수평스티프너로 보강된 모멘트 접합부의 변형능력

Deformation Capacity of Existing Moment Connections retrofitted with Horizontal Stiffeners

오상훈[*] 김영주[**] 문태섭[***]
Oh, Sang Hoon Kim, Young Ju Moon, Tae Sup

국문 요약

최근에 수행된 철골모멘트 접합부의 실험 및 해석결과에 의하면, 기둥으로 각형강관을 가진 접합부가 H형강을 기둥으로 사용하는 접합부에 실험체에 비해서 조기에 스캘럽 단부에서 취성파단이 발생하는 열등한 변형능력을 나타냈다. 이는 각형강관 기둥의 면외변형에 따른 보 웨브의 모멘트 전달효율의 저하, 스캘럽이 가지는 노치효과 및 합성보의 구속효과가 주요원인으로 밝혀졌다. 실험결과는 또한 개선된 수평스티프너로 하부 플랜지를 보강한 실험체는 우수한 변형능력을 가짐을 보였다. 본 연구에서는 이러한 영향을 고려하여, 다양한 수평스티프너로 보강된 접합부에 관한 유한요소해석을 실시하였다. 해석결과를 바탕으로 파일럿 테스트를 실시한 결과, 수평스티프너로 보강된 RBS 접합부 (SR)와 연장된 수평스티프너를 가진 접합부(LH)는 우수한 변형능력을 보였다.

1. INTRODUCTION

Many fractures have been found in the connections of MRFs shaken during the 1995 Kobe earthquakes. Investigation of these fractured connections indicated that most of the fractures occurred at the bottom flange. The fracturing initiated near the weld access hole region has been documented in the laboratory (Okada et al., 2001), which was caused by the slab effects. In Japan, rectangular hollow section (RHS) is generally used for columns and H-shaped section for beams. The contribution of web moment of this type of connection was reduced at the vicinity of joint due to the out-of-deformation of column flanges (Matsumoto et al., 1999). Considering this effect including the slab effects as well as the inevitable geometrical and metallurgical notches near the weld access hole, deformation capacity of the connection may be remarkably reduced. This has been proved in the further research based on the experimental result (Okada et al., 2003). On the basis of these results, Kim et al. (2004a) and Oh et al. (2004) investigated the retrofit methods to develop the deformation capacity of the connection. Test results were as follows: (1) the RBS was not, by itself,

---

[*] Senior researcher, Research Institute of Industrial Science & Technology (RIST).

[**] Graduate student, Department of Architectural Engineering, Hanyang University.

[***] Professor, Department of Architectural Engineering, Hanyang University.

sufficiently improved for the deformation capacity of the connection; (2) both the RBS reinforced by a horizontal stiffener and the RBS shape horizontal stiffener details moved the plastic hinge away from the column face and significantly improved the deformation capacity. Analytical results exhibited that the strain concentration at connection with a RHS column and a weld access hole is influenced by the efficiency in transmitting the moment in the web of beam through the beam-to-column joint (Kim et al., 2004b).

This paper is focused on the retrofitting of pre-Kobe steel moment frame connections using a stiffened RBS and a welded horizontal stiffener. These retrofit methods were considered in only beam bottom flange. A parametric analytical study was performed using nonlinear finite element analysis to improve the connection performance.

2. PARMETRIC STUDY

The general purpose finite element program ANSYS was used for this numerical study. The analysis account for material nonlinearities through classical metal plasticity theory based on the von Miese yield criterion. Isotropic hardening is assumed for monotonic analyses, whereas kinematic hardening is assumed for cyclic analyses. A three-dimensional finite element model was generated to represent a structural subassemblage as shown in Fig. 1.

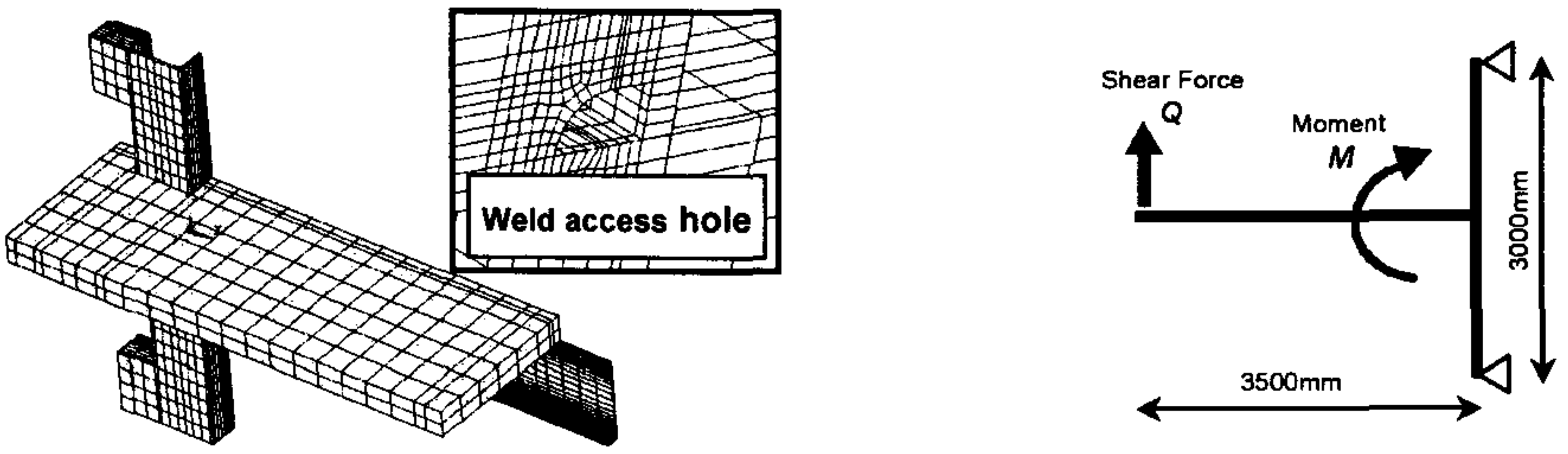
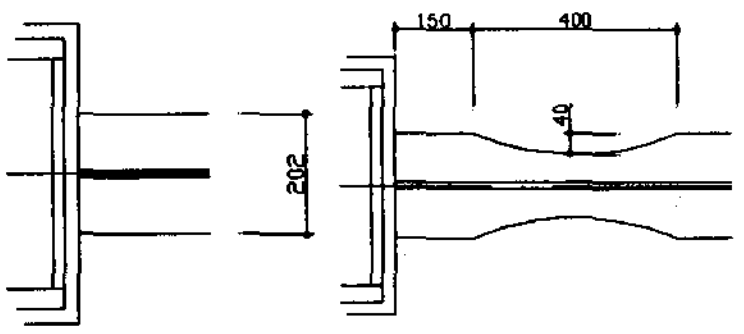


Fig. 1 Finite element model (CT1)

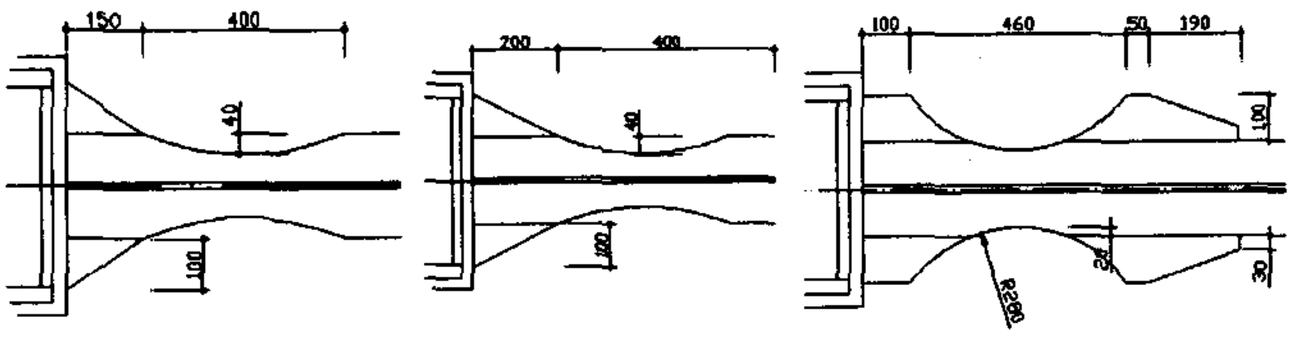
The geometry of the analysis configurations utilized in this research is shown in Fig. 2-5. Key geometric parameters are the presence of reduced beam section (RBS) and the shape (length & width) of horizontal stiffener. A total seventeen specimens were adopted and analyzed, and the models are divided largely into four series as follows: (a) CT series (Conventional Type; CT1-CT3); (b) SR series (Stiffened RBS; SR1-SR3); (c) RH series (RBS type Horizontal stiffener; RH1-RH5); and (d) LH series (Lengthened Horizontal stiffener; LH1-LH4).

As shown in Fig. 2, CT series consist of typical composite beam connection (CT1), bare steel RBS connection which is cut in both top and bottom flanges (CT2), and composite RBS connection which is cut in bottom flange only (CT3). They have also conventional type of the weld access hole in the vicinity of bottom flange (Fig.1). CT1 model is the standard model for comparison with other ones. CT3 is modeled to make clear the effects of the RBS cutout on the strain concentration of composite connections as compared to that of bare steel beam connection model, CT2. SR series consist of three types of stiffened RBS connections. RBS cutouts for SR1 and SR2 models are shown in Fig. 3. RBS cutout was designed by referring to the research by Engelhardt et al. (1998). SR3 model is different from the SR1 and SR2 models by the manufacturing procedure; first provides an RBS cutout and then welds a horizontal stiffener to the bottom flange. The steel design guide (1999) suggested that simple adding an RBS cutout to the beam flanges may not, by itself, be adequate to assure significantly improved connection performance. RH series consist of five of RBS type horizontal stiffeners as shown in Fig. 4. These models are similar to the RSS04 and RSS05 specimens experimented in the previous test, Theses specimens developed a good ductility and an excellent energy absorption capacity. Therefore using RH models, an enlarged plastic zone can be obtained in the pre-selected area. Main parameter is the shape of RBS type horizontal stiffeners. LH series consist of four of lengthened horizontal stiffeners (Fig. 5). Both the lengthened horizontal stiffeners are welded to both side of beam flange and through-diaphragm. The lengthened horizontal stiffener consists of three parts, which are a main reinforced part, a curved part, and an extension. The curved part is intended to provide a smooth transition from the main reinforcement part of the horizontal stiffeners, and to prevent undercutting of the horizontal stiffener plate during manufacture and possible crack initiation. Main parameter is the size of a lengthened horizontal stiffener.



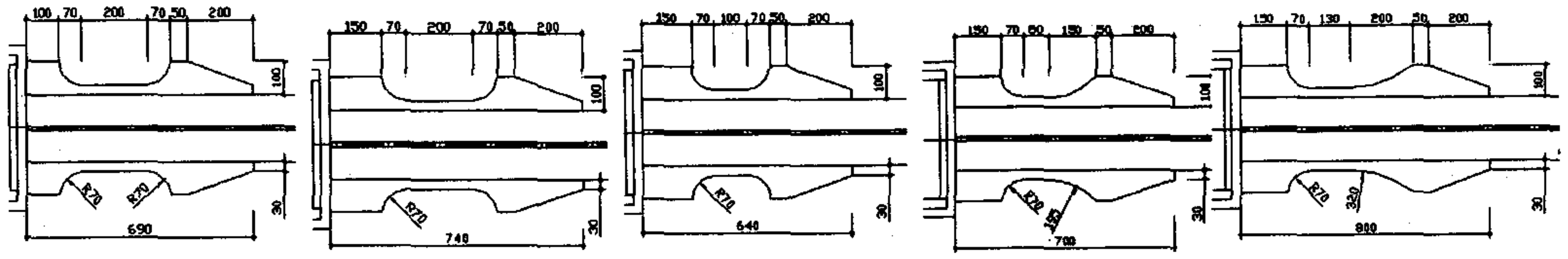
(a) CT1 (b) CT2 & CT3

Fig. 2 Analytical model



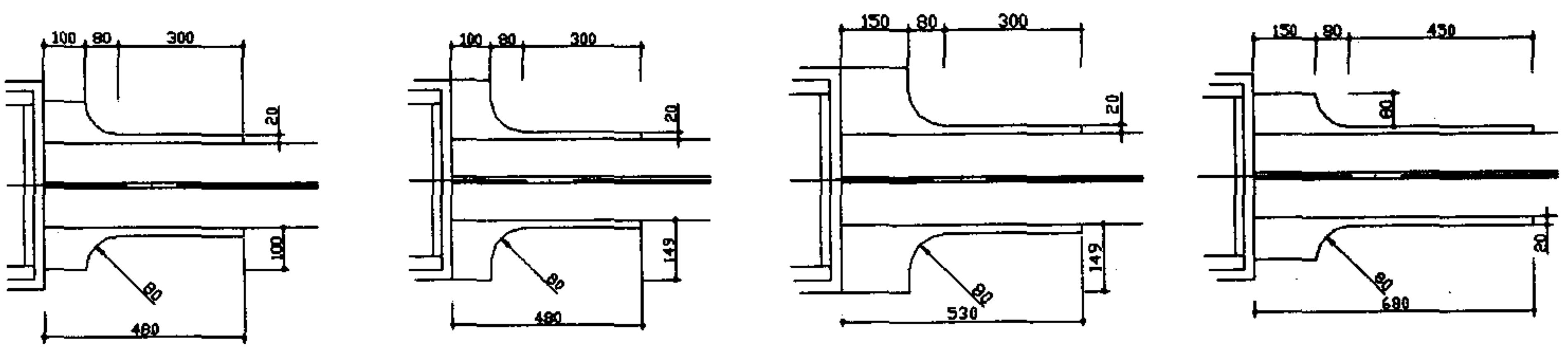
(a) SR1 (b) SR2 (c) SR3

Fig. 3 Analytical model (SR series; composite)



(a) RH1 (b) RH2 (c) RH3 (d) RH4 (e) RH5

Fig. 4 Analytical model (RH series; composite)



(a) LH1 (b) LH2 (c) LH3 (d) LH4

Fig. 5 Analytical model (LH series; composite)

3. ANALYTICAL RESULTS

3.1 Moment vs. Rotation Relationships

Fig. 6 presents the moment versus total rotation curves to examine the global behavior of the connections for all models. Although SR series model were stiffened with a horizontal stiffener, the curves for CT1 and SR series were virtually identical due to the RBS cutout of the SR series. It can be found that as expected, SR series will not increase the flexural moment of the connection as compared to the standard model, CT1. However, the flexural moment RH or LH series increased due to the horizontal reinforcement. The flexural moment of the RH series are about 1.18 times higher than that of CT1 model and the those of the LH series are distributed 1.12~1.15. Focusing on these points, SR series may be suitable for the upper story of steel building, while RH and LH series may be proper for the lower story, pushing to meet the strong column-to-weak beam requirement.

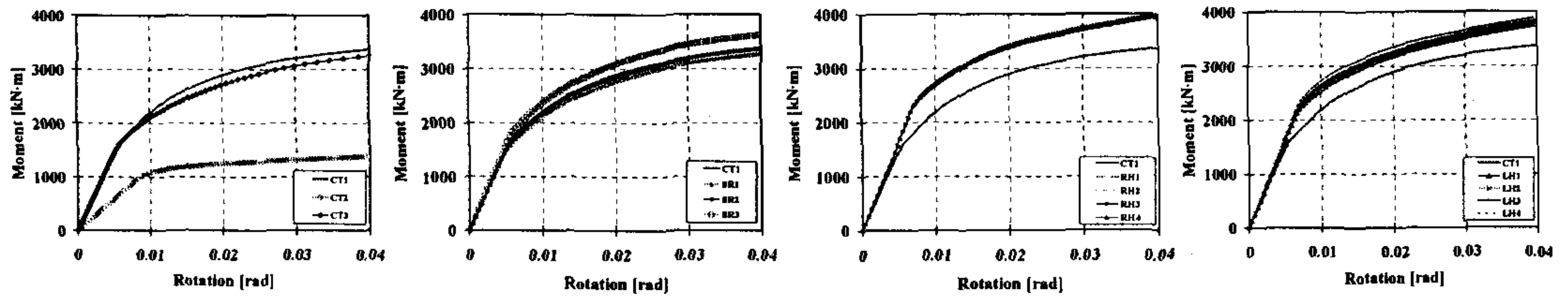


Fig. 6 Moment vs. rotation relationships (LH series)

3.2 Stress Distribution

Analytical results are examined for the stress distribution and spread of the yielding zone in the connection. Fig. 7 plots the von Mieses stress distributions in the bottom flange part of the connections for each model at a total rotation of 4% rad. because AISC (2000) recommended a 4% minimum story drift rotation for the connection of the SMRFs. In Fig. 7(a), the beam bottom flange for CT1 model yielded near the column face because the critical section through which the beam forces were transferred to the connection was the connection-beam interface. Localized stress concentrations were observed in the weld access hole region, owing to the geometric discontinuity, the decrease of the web moment transfer efficiency, and the slab effect at the connection. Additionally, the highest stress was in the vicinity of the weld access hole, from which the beam bottom flange fractured during the test (Kim et al., 2004a). CT2 model is a bare steel model cutout in both the top and bottom flanges, while CT3 is a composite model cutout in the bottom flange only with a floor slab. The stress distributions of both models were different from each other. In the case of CT2 model, the von Mieses stress distribution was the same in compression and tension respectively, especially in the RBS cutout zone. The stress concentration was not observed in beam flange near the column face. On the other hand, although the extensive yielding of the beam bottom flange for CT3 model occurred in the RBS cutout zone, the stress was highly concentrated in the weld access hole region across the beam bottom flange. This shows that addition of supplemental reinforcement at the connection may be needed for an existing composite connection, even though the RBS cutout in bottom flange is adopted.

Fig. 7(b) plots the von Mieses stress distributions in the bottom flange part of the connections for SR series models. This revealed that extensive yielding of the beam section occurred away from the column face, such that the plastic hinge in the beam was developed in the pre-selected zone. No localized high stress concentration was observed in the beam flange because of the effect of RBS reinforced by a horizontal stiffener. These observations clearly indicated that the stiffened RBS leads to the formation of the plastic hinge in the beam section away from the column face. The energy dissipated within the beam section is deemed to be more reliable than that dissipated starting from the face of the column.

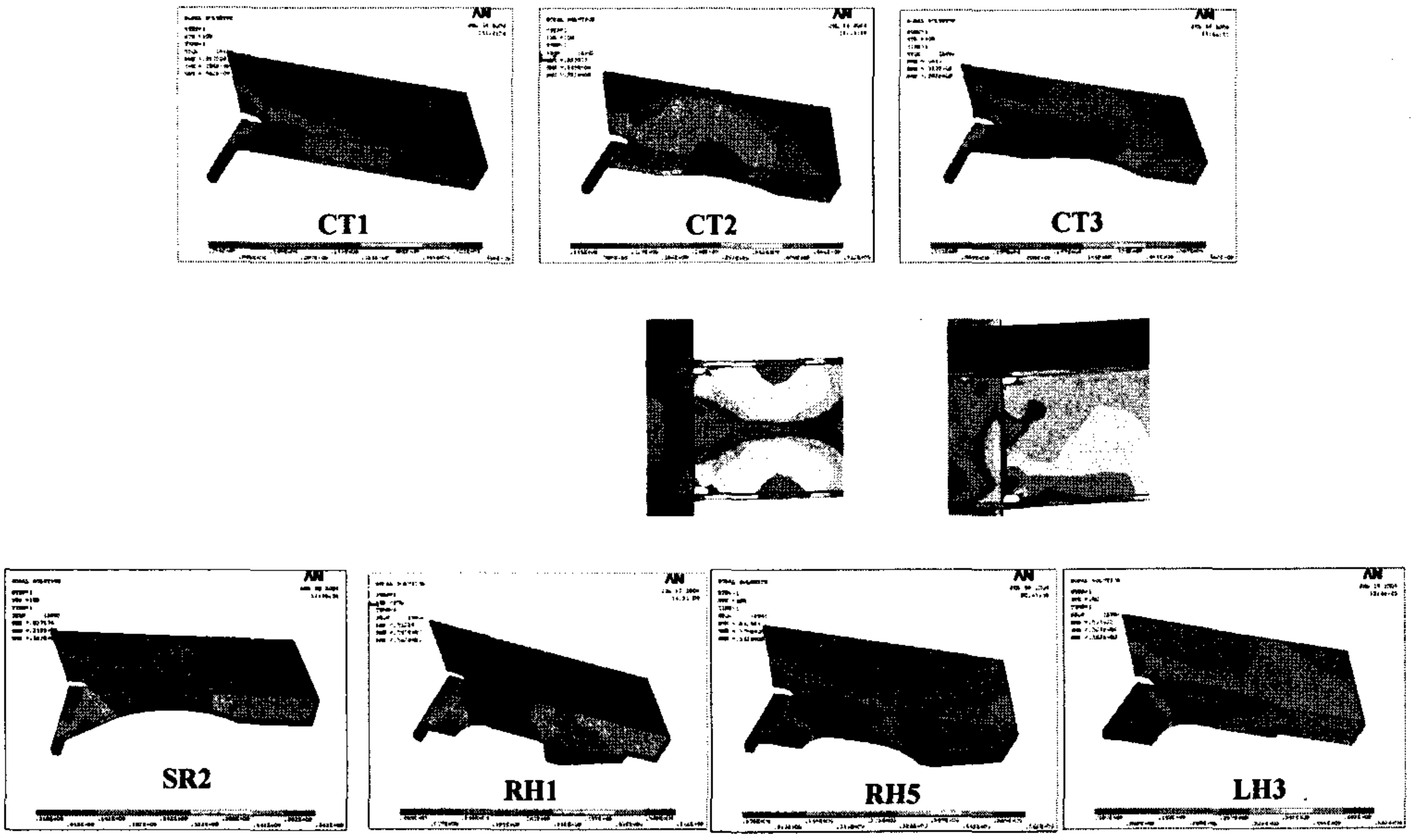


Fig. 7 Von Mieses stress contour

Fig. 7(c) and (d) display the von Miese stress contour in the bottom beam flange for RH and LH series stiffened by the RBS type horizontal stiffener and the lengthened horizontal stiffener, respectively. Both series exhibited behavior as observed in the finite element analysis in the way of forming plastic hinge at a distance far from column face and developing much yielding in the beam flange within the horizontal stiffener part. As addressed in previous study (Oh et al., 2004), this type of specimens, RSS04 and RSS05 developed a good ductility and energy dissipating capacity. Both of the Test and FEM results demonstrated the effectiveness of the RH and LH models for deformation capacity. The reinforcement of the RH and LH definitely reduced the stress demand in the beam flange. Irrespective of stiffener length, no severe stress concentration was observed in the beam-to-column complete joint penetration groove weld.

3.3 Strain Behavior

Fig. 8 illustrates the tensile strain profiles along beam width in the bottom beam flange. The location of measurement was 75mm from the column face, which was located in the line of the toe of weld access hole so that the stress situation near the weld access hole could be investigated in detail.

Fig. 8(a) shows that the tensile strain profile of CT1 model was much larger than those of CT2 and CT3 model. Main reason is due to the effect of the concrete slabs, the presence of the weld access hole, and the lateral deformation of the column flange. This result exhibited that, considering these effects, the neural axis moves toward the top flange when concrete slab is under compression, and consequently this causes the strain on the bottom flange to be much larger than that of the top flange, which leads to premature failure of the connection. Fig. 8(a) also shows that the maximum tensile strain profile of CT3 is about 4.5 times larger than that of CT2. This result exhibits that the RBS retrofit may not be just suitable for composite beam connection, which shows that addition of supplemental reinforcement at the connection may be needed, in the case of composite beam connection, even though the RBS cutout in bottom flange is adopted. The maximum tensile strain profile of CT1 is about 5.8 times larger than those of SR series. Tensile strain profile of SR series is nearly the same as CT2 model which is pure steel connection with RBS cutout. This shows that SR series are effective for reducing the beam flange strain level. SR series may be also advantageous in situations where strong column-weak beam requirement are critical due to RBS cutout. Differ from CT3 model, no localized high stress concentration was observed in the beam flange because of the effect of the stiffeners with RBS cutouts. The maximum tensile strain profile of CT1 is about 2.9 times larger than that of RH1 and is about 3.4 times larger than those of the other RH series. In other words, the maximum tensile profile of RH1 exhibited 15% larger than that of the rest RH series. As shown in Fig. 4, the length of main reinforced part for RH1 is about 50% [100mm] of the beam flange width, while the length of main reinforced part for the rest is about 75% [150mm] of the beam flange width. The strain profiles of LH series tend to be alike those of RH series. The maximum tensile strain of LH1 and LH2 models whose length is about 50% of the beam flange width is slightly larger than those of the rest LH models whose length is about 75% of the beam flange width. The maximum tensile strain profile of LH1 and LH2 exhibited 30% larger than that of the rest LH series. In the case of RH and LH series, therefore, these result shows that to lessen the beam flange strain level, main reinforced part whose length is about 75% of the beam flange width may be needed.

Fig. 9 plots the tensile strain versus rotation relationships. The abscissa is the connection rotation, and the ordinate is the average tensile strain. To evaluate clearly the strain concentration index (γ_ε), the equation ($\gamma_\varepsilon = \Delta\varepsilon / \Delta\varepsilon_0$) which was confined in previous study (Kim et al., 2004b) is used. In Fig. 9, $\Delta\varepsilon$ is strain increment per unit rotation of each model. The strain increment of HN, $\Delta\varepsilon_0$, was also adopted because the moment transfer efficiency of HN developed 100%. In CT series, the strain concentration indices for CT1, CT2 and CT3 are 4.72, 0.85 and 3.07, respectively. The strain concentration index for CT3 is slightly smaller than that of CT1. This result shows that, as discussed above, CT3 model is potential for premature fracture of the connection due to strain concentration at the weld access hole in bottom beam flange and may be needed additional reinforcement. The mean strain concentration index for SR series is about 0.8, which shows the effectiveness of the stiffened RBS model for reducing the stress/strain level. The average strain concentration index for RH and LH series are 1.27 and 1.02, respectively. In RH series, the strain concentration index for RH1 is slightly larger than those of the rest RH series. In LH series, the strain concentration index for LH1 is slightly bigger than those of the rest LH series. As mentioned above, the length of the main reinforcement part for RH and LH series play a more critical role in decreasing the stress/strain level.

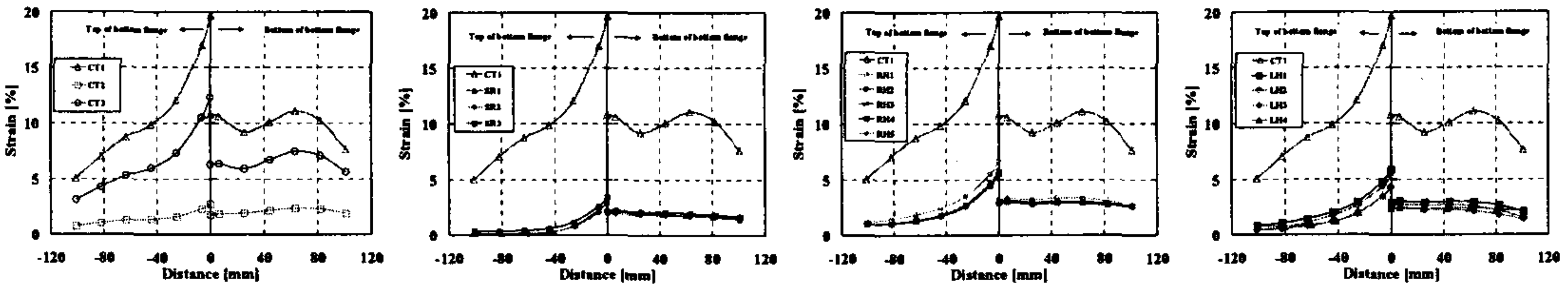


Fig. 8 Tensile strain profiles along beam width

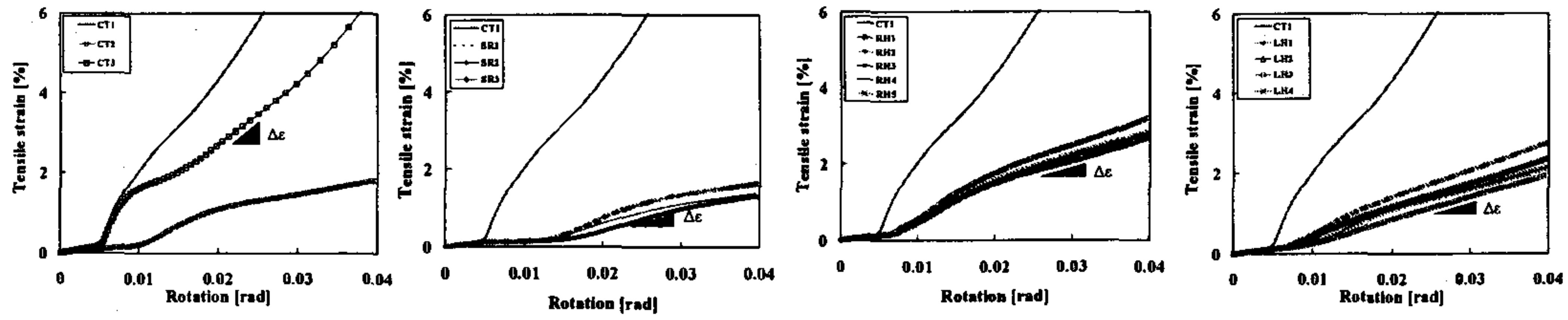


Fig. 9 Tensile strain vs. rotation relationships

4. PILOT TEST

Experimental research has investigated the cyclic behavior of moment connections reinforced with horizontal stiffener. Two pilot test specimens, SR and LH were designed to identify the cyclic behavior. These pilot test specimens consist of H-beam of H-612x202x13x23 and RHS column of RHS-450x450x22. Fig. 10 shows test setup, which simulates the boundary conditions of the subassemblage. Fig. 11 plots moment versus rotation curve for three specimens including conventional type of specimen CT to compare deformation capacity. Both SR and LH specimens with a horizontal stiffener developed satisfactory levels of ductility required of special moment frames in contrast to the Conventional type of specimen CT.

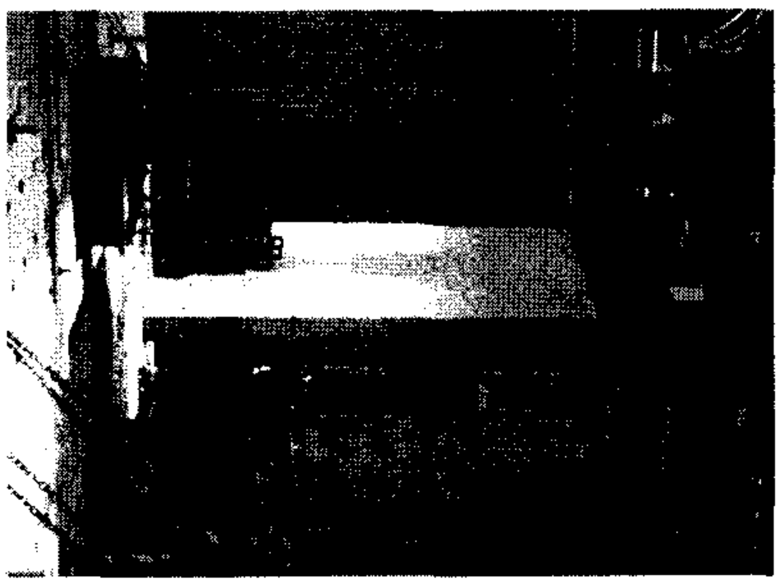


Fig. 10 Test setup

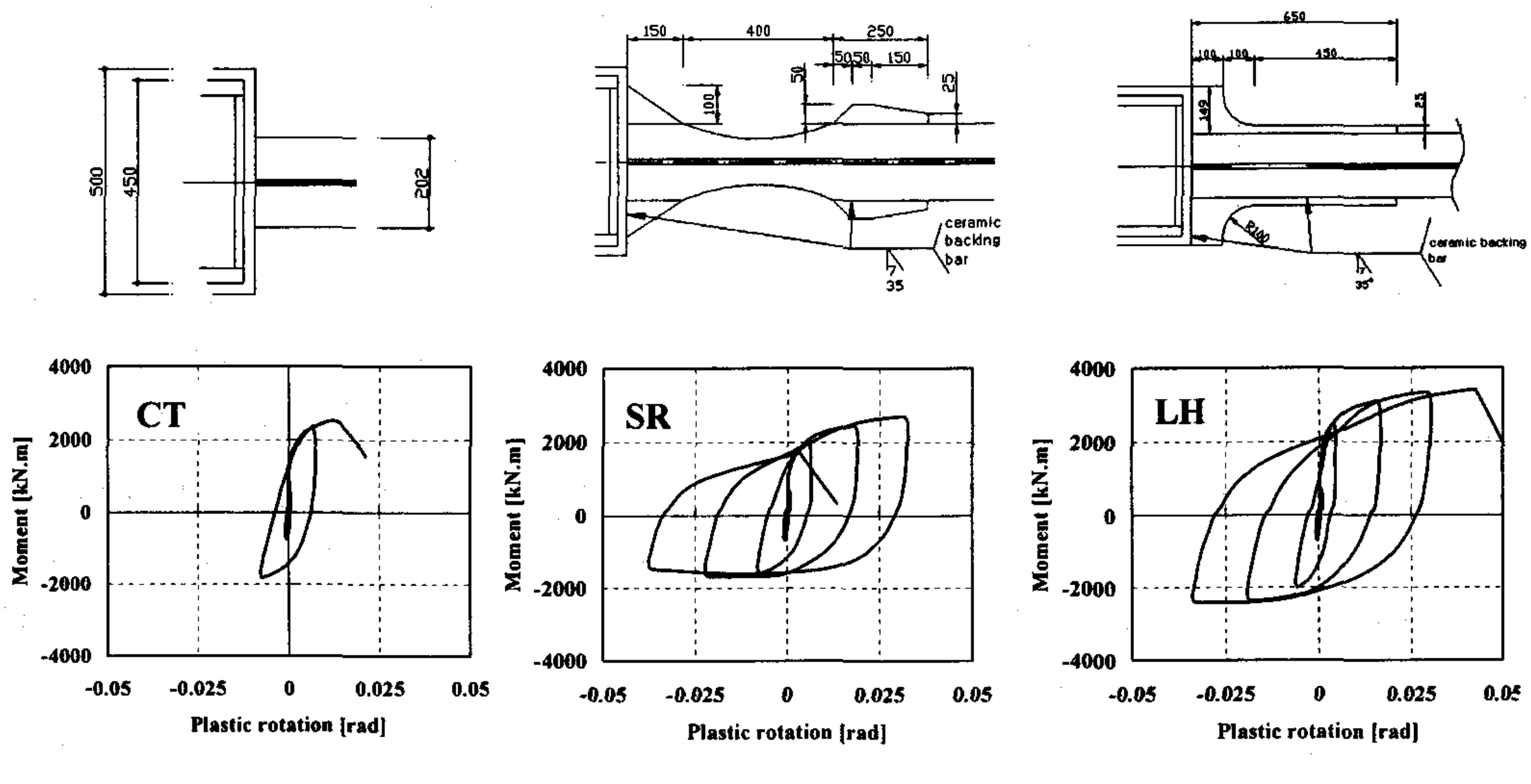


Fig. 11 Moment vs. plastic rotation curves

5. CONCLUSION

As demonstrated in the FEM analysis, in the case of SR, RH and LH series, the plastic hinge mechanism of the beam formed away from the face of the column. The yielding zone of the beam, located in the pre-selected areas, ensured the development of reliable energy dissipating capacity by the extensive yielding of the beam. From the result of the pilot test, it is speculated that moment connections retrofitted with a stiffened RBS (SR) and a lengthened horizontal stiffener (LH) are effective for achieving a more reliable connection performance even in composite connections.

ACKNOWLEDGEMENTS

This research (03R&D C04-01) was financially supported by the Ministry of Construction & Transportation of South Korea and Korea Institute of Construction and Transportation Technology Evaluation and Planning, and the authors are grateful to the authorities for their support.

REFERENCES

1. K. Okada, S.H. Oh, S. Yamada, T. Imaeda, M. Yamaguchi and A. Wada. (2001). Experimental study on deformation capacity of composite beams with conventional type beam-to-column connections. Journal of Structural Construction and Engineering, AIJ, 547(9): 161-168
2. Y. Matsumoto, H. Akiyama and S. Yamada. (1999). Relationship between deformation capacity of beam at steel beam-to-column connection and joint efficiency. J. Struct. Constr. Eng., AIJ, 523(9): 117-124
3. K. Okada, S.H. Oh and S. Yamada. (2003). Effect of joint efficiency at beam-to-column connection on ductility capacity of composite beams. J. Struct. Constr. Eng., AIJ, 573(11): 185-192
4. Y.J. Kim, S.H. Oh and T.S. Moon. (2004a). Seismic behavior and retrofit of steel moment connections considering slab effects. Eng. Struct., 26(13): 1993-2005
5. S.H. Oh, Y.J. Kim and T.S. Moon. (2004). Experimental study on seismic retrofit of steel moment connections considering constraint effect of the floor slab. J. Korean Society of Steel Construction, KSSC, 16(2): 247-255
6. Y.J. Kim, S.H. Oh and T.S. Moon. (2004b). Analytical study on relationship between moment transfer efficiency of a beam web and strain concentration at steel moment connections. J. Korean Society of Steel Construction, KSSC, 16(5): 695-703-255
7. M.D. Engelhardt and T.A. Sabol. (1998). Reinforcing of steel moment connections with cover plates: benefits and limitations. Engineering Structures, 20(4-6): 510-520
8. Steel Design Guide Series 12. (1999). Modification of existing welded steel moment frame connections for seismic resistance, AISC