelastic dynamic finite element analysis. The importance of including catenary action to assure proper progressive collapse resistant analysis and design was also emphasized.

***Keywords**: progressive collapse; steel moment frames; plastic hinge; moment-axial tension interaction; nonlinear dynamic analysis.*

INTRODUCTION

Progressive collapse may be described as the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it (ASCE 2005). The most viable design approach to resist progressive collapse is to provide sufficient redundancy and ductility in the structural system such that surrounding structural elements could mobilize their reserve capacity to bridge over the damaged area. In this sense, ductile steel moment frames may be a good structural system to resist progressive collapse.

Recent design procedures to mitigate the potential for progressive collapse in structures can be found in the design guidelines issued by the U.S. General Services Administration (GSA 2003) and the Department of Defense (DoD 2005). The direct method, or the alternate path method is preferred in these design guidelines. In this method, a single column is typically assumed to be suddenly missing, and analysis is conducted to determine whether or not the structure can bridge across the missing column.

Experimental and analytical studies on the progressive collapse behaviour of steel moment frames have been conducted recently under the missing column scenario. For example, full-scale subassemblies of steel moment frames in double-span configurations were tested pseudo-statically by Karns et al. (2006) in order to identify beam-to-column connection systems capable of arresting progressive collapse. Khandelwal and El-Tawil (2007) numerically investigated several critical design variables that can affect the formation of catenary action in steel special moment frames. Their numerical study demonstrated the ductility of seismically-designed special moment frame connections and the ability to deform in catenary mode. Hamburger and Whittaker (2004)

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

Masato DOI[1], Yukako ICHIOKA[2], Yoshihiro OHTA[3], Susumu KONO[4] and Fumio WATANABE[5]

Note: This paper was submitted to 14WCEE, The 14th World Conference on Earthquake Engineering October 12-17, 2008, Beijing, China

SUMMARY

This research aims to propose an economical seismic response controlling system of RC frames using corrugated steel shear panels (CSSP). This hybrid system was originally proposed by Mo and Perng in 2000. In this study, their system was revised and new hybrid system was proposed to prove that the shear capacity and stiffness of CSSPs can be fully utilized if sufficient anchorage is provided. In an experimental phase, a stud-type anchorage often used in bridge box girders was employed. The behavior was stable if the number of studs satisfied the Japanese design guidelines. The behavior after buckling of CSSP was ductile and degradation in lateral load carrying capacity was about 20% even at 5% story drift angle. However, even specimens with half number of studs showed the similar behavior. All hybrid frames showed more than 30% increase in lateral load carrying capacity as originally designed. In an analytical phase, the analytical model proposed well simulated the behaviors of frames with different anchorage of the shear panels, considering the effective area of CSSP which carries the lateral load. The analytical result showed that CSSP carries more than 90% of its shear capacity even if the panel has the half number of studs determined by the design guidelines. This study proved that CSSP has a potential to make a main structural component to carry lateral load. The use of CSSP in building structures has just begun in Japan.

Keywords: *Corrugated steel shear panel; Improvement of ductility; Damage control; Energy dissipation; Frame analysis.*

INTRODUCTION

It is a common practice to use reinforced concrete shear walls in reinforced concrete structures to maintain high lateral load carrying capacity and stiffness. However, high lateral stiffness with brittle ultimate failure mode of RC structural walls often requires high lateral load carrying capacity according to their seismic response characteristics. In order to improve the ductility of reinforced concrete shear walls, some efforts have been made such as using low yield strength reinforcement and introducing slits but the ductility enhancement was not very prominent. Use of steel shear walls in order to increase ductility has some decades of research history. In 1973, Takahashi et al. (1973) studied the characteristics of load-deflection relations of steel shear walls obtained experimentally and reported the effects of configuration, width-thickness ratio, stiffeners' stiffness, etc. on the load-deflection relations. Studies on steel shear walls have been continued since then by Gaccese V (1993) and Driver (1998). However, flat steel shear panels need stiffeners to prevent plate buckling, leading to the increase

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MINIMUM AND MAXIMUM SHEAR REINFORCEMENT OF REINFORCEMENT CONCRETE BEAMS

Jung-Yoon LEE[1] and Hyun-Bok HWANG[2]

SUMMARY

The ACI 318-05 design code requires a minimum shear reinforcement in reinforced concrete (RC) beams to ensure adequate reserve shear strength and to prevent possible sudden shear failure following the formation of the first diagonal tension crack for high strength concrete. This code also requires a maximum amount of shear reinforcement in RC beams for two reasons, first to ensure adequate reserve shear strength and to prevent possible sudden shear failure due to concrete crushing before yielding of stirrups due to over shear reinforcement and second to reduce unsightly cracking. While the maximum shear reinforcement ratio in EC2-02 or CSA-04 is expressed in terms of the compressive strength of concrete and the yield stress of the shear reinforcement, the maximum shear reinforcement of ACI 318-05 is proportional to 1/2 power of the concrete compressive strength. In this paper, evaluation equations of the minimum and maximum amount of shear reinforcement for RC members are presented to prevent their abrupt shear failure, based on the compatibility-aided truss model. The proposed equation takes into account the influences of the longitudinal reinforcement ratio, the shear span-to-depth ratio, and the compressive strength of concrete. Even if the proposed equation is rather tedious, this equation will help to understand the effects of the effective strength of concrete, the crack angle, and the stress conditions of the materials on the minimum and maximum amount of shear reinforcement of RC beams.

Keywords: *RC beams; maximum amount of shear reinforcement; minimum amount of shear reinforcement.*

INTRODUCTION

The failure modes of reinforced concrete(RC) members dominated by shear force are generally classified into four types of modes: minimum shear reinforcement failure, tension failure, balanced failure, and compression faiure(over reinforcement failure). These four failure modes are influenced by the compressive strength of concrete and the amount of shear reinforcement in the concrete. Among these failure types, the minimum shear reinforcement failure and over reinforcement failure occur abruptly without sufficient advanced warning.

The ACI 318-05 design code (ACI, 2005) requires a minimum amount, $\rho_{\min}$ of shear reinforcement in RC beams to reserve shear strength and to prevent sudden shear failure upon first diagonal tension cracking. The code also requires the maximum spacing, $s_{\max}$, of the vertical stirrups as the smaller of d/2 or about 600mm, so that each crack will be intercepted by at least one stirrup. The ACI 318-99 (ACI, 1999) design code required that the minimum amount of shear reinforcement be $\rho_{\min} = 0.33/f_y$ (MPa) for the compressive strength of concrete, f_c' up to 69MPa. For $f_c' > 69MPa$, the code increased the $\rho_{\min}$ requirement by multiplying a factor of $f_c'(MPa)/35$, but not to a value greater than 1.0MPa where f_y is the yield stress of stirrups. The requirement of $\rho_{\min}$ in the ACI 318-99 did not include the influence of f_c' in the relation when $f_c' \le 69MPa$ and

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EFFECT OF ANCHORAGE PERFORMANCE OF BEAM BARS WITHIN BEAM-COLUMN JOINTS AND JOINT SHEAR STRESS LEVELS ON RESTORING FORCE CHARACTERISTICS IN REINFORCED CONCRETE FRAMES

Masaru TERAOKA [1]

SUMMARY

To investigate the effect of beam bar bond performance of beam longitudinal bars passing through the interior beam-column joints and joint shear stress levels on restoring force characteristics of reinforced concrete frames, cyclic loading tests were carried out on eleven half-scale interior beam-column subassembliages using high strength materials. The following findings were obtained from this study:

(1) The data of relationship between the local bond stress and the amount of slippage for beam bars within beam-column joints is obtained.

(2) The restoring force characteristics in structural frames are influenced not only by the bond stress level of beam bars passing the through joints but also by shear stress level of joint panels.

(3)The measured restoring force characteristics can be approximately predicted according to the method proposed by the author.

Keywords *:beam-column joint; bond performance of beam bar within joint; joint shear stress level; restoring force characteristic.*

INTRODUCTION

Recently high strength materials are used when an RC structure is to make high-rising, span is to make longer, and cross-section is to be reduced. These result in higher shear stress in joint panels and higher bond stress in beam longitudinal bars passing through the beam-column joints, deteriorating the seismic capacity and increasing the risk in the structure. The bond performance is related not only to slippage of beam bars but also to the shear strength of joint panels, and affects the restoring force characteristics in structural frames.

In this study, to investigate the effect of the bond performance of beam longitudinal bars passing through the beam-column joints and joint shear stress levels on restoring force characteristics in reinforced concrete frames, cyclic loading tests were carried out on half-scale interior beam-column subassembliages using high strength materials.

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EXPERIMENTAL STUDY OF STEEL REINFORCED CONCRETE BEAMS WITH WEB OPENINGS

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SUMMARY

The openings in the webs of beams in building are necessary for the passage of utility ducts. This study aims to investigate the behavior of steel reinforced concrete (SRC) beams with an opening, including the effects of various opening shapes and different values of moment to shear ratio on the strength. Thirteen full-scale SRC beams were designed to have different opening shapes and tested at different moment to shear ratios. Test results indicate specimens with high moment to shear ratio demonstrated ductile behavior due to the confinement attributed to the stirrup and structural steel. Specimens with low moment to shear ratio failed owing to the shear cracking. An interaction between bending and shear was examined for tested SRC beams.

***Keywords**: web opening, steel reinforced concrete, shear failure, interaction, SRC.*

INTRODUCTION

The transverse holes in the webs of beams in buildings are necessary for the passage of service ducts and piping in order to minimize the story height and to attain economic requirements. The web openings of the beam result in the decrease of flexural and shear strengths, flexural stiffness, and the increase of the deflection. The reinforcement at the opening is needed to ensure the proper strength and stiffness of the beams.

Numerous experimental and analytical studies have been conducted to investigate the strength of reinforced concrete flexural members with openings. The design for a flexural member with a large opening had been proposed that the top and bottom cross members of the opening could be assumed to behave like a vierendeel panel. The top and bottom chord members were expected to carry shear and axial force acted from the bending moment (Nasser et al. 1967; Mansur et al. 1985). Further, ACI design provisions specify that the openings in the web of a member can reduce the shear strength, and the effect of any openings on the shear strength shall be considered (ACI 318 2005).

Meanwhile, numerous studies for structural steel beams, including steel beams supporting a reinforced concrete slab, with web openings have been carried out and design provisions have been recommended. AISC specifications specify that the effect of web openings on the nominal shear strength of beams shall be determined, and adequate reinforcement shall be provided if necessary (AISC 2005). Several strength limit states may control the design of a flexural member with web openings, such as local buckling of the compression flange or the web, yielding or local buckling of the tee-shaped compression zone, and moment-shear interaction. Moreover, the design procedure is given in the literature (ASCE 1999; Darwin 1990).

Although many investigations have been conducted on reinforced concrete and structural steel beams with

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FULL-SCALE HIGH-STRENGTH CONCRETE TIED COLUMNS UNDER CONCENTRIC COMPRESSION

Rong-Jing WANG[1], Hung-Jen LEE[2], Cheng-Cheng CHEN[3], Chi-Chun TAO[4], and Chia-Wei CHEN[5]

SUMMARY

This paper presents a beginning experimental program consists of 21 large-scale high-strength concrete tied columns subjected to concentric compression. Each column had a cross sectional dimension of 500 mm and 1500-mm height. The concrete compressive strengths ranged between 53 and 98.8 MPa. Other test variables include steel yield strength, tie configurations, and tie spacing. The axial load-deformation behavior and axial capacities are evaluated with respect to building code equation. High-strength concrete columns contained insufficient volumetric ratio of closely spaced transverse reinforcement could not sustain nominal axial strength due to early buckling of column bars after the premature palling of cover concrete. The mean concrete strength measured in column specimens are below 85% of cylinder compressive strength.

Keywords: *column; confined concrete; high-strength concrete;*

INTRODUCTION

The strength of structural concrete has increased over the years. High-strength concrete (HSC) has been used in many metropolises of the world, mostly in columns of high-rise buildings due to high axial loads. After the New RC Project, the use high-strength concrete in the construction of high-rise buildings has increased rapidly. Utilizing HSC together with high-yield-strength reinforcement in the precast construction of high-rise buildings is very attractive because of time, labor, and material savings. Borrowing the successful ideas of New RC Project in Japan, the construction industry in Taiwan would also like to upgrade the strengths of concrete and reinforcement. By reviewing the HSC column tests carried out in Japan as part of the New RC project, it is concluded that the axial load-dominated elements, such as HSC columns, are key elements in the construction of high-rise buildings.

Kato et al. (1998) reviewed tests carried out in Japan on 91 square columns and 59 circular columns under concentric compression. The compressive strength of the concrete in the specimens ranged between 28 and 131 MPa (4 and 19 ksi), while the yield strength of the transverse reinforcement ranged between 172 and 1365 MPa (25 ksi and 198 ksi). It can be seen in **Figure 1** that a variety of material strengths has been studied in Japan, but most data were small- or medium-scale specimens with cross-sectional dimension about 250 mm (10 in.). However, in-field dimension of HSC columns could be 1000 mm (40 in.). It often raises questions and safety concerns to project the test data obtained from small-scale specimens to the application of large-scale structures. Given the difficulties associated with testing large-scale HSC columns under concentric compression, very little experimental data are available for concentric compression tests of large-scale HSC columns. As shown in Fig. 1. There are only 4 data points available for HSC tied columns with a cross sectional dimension of 470 mm (without cover), which were test by Tanaka and Hiraishi et al. (1993) as a series of New RC project.

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EXPERIMENTAL DETECTION OF A MECHANICAL WEAK POINT IN CONCRETE WALL STRUCTURES

Yasushi SANADA[1], Botirjon YORKINOV[2], and Taizo HIROSE[2]

SUMMARY

Accurate performance evaluations of structures and/or structural elements are playing more important roles in the latest design methodologies. Although a huge number of laboratory tests on structural members have been carried out for performance evaluations, experimental data on stress or internal force are extremely limited because of technical difficulties in measuring them. Therefore, a series of structural tests on concrete wall structures was conducted to obtain such experimental data. In this study, in particular, a new force-measuring system was developed and applied to a concrete block infilled reinforced concrete frame specimen. The results of force measurements revealed that the internal forces on the critical section were significantly concentrated around the boundary column on the compressive side according to the damage. This result indicates that a compressive column bottom is not only a structurally important section but also a mechanical weak point in this kind of structure, and that retrofitting the weak point would possibly induce rational improvements of seismic performance.

***Keywords**: concrete block infill; cyclic loading test; force measurement; reinforced concrete frame; stress concentration.*

INTRODUCTION

Accurate performance evaluations of structures and/or structural elements are playing more important roles in the latest design methodologies. The design of reinforced concrete (R/C) structures is no exception, hence a huge number of laboratory tests on R/C members have been carried out for performance evaluations. Actually, however, experimental data on stress or internal force are still extremely limited because of technical difficulties involved in measuring them.

Several past studies have focused on internal forces carried by R/C members or partial frames, mainly to clarify force distributions and transfers among structural components (e.g. Clough and Bertero. 1977, Sugaya et al. 2000, Canbay et al. 2003). However, such objectives are different from that of this study, which is to clarify internal force distributions and transfers of an individual structural member.

The authors collected experimental data on internal forces of a shear-critical R/C wall in a previous study (Sanada and Kabeyasawa. 2006). Therefore, in this study, a series of structural tests on concrete block infilled R/C frames was conducted to obtain the same kind of experimental data. A new force-measuring system was developed to accomplish this objective, and was also verified through structural testing. Focusing on internal shear transfer of a specimen, specially designed herein, the seismic behavior and performance were investigated experimentally.

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Design Equations for Flexural Strength of Prestressed Concrete Beams Based on Flexural Deformation

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SUMMARY

In Standard for Structural Design and Construction of Prestressed Concrete Structures and some guidelines published by AIJ the design equations for flexural strength of prestressed concrete beams have been proposed. These all equations the based on the assumption that every longitudinal bars (normal steel bars and PC steel bars) are in their yielding condition. But PC steel bars which figure are round type or strand type have less bond strength than that of deformed type and PC steel bars usually placed in the center side in the section of members. Therefore this assumption should not be applicable to PC steel bars without conditions. In this paper the equation to estimate the stress of PC steel bars to calculate the flexural strength are proposed and the equation are based on their conditions, those being, type of figure, location in the section, tendon in original condition, and shear-span ration of member.

Keywords: *earthquake engineering; prestressed concrete beam; flexural strength; flexural deformation; analytical investigation.*

BACK GROUND

Design equations of the ultimate flexural strength of prestressed concrete beams are proposed in standard (Standard for Structural Design and Construction of Prestressed Concrete Structures) and some guidelines[2),3)] published by AIJ (Architectural Institute of JAPAN). In ref. 4, flexural strength calculated by the proposed equations were compared to the experimental data and it is reported that the arithmetic precision of the proposed equations is comparatively adequate.

In this comparison between calculated values and the experimental data, the employed experimental data are the maximum value of flexural strength measured during loading test. Therefore the deformation of the test specimens at their maximum flexural strength might be larger than the practical level deformation which estimated in the earthquake. In this case the flexural strength will not reach that calculated by those design equations proposed in the standard and the guidelines. This phenomenon are caused by this fact that PC steel bars may not reach their yielding strain in the practical deformation level. Those are referable to difficulty of PC steel yielding which caused by following 3 reasons. 1) PC steel is usually located on the inside of the cross section compared to the longtudinal bars. 2) There are bond degradation between grout and PC steel. 3) PC steel has longer elastic range than normal longitudinal bars.For these reasons, the final flexural strength calculated by the proposed equations may overestimate the actual strength of pc beams.

In order to resolve this problem, we proposed the method to estimate the stress of pc steel with relation to the deformation of members in the ref. 5 and by applying the estimated value of pc steel to the equation (1) - (3) the precision of calculation of ultimate flexural strength is raised.

In those equations, strain of PC steel bar (ε_{pc}) is calculated by considering of parameters such as strain level at pre-stressing (ε_{ori}) (at first stage tensioning), location of PC steel bars (d_{p2}), the areas of PC steel bars (q_{pr}), deformation angle (R) of member and bond strength between concrete and PC steel bars which is defined by F (figure-1). In those factors F (the bond strength) which is effected by the figure of PC steel bars (round bar,

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SEISMIC BEHAVIOR OF A 3D PRECAST/POST-TENSIONED REINFORCED CONCRETE SUB-STRUCTURE UNDER BI-AXIAL LOADS

C. T. CHENG[1], H. H. CHEN[2], K.C. LIN[3], P. C CHEN[4] and S.J. JHUANG[4]

SUMMARY

The precast and post-tensioned structures can be designed in a way of self-centering that minimizes permanent drift after an earthquake. The self-centering function is achieved through the post-tensioned strands applied in the precast beams that allow the gap-open in the beam-column interface during earthquakes. Literatures have proved that the self-centering was effective in the beam-column connections. In real structure, there are floors integrating beams and columns that are usually designed as a rigid diaphragm to transfer the seismic loads between gravity and moment-resisting frames. However, this rigid floor may limit the gap-open in the beam-column interface of the self-centering structures. Therefore, a new design of the floor system that also has self-centering features is proposed. In order to investigate the self-centering function in real structure, a one-story 3D precast and post-tensioned sub-structure with floor slab was constructed and tested. The tested specimen consists of four columns, four beams and one slab, representing one gravity frame, one moment resisting frame and the floor system that connects both frames. This test aims to investigate the force transfer by the floor slab as well as the seismic performance of the structure, in which the gaps in the beam-column and beam-floor interfaces may be opened in two directions, when subjected to bi-axial lateral loads. In addition to the experimental investigations, a theoretical model is also proposed to simulate the envelope of the force-deformation of the sub-structure.

Keywords: *full-scale testing, precast concrete,pPost-tensioned, self-centering, seismic behavior, and bi-axial loads.*

INTRODUCTION

In 2002 Christopoulos et al. and Ricles et al. investigated the self-centering function of beam-column connections under earthquake loads. They proved that the self-centering could effectively minimize residual drift after seismic loads in the prestressed beam-column connections. Under extreme loads, the self-centering function is achieved through the gap-open in the beam to column interface that eliminates the plastic deformation at the beam-ends. And then the prestress inside the beams restores its original position after loads. However, in real structures, the role of the floor in the self-centering structure is not clarified yet. If the floor slab is designed to be a rigid diaphragm integrating beams and columns together like the conventional structures, this floor slab may constrain the gap-open in the self-centering connections. As a result, Garlock and Li 2005 proposed a new floor detail using collector beams to transfer the seismic loads between frames, but required it soft enough not to hinder the gap-open in the connections at the same time. Cheng and Ke 2007 proposed a new concept for the design of floor slab in the self-centering structure by using precast/post-tensioned slab. The test results showed that damage in the beam cover concrete and out-of plane deformation in the thinner slab reduced the gap-open in the post-tensioned frame, resulting in less efficiency of the force transfer in the slab.

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EXPERIMENTAL STUDY ON MECHANICAL PROPERTIES OF PRESTRESSED CONCRETE BEAMS AT ELEVATED TEMPERATURE

Seong-jun IM[1], Minehiro NISHIYAMA[2] and Masanori TANI[3]

SUMMARY

Performance-based design has been common in structural design of buildings while it has not yet been used in fire resistance design in Japan. Prescriptive design method is still applied to fire resistance design of prestressed concrete members. What structural designers do is to provide the minimum concrete cover to prestressing steel specified in Standard for Structural Design and Construction of Prestressed Concrete Structures published by AIJ or Architectural Institute of Japan. The specification is based on experiments on prestressed concrete beams, which were carried out a few decades ago. Fire resistance tests on prestressed concrete members have not been conducted since those experiments although such tests have been carried out on conventional reinforced concrete members, especially constructed of high-strength concrete.

Fire tests conducted to obtain experimental data on load-deflection behavior of prestressed concrete beams at elevated temperature are reported in this paper. Three post-tensioned prestressed concrete beams were constructed and tested in a furnace under the constant vertical load simulating service load. They were identical and about one-third as large as prestressed concrete members in practice. The experimental parameter was duration of heating. Deflections and temperatures at several locations in the beams were measured with tensile forces in the prestressing steel bars. The test results are expected to be used for development of performance-based fire resistant design of prestressed concrete members.

***Keyword**: prestressed concrete; fire test; flexure; stiffness; residual structural performance*

INTRODUCTION

Performance-based fire resistance design concept can provide fire resistance required for a building structure. In the last few decades the design method has been widely used for concrete structures due to its cost-effectiveness and flexibility. Several national building codes (Sweden, Norway, New Zealand, Australia, etc.) have already been revised to follow this design concept. However, in Japan a prescriptive design method is still applied for evaluating fire resistance of prestressed concrete (PC) building structures. In the code, concrete cover thickness is specified to ensure the maximum temperature of prestressing steel to be attained be lower than a critical temperature which reduces the yield strength to their initial stress at room temperature. It does not include a deflection limitation for PC members in fire, which is one of the fire resistance performances. Developing a more reasonable design method is needed.

What structural designers do is to provide the minimum concrete cover to prestressing steel specified in Standard for Structural Design and Construction of Prestressed Concrete Structures published by AIJ or Architectural Institute of Japan. The specification is based on experiments on prestressed concrete beams, which were carried out a few decades ago. Fire resistance tests on prestressed concrete members have not been conducted since those experiments although tests have been carried out on conventional reinforced concrete members, especially constructed of high-strength concrete. High-strength steel such as prestressing steel is considered to be more

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Ultimate Flexural Strength Evaluation for Prestressed Concrete Columns

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SUMMARY

Four ultimate flexural strength evaluation methods are investigated by comparing their results with experimental data of past research on precast prestressed concrete (PCaPC) columns assembled by post-tensioning. *Method A* and *B* using the plane section assumption predict the experimental results with good accuracy. However, there are some specimens whose flexural strengths are overestimated. In *Method C* assuming yield of post-tensioning (PT) bars, results are inconsistent because it cannot take the axial compression force into account. The results of *Method D* are more consistent than those of *Method C* because of considering influence of axial force. The results are independent on the length-to-depth ratio provided that the bond strength was taken into consideration.

Keyword: *prestressed concrete; post-tensioned; effective prestressing force; length-to-depth ratio; reinforcing index; axial load level; strain compatibility*

INTRODUCTION

In flexural strength evaluation of prestressed concrete columns, the plane section analysis with stress-strain relationships of concrete and reinforcement is recommended. The analysis is, however, complicated and time-consuming in practical design. An approximate calculation[1)] assuming that all PT bars yield and an equivalent rectangular stress block is substituted for the stress distribution of a compressed concrete is generally used in practice. However, all PT bars do not always yield especially under axial compression force on the columns. When the ultimate flexural strength of a column cross section with multi-layered PT bars is predicted by the approximate calculation, the problem is how to take PT bars in the compression zone of the section into account. The authors modified[2)] the approximate calculation originally for a beam section with multi-layered PT bars, in which the tensile force of PT bars in the compression zone is accounted[3)].

There is a little experimental research about prestressed concrete columns in the past (ex. Ref.4)-9)), in which the prediction of ultimate flexural strengths were obtained using the plane section assumption with the ACI equivalent rectangular stress block (ACI method[10)]). There are only few discussions on the prediction accuracy of the evaluation method based on the comparison with experimental data.

In this paper, several flexural strength prediction procedures are investigated in terms of their accuracy by comparing their results with experimental data in the literatures. The experimental data are obtained from flexural loading tests on post-tensioned prestressed concrete columns.

Bond property between PT bar and grout mortar has a significant effect on flexural strength development. A new flexural strength prediction approach to take the bond property into account is proposed and compared with the experimental results.

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