Dynamic Inelastic Numerical Simulation for a Shaking Table Test of a Full Scale Steel Moment Frame Structure based on OpenSEES

Chen Xuewei[1] Han Xiaolei[1,2] Jack CHEANG[1] Lin Shengyi[1] Mao Guiniu[1]

[1] *Tall Building Structure Research Institute, South China University of Technology, Guangzhou 510640, China*
[2] *State Key Laboratory of Subtropical Architecture Science, South China University of Technology, Guangzhou 510640, China*

ABSTRACT :

This paper presents a Dynamic nonlinear numerical simulation of a 4-story full-scale steel moment frame structure based on OpenSEES. Fiber model which has fewer DOFs is used for structure member simulation. The responses of the structure under frequent, moderate and severe ground motions are predicted prior to the full-scale shake-table test. Correlated with the experimental results, the analytical results confirm the accuracy of the numerical simulation method, in both representing the inelastic behavior and the failure mechanism. The seismic responses and the collapse moment under severe ground motion are precise predicted. The study establishes the validity of the inelastic analysis method of overall structure based on fiber model, which is suitable for seismic analysis of overall structure.

KEYWORDS: Inelastic Time-History Analysis, Steel Frame, Full Scale Specimen, Shaking Table Test, Blind Analysis

---

1. FOREWORD

A shaking table test of a full scale steel frame was carried out on E-Defense, the world’s largest earthquake simulator, in Japan on September 27th 2007. A Bind Analysis Contest[1] was held prior to the test. The contest required the participants to predict the responses of a full scale steel frame in a shake-table test and submit the results of the analysis, including responses of the structure under different ground motions and the collapse moment of the frame. the accuracy of the analysis result submitted would be valued by the data of the test. The paper will focus on the statements of the shake-table test and the method of inelastic numerical simulation based on the fiber model.

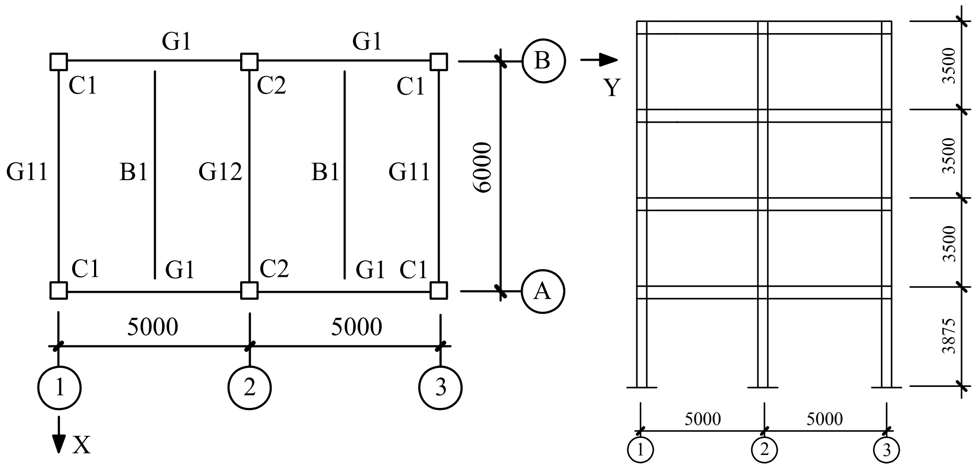
2. FULL SCALE MODEL

2.1 E-Defense Shaking table

The world’s largest earthquake simulator, E-Defense shaking table[2], was built by the National Research Institute for Earth Science and Disaster Prevention of Japan (hereinafter NIED) on January 15th 2005. The size of E-Defense is 20m×15m. The max capacity of the shaking table in vertical direction is 12000kN and the max acceleration in X&Y direction is 900 cm/s^2, while 1500 cm/s^2 in Z direction.

2.2 Full Scale Steel Frame

The specimen of the test is a 4-story full-scale steel moment frame structure with profiled steel sheet and concrete composite floor. The plane dimension of the frame is 10m×5m as shown in Fig.1 (a). There are two spans in Y direction and one span in X direction. The structure is mainly loaded in Y direction. The foundation elevation is 1.5m. The height of the first story is 3.875m, while 3.5m for 2nd-4th story. The height of the parapet is 0.6m. The total height of the structure is 16.475m. The plane layout is shown in Fig.1 and the section and material of each member are listed in Table.1.



(a) Plane Graph (b) Elevation view of axis A

Figure.1 Framing Plan and Elevation of the Specimen

Table.1 Sections and Materials of the Steel Frame

	Beam(SN400B)			Column(BCR295)
Story	G1	G11	G12	C1,C2
4	H-346×174×6×9	H-346×174×6×9	H-346×174×6×9	RHS-300×9
3	H-350×175×7×11	H-350×175×7×11	H-350×175×9×14	RHS-300×9
2	H-396×199×7×11	H-400×200×8×13	H-400×200×8×13	RHS-300×9
1	H-400×200×8×13	H-400×200×8×13	H-390×200×10×16	RHS-300×9

The structure is connected to the shaking table rigidly and the influence on the superstructure caused by the deformation or failure of the connections is ignored, even the superstructure reaches the state of failure, the base connections remain elastic. The columns are fixed to steel supports with sixteen M22 and eight M36 anchor bolts. The steel supports are connected to the shaking table with twenty M48 foundation bolts rigidly.

For the purpose of saving the framework and the convenience of construction, profiled steel sheet and concrete composite floor is adopted for the floor system. The profiled steel sheet can be used as the concrete framework.

2.3 Ground Motion Input

The ground motion recorded in Takatori recording station in south of Hyogo in Japan on January 27th 1995 is taken as the input ground motion in the shaking table test. The magnitude of the ground motion is Ms=7.2 while the epicenter located under the sea between Kobe city and Awaji Island and the focal depth is 17,27 kilometers beneath the earth. The ground motion, which lasts for 41 seconds can be classified as shallow earthquake according to the records. The max acceleration recorded in N-S and E-W direction is 0.606g and 0.657g respectively while that of the vertical direction is 0.279g. During the input process, the ground motion in N-S, E-W and vertical direction are taken as the X, Y, Z direction respectively.

3. INTRODUCTION TO OPENSEES AND THE MOEDL

3.1 Introduction to OpenSEES

OpenSEES stands for Open System For Earthquake Engineering Simulation[3]. It is an open-source system of earthquake simulation and allows the customers add new material constitutions and new types of elements to the system by means of programming. OpenSEES is mainly applied to analysis the responses of the structure in earthquake and it can carry out the linear、static and dynamic nonlinear analysis and the eigen-value calculation, etc. The program has a large library of finite elements and the element employed for nonlinear analysis in this paper is identified as nonlinear beam column, which is based on fiber model and could take the P-Delta effect into consideration.

3.2 Material Constitutive Model

The Kent-Scott-Park constitutive model[4],[5] is adopted as the constitution of the concrete in the paper. Its hysteretic stress-strain curve is shown in Fig.2. The tension behavior is ignored in the Kent-Scott-Park constitutive model. The degradation of strength in unloading process and the press-bearing ability after crack formed could be reflect by this model. The constitutive model is obtained by regression analysis through massive experimental results and convenient for application in engineering for its simple form.

The Giuffré-Menegotto-Pinto[6],[7]constitutive model is adopted as the constitution of steel, as shown in Fig.3, which can simulate the behavior of stiffness degradation and buckling of steel well. This model was firstly put forward by Menegotto and Pinto and modified by Filippou, taking isotropic strain hardening into account. It is effective in calculation due to the explicit form of strain function. Meanwhile, it matches well with the results gained by cyclic loading test of steel.

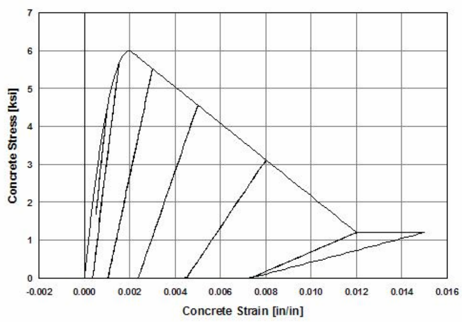


Figure.2 Kent-Scott-Park Concrete Model

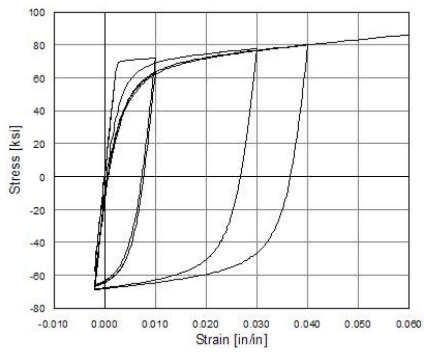


Figure.3 Giuffré-Menegotto-Pinto Steel Model

3.3 Fiber Beam-Column Element

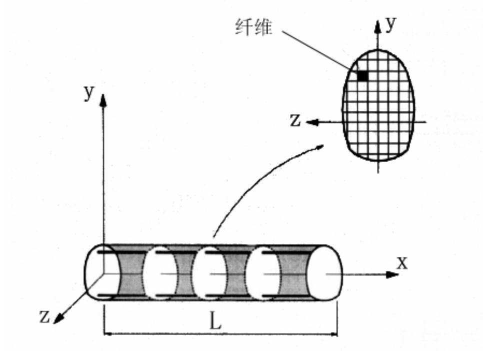
Basic assumptions of fiber model are as follows:

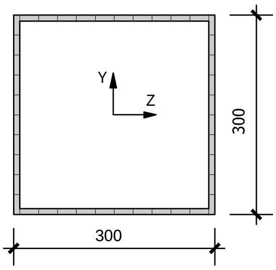
1.Plane section assumption is adopted;
2.Fibers are fully bonded and have no relative slip;
3.Shear deformation is ignored.

The main idea of fiber model is to divide element section into certain small parts, namely, fiber. The stress-strain relationship of overall section, by which a better treatment for the problem of biaxial bending coupling with axial force could be gained, could be calculated according to uniaxial stress-strain relationship of the fiber.

With the fiber model method, there are n integration points by which stiffness matrix could be calculated along longitudinal division in each element of the structure. Each integration point is subdivided into several fibers, as shown in Fig.4. Calculation accuracy can be satisfied so long as sufficient cross-section subdivisions and correct constitutive model of materials. As to different materials, different uniaxial constitutive models could be assign to the fibers according to their location and area. As to the same material, different uniaxial constitutive models could be assign to the fibers which have different mechanical behavior due to different lateral restraints, such as the restraints form stirrup, steel tube and carbon fiber sheet. It is convenient to define the sections of the model by the fiber model which is unaffected by materials and section shapes.

An inelastic 3D time history analysis for the steel frame is carried out using OpenSEES v1.7.2. The beams and columns are simulated with fiber elements, taking the P-Delta effect into account. In order to simulate the buckling of the steel columns, the elements of steel columns need to be subdivided[8]. The fiber sections of beam and column elements are subdivided respectively ,as shown in Fig.5 and Fig.6 .





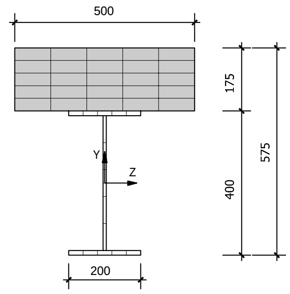


Figure.4 fiber element in local coordinates Figure.5 Fiber Section of Column Figure.6 Fiber Section of Composite Beam

Before the dynamic test of the structure, the low cyclic loading experiments for the steel members, including steel beams、composite beams and steel columns are carried out by NIDE in order to understand the seismic performance of the steel members. The paper builds a numerical model using fiber elements for low cyclic loading test and compared the analytical results with results obtained from experiments. The comparison shows that the analytical results match well with the experimental ones even without any adjustment of material properties. The properties of capacity and ductility of the composite beams need to be adjusted in order that the results after adjustments can basically agree with the experimental results. The model for the simulation and the comparison of the results are shown in Fig.7 and Fig.8 respectively.

The simulation of the overall structure is in progress after the material properties and the member model are determined, as shown in Fig.9. The specific mass data of the model is provided by NIDE. The frame is simulated both in OpenSEES and SAP2000. The modal analysis is carried out and the periods calculated by SAP2000 are listed as following: T1=0.7313s，T2=0.6973s，T3=0.5322s; while the ones obtained in OpenSEES are T1=0.7226s， T2=0.6906s， T3=0.5281s. The results are basically coincidence.

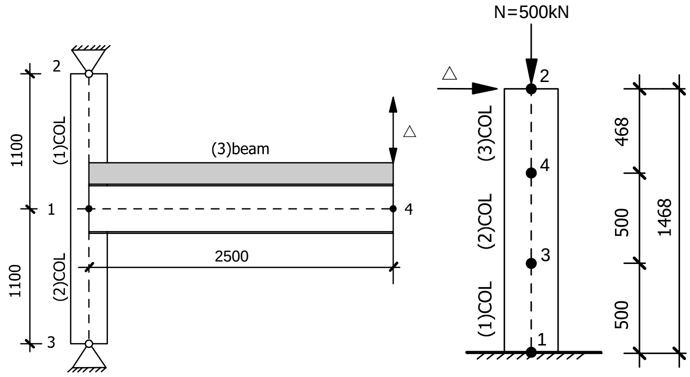
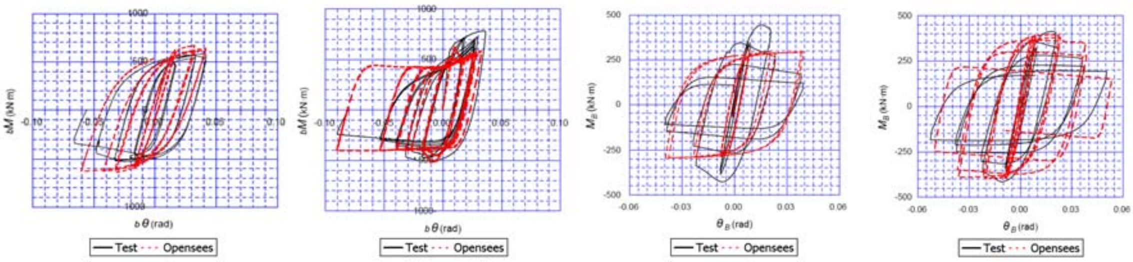


Figure.7 Simulation of the Steel Beam and Composite Beam under Cycle Loading



(a) steel beam (b) composite beam (c) steel column (0°) (d) steel column (45°)

Figure.8 comparison of M-θ curve

(a)Modal1 (b) Modal2 (c) Modal3

Figure 9 Modals of Structure

4. INELASTIC ANALYSIS OF THE STEEL FRAME

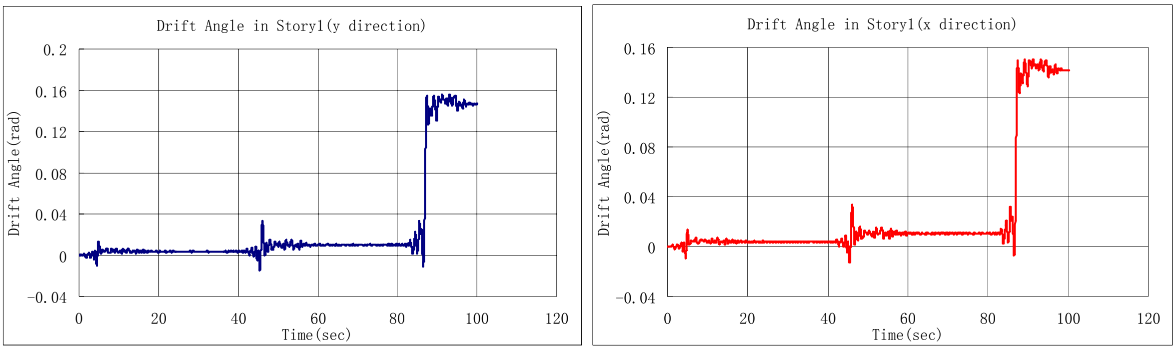
4.1 Analysis Description

There are three procedures of the analysis: 0.4×Takatori, representing the frequent seism; 0.6×Takatori, representing the moderate seism and 1.0×Takatori for severe seism. Rayleigh damping is adopted in the numerical model and the damping coefficient for 1st and 2nd modal are set as 0.02. The step size for seismic analysis is 0.02s. The ground motions for time history analysis are input in three directions. The Newmark direct integration method is adopted as the solving method with the coefficients: α=0.5，β=0.25. The energy criteria are employed as the convergence criteria.

The model is three-dimensional with 78 joints and 100 nonlinear beam column fiber elements. The ground motion input is 0.6×Takatori, representing the moderate seism. The step size is 0.02s and the number of the steps is 2500. The PC used for calculation is described as following: CPU: Intel(R) Pentium(R) Processor 1.50GHz; Ram: 760MB. It costs 25 minutes to execute the calculation. It can be concluded that the nonlinear numerical model based on fiber elements has higher calculation efficiency due to fewer DOFs.

4.2 Analysis Results

The 1st story drift angle responses in time history analysis in X and Y direction is shown in Fig.10. The structure is assumed to collapse when the story drift angle reaches ±0.13(rad)≈1/8. According to the analysis in OpenSEES, after the frequent and moderate ground motion, the 1st story drift angle will reach 0.13(rad) at the moment of 6.26s after severe ground motion is input. The collapse moment recorded in the experiment is 6.24s.



(a)Y direction (b)X direction

Figure.10 Curve of Drift Angle Time-History in Story 1

The relative story displacement response under the excitation of 0.6×Takatori is obtain from the experiment, as well as the absolute story acceleration response. The corresponding story shear force and overturning moment are calculated according to the data recorded. The comparisons between the results from OpenSEES and experiment[9] are shown in Fig.11. According to the comparisons, the prediction of the deformation shows little deviation from the test data while the results of absolute story acceleration, overturning moment and story shear are basically match with the experimental records.

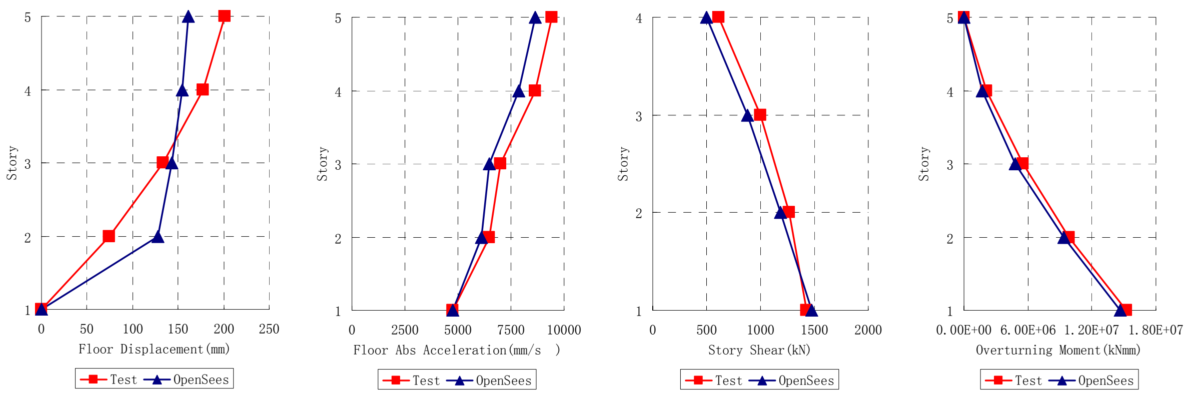


Figure.11 comparison of results form experiment and OpenSEES

The experiment shows that the structure reaches the state of failure under the excitation of 1.0×Takatori. The moment of collapse (t=6.24s) is shown in Fig.12. Obviously, the max story deformation occurs at the first story and the collapse of the whole structure is due to the local buckling at the ends of the columns, as shown in Fig.13. The moment of collapse obtained in OpenSEES is 6.26s in severe ground motion and the animation created by graphic post-process program is shown in Fig.14. According to the comparison, it can be concluded that OpenSEES can simulate the structure response under severe ground motion accurately and it can predict the moment of collapse and the deformation of the structure precisely.

Fig.12 Photo Shot of Collapse Fig.13 Local Buckling of Column End

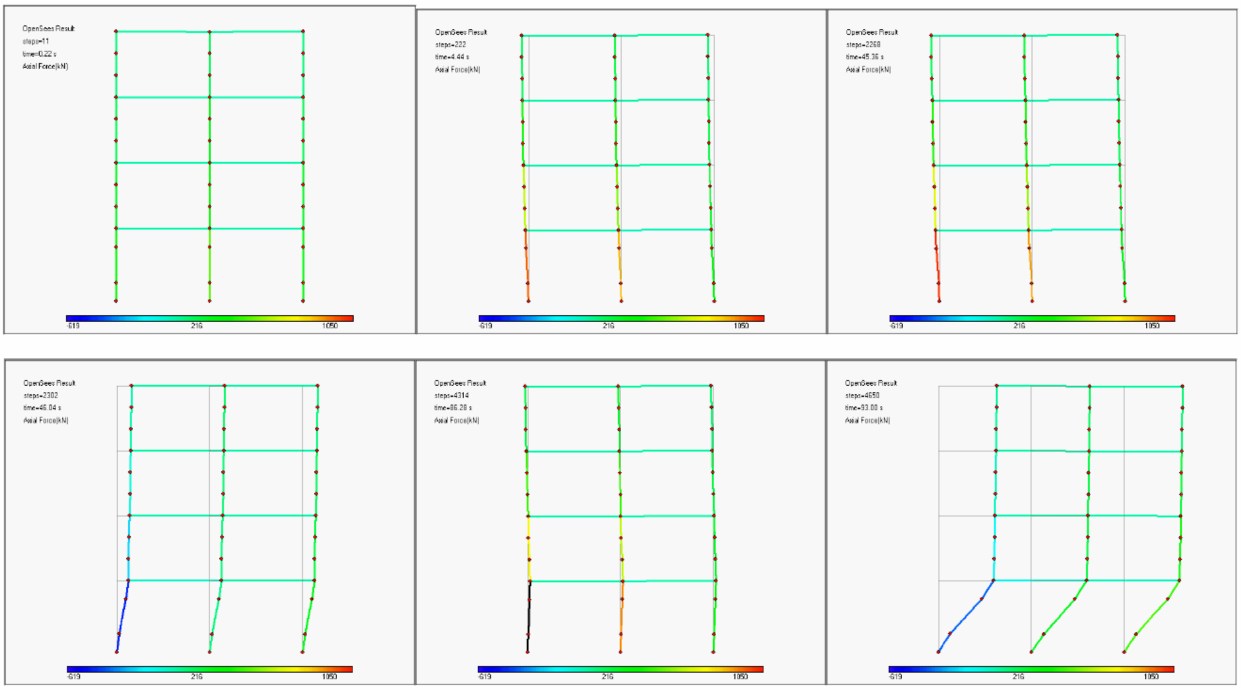


Fig.14 Deformation of Collapse in OpenSees

5. CONCLUSION

A nonlinear analysis of a full scale steel frame is carried out with OpenSEES, a finite element program based on fiber-elements. Prior to the analysis of the whole structure, the author simulates the low cyclic reciprocating load test for the steel beams and steel columns in OpenSEES and compares the analytical results with the data provided by NIED. According to the comparisons, adjustments to the capacity of the composite beam and the ductility properties are made so that the fiber elements can simulate the inelastic behavior of the composite beams accurately. The analysis results of steel members match well with the experimental data even without any adjustment. Local buckling occurs in steel columns under severe ground motion. The simulation of local buckling is realized by subdividing the column elements and taking the P-Delta effect into account. The modal analysis and elastic time history analysis are also carried out and the results are compared with the data from SAP2000 so as to verifying the reasonableness of the model.

After establishing the model of the structure, OpenSEES is used to carry out the static and dynamic inelastic analysis. According to the analysis, the failure mechanism is formed due to the decrease in the lateral stiffness of the structure, which is caused by local buckling at the ends of the columns. There are three procedures of dynamic inelastic analysis, including 0.4×Takatori, representing the frequent seism; 0.6×Takatori, representing the moderate seism and 1.0×Takatori for severe ground motion. The max structure responses obtained in OpenSEES, including relative story displacement, absolute story acceleration, story shear and overturning moment, match well with the experimental data. The collapse moment predicted by OpenSEES and the one recorded in experiment matches well with each other. The analysis indicates that the model can simulate the inelastic behavior and the buckling of the steel frame accurately, and can also evaluate the structure anti-seismic performance precisely. The results of the study indicate that the inelastic analysis method based on fiber-element model can evaluate the inelastic behavior of the structure accurately and has a high efficiency of calculation due to fewer DOFs.

Performance-based seismic analysis method is the direction for the seismic engineering development, the nonlinear analytical method that verified by tests will be the foundation of the performance-based seismic analysis method.

REFERENCES

[1] National Research Institute for Earth Science and Disaster Prevention. (2007). Four-Story Steel Bulding Collapse Analysis Blind Prediction Contest Rules[Z]. http://www.blind-anlaysis.jp/index_e.htm, appendix_e.pdf.
[2] Masayoshi SATO, Takahito INOUE. (2004) General frame work of research topics utilizing the 3D full scale earthquake testing facility[J]. Journal of Japan Association for Earthquake Engineering, Vol.4, No.3.
[3] Mazzoni S, Mckenna F, Scott M H, Fenves G L. (2006) OpenSEES Command Languqage Manual[Z].http://OpenSEES. Berkeley.edu/OPENSEES/manuals/usermanual/OpenSEES Command Languaeg Manual June 2006. pdf.
[4] Kent D C, Park R. Flexural Members with Confined Concrete. ASCE,1971.97(ST7):1969~1990.
[5] B. D. Scott, R. Park, M. J. N. Priestley. (1982). Stress-strain behavior of concrete confined by overlapping hoops at low and high strain rates. No 99-2, p. 13-27
[6] Menegotto, M. and Pinto, P.E.: (1973). Method of analysis of cyclically loaded RC plane frames including changes in geometry and non-elastic behavior of elements under normal force and bending, Preliminary Report, IABSE, Vol. 13, pp. 15–22.
[7] Thomas Telford. (1996).CEB: RC Elements under Cyclic Loading – State of the Art Report.
[8] Murat Dicleli, Anshu Mehta. (2007). Simulation of inelastic cyclic buckling behavior of steel box sections[J]. Computers and Sturctures. 85, 446-457.
[9] National Research Institute for Earth Science and Disaster Prevention.(2007). Blind Analysis Contest 2007 Results of The Contest[Z]. http://www.blind-anlaysis.jp/index_e.htm, appendix_e.pdf.