Seismic Design of Reduced Beam Section (RBS) Steel Moment Connections with Bolted Web Attachment

보 웨브를 볼트 접합한 RBS 철골모멘트접합부의 내진설계

이 철 호* 김 재 훈**
Lee, Cheol-Ho Kim, Jae-Hoon

국문요약

최근에 수행된 보 플랜지 절취형 (Reduced Beam Section, RBS) 내진 철골모멘트접합부의 실험 결과에 의하면, 보 웨브를 볼트 접합한 시험체는 보 웨브를 용접 접합한 시험체에 비해서 조기에 스캘럽을 가로지르는 취성파단이 발생하는 열등한 내진성능을 보였다. 실험결과에 의할 때, 이러한 접합부 취성파괴가 발생하는 주요 원인은 웨브 볼트의 슬립과 고전 휨이론에 의한 예측과는 전혀 다른 응력 전달메카니즘에 의한 스캘럽 부근의 응력집중 때문으로 사료된다. 이는 고전 휨이론에 기초한 전통적 보 웨브 볼트접합부의 설계법을 재검토할 필요가 있음을 시사하는 것이다. 본 연구를 통하여 고전 휨이론에 기초한 현행의 보 웨브 설계법에 문제가 있음을 지적하였다. 실험 및 해석결과를 바탕으로 RBS 접합부의 실제 응력전달경로에 부합되는 새로운 보 웨브 볼트접합 설계법을 제안하였다.

주요어 : 철골모멘트접합부, 볼트접합 웨브, 슬립, 취성파괴, 내진설계

ABSTRACT

Recent test results on reduced beam section (RBS) steel moment connections showed that specimens with a bolted web tended to perform poorly due to premature brittle fracture of the beam flange at the weld access hole. The measured strain data appeared to imply that a higher incidence of base metal fracture in bolted-web specimens is related to, at least in part, the increased demand on the beam flanges due to the web bolt slippage and the actual load transfer mechanism which is completely different from that usually assumed in connection design. In this paper, the practice of providing web bolts uniformly along the beam depth was brought into question. A new seismic design procedure, which is more consistent with the actual load path identified from the analytical and experimental studies, was proposed together with improved connection details.

Key words : steel moment connection, bolted web connection, slip, brittle fracture, seismic design

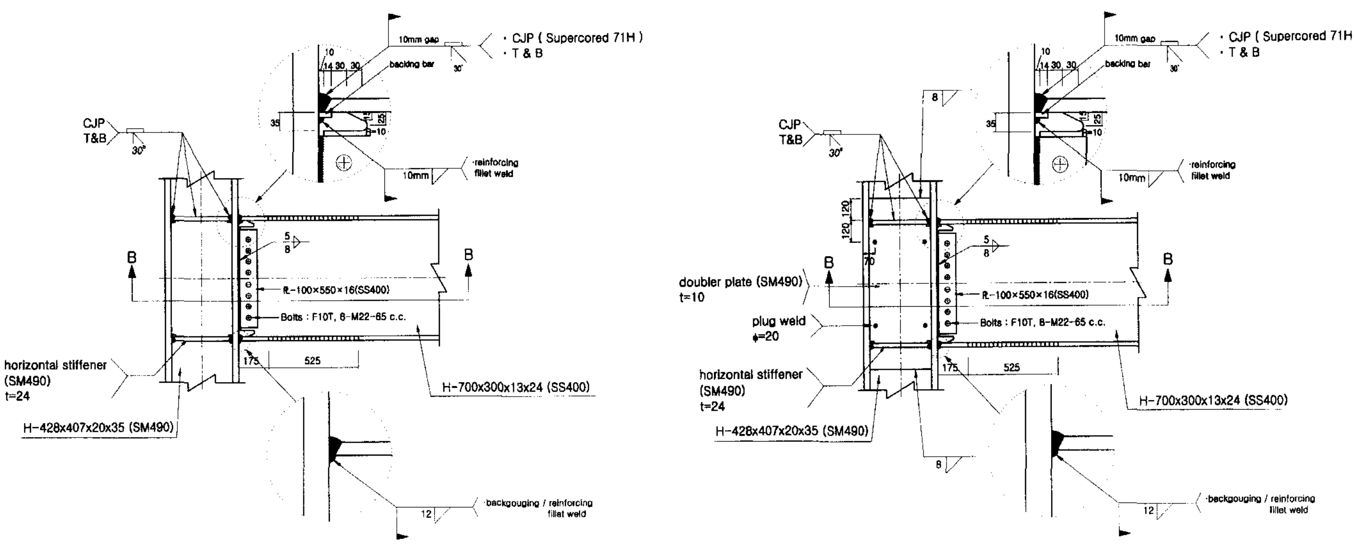
1. INTRODUCTION

The 1994 Northridge and the 1995 Kobe earthquakes caused widespread damages in connections of steel momentresisting frames. After these earthquakes, a number of improved beam-to-column connection design strategies have been proposed. Of a variety of new designs, the reduced beam section (RBS) connection has been shown to exhibit satisfactory levels of ductility in numerous tests and has found broad acceptance in a relatively short time.[(1)-(4)] In the RBS connection a portion of the beam flanges at some distance from the column face is strategically removed to promote stable yielding at the reduced section and to effectively protect the more vulnerable welded joints. Although this type of moment connection has been widely used in the past few years, there remain several design issues that should be further examined.[(5)-(7)] An issue on RBS performance, which requires further examination, is the influence of the beam web connection method. Most of the past tests have been conducted on specimens with a fully welded beam web. While both welded and bolted web specimens have shown good performance, Jones et al.[(5)] indicated that the use of a welded web connection does provide some benefit to the connection performance and it tends to reduce the vulnerability of the connection to weld fracture. In recent test conducted by Lee et al.[(8),(9)] to further investigate the influence of the beam web connection type, bolted web specimens that were slip-critically designed according to the common design practice performed poorly due to premature brittle fracture of the beam flange at the weld access hole. No consensus seems to exist on whether or not a bolted web attachment should be permitted for the prequalified RBS connections. The first objective of this study was to identify the actual load path of the connection. The second objective was to propose a more rational seismic design procedure for bolted web attachment in RBS steel moment connections based on the experimental and analytical studies.

* 정회원, 서울대학교 건축학과 부교수 (대표저자: ceholee@snu.ac.kr)
** 서울대학교 건축학과, 박사과정
본 논문에 대한 토의를 2004년 8월 31일까지 학회로 보내 주시면 그 결과를 게재하겠습니다. (논문접수일 : 2004. 4. 8 / 심사종료일 : 2004. 5. 6)

2. BRIEF REVIEW OF RECENT TEST RESULTS

Recent testing program conducted by Lee et al.[8),(9] is briefly discussed first. The RBS design in the testing program followed the recommendations by Iwankiw[10] and Engelhardt et al.[4] Two bolted web specimens, DB700-SB and DB700-MB (see Fig. 1), were included in the testing program to compare directly the effects of different beam web connection methods. The bolted-webs were slip-critically designed against the expected maximum beam shear following the common design practice. With a slip coefficient of 0.33, the slip-critical bolted web connection consisted of eight fully tensioned-M22-F10T high strength bolts. The bolts were tightened with the calibrated wrench method with a specified tension level of 201kN. The ultimate strength of the bolted web connection was about two times the expected maximum beam shear. The specimens were tested pseudo-statically according to the SAC standard loading protocol.[11] The cyclic responses are presented in Fig. 2 (the following abbreviations were used for the specimen designation: S= strong panel zone, M= medium panel zone, W= welded web, and B= bolted web). Both strong and medium panel zone specimens with a welded web connection developed satisfactory levels of ductility required of special moment frames. On the other hand, specimens with a bolted web connection performed poorly due to premature brittle fracture of the beam flange at the weld access hole (see Fig. 3). Specimens DB700-SB and DB700-MB failed in a brittle manner at the 2% and 3% story drift cycle, respectively. A complete fracture across the beam flange width was developed.



(a) DB700-MB (b) DB700-SB

Fig 1 Bolted web connection details (Lee et al. 2003)

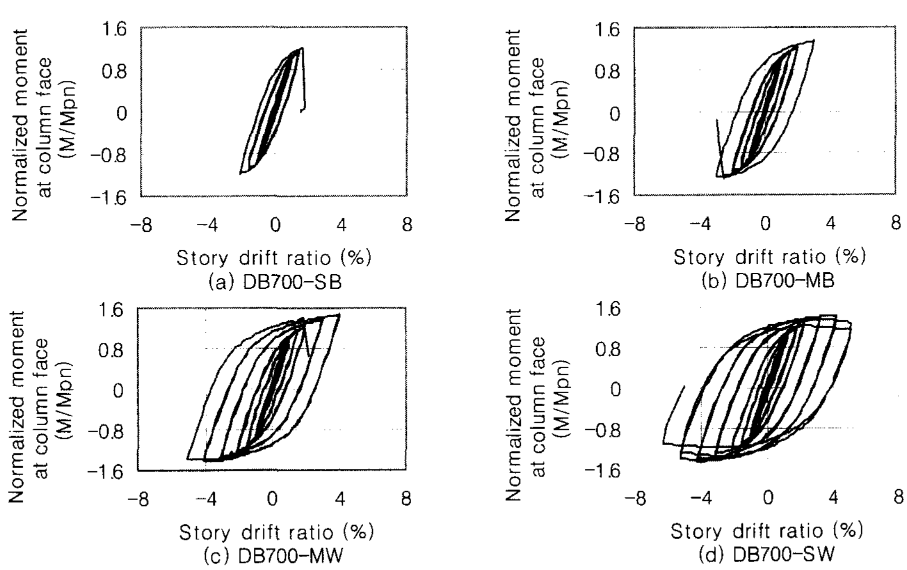


Fig. 2 Normalized moment versus story drift ratio relationship

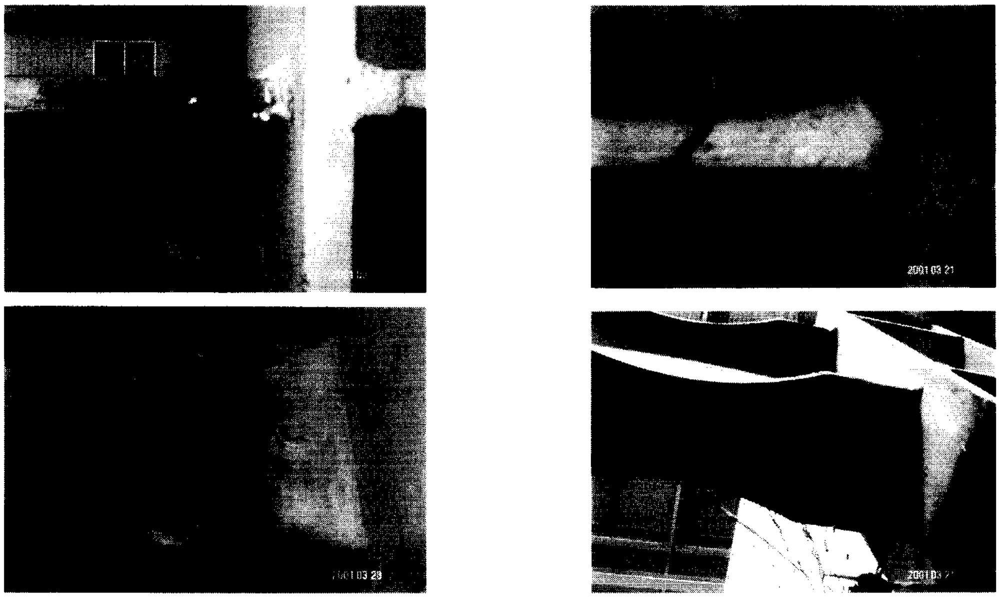


Fig 3 Brittle fracture of beam flange at the weld access hole in specimens with bolted webs

Fig. 4 compares the cyclic flexural strain responses of specimens DB700-SB and DB700-SW near the groove weld of the beam bottom flange up to the fracture point of specimen DB700-SB. Much higher strain demand on the bolted web specimen is evident. These measured results appear to be consistent with the observation by Tsai and Popov.[12] They indicated that web bolts typically slip during testing, leaving the welded flanges alone to resist the total moment. It was also pointed out by Goel et al.[13] that the area in the middle of the beam web near the shear tab is virtually devoid of stresses and much of the shear force is transferred through the beam flanges, thus leading to overstressing of the beam flanges. The measured cyclic shear strain responses are presented in Fig. 5. These measured results support the foregoing observations by Goel et al. Reverse shear occurs in the middle of the beam web. This is undesirable because reverse shear will increase the shear demand in other part of the connection to meet the force equilibrium. It is evident the shear transfer mechanism in the RBS connection is still not consistent with that predicted by the classical beam theory and should be reexamined more thoroughly. It is speculated that other than possible low toughness of the base metal, the higher incidence of base metal fracture in specimens with bolted web attachment is related to, at least in part, the increased demand due to the web bolt slippage and the actual load transfer mechanism which is significantly different from that usually assumed in connection design. Plastic straining of the beam flange can lead to a re-distribution of shear stress. However, plastic straining would concentrate in the RBS region and the shear stress redistribution approaching that of the beam theory is difficult to occur near the column face.

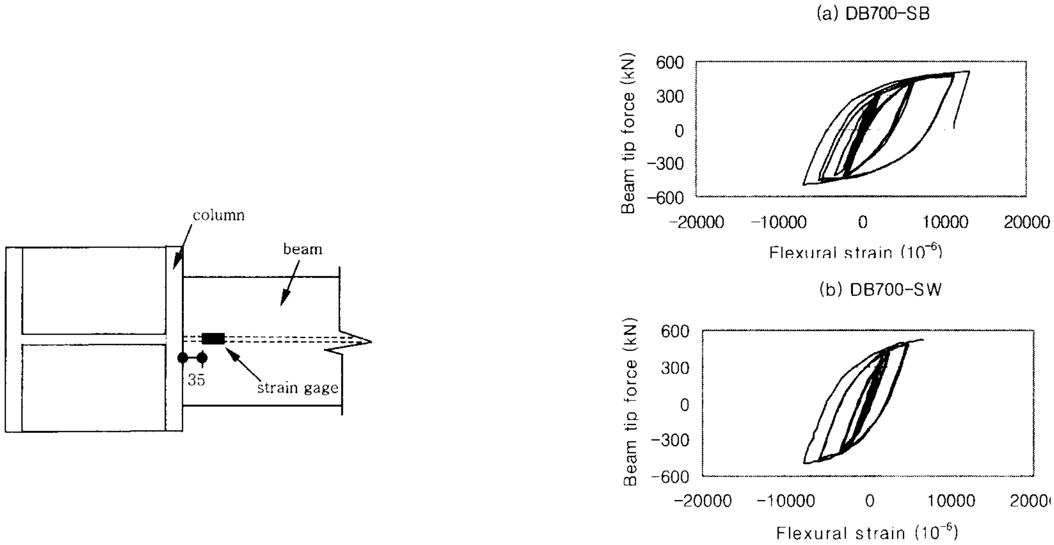


Fig 4 Comparison of measured strain responses near the groove weld of beam bottom flange

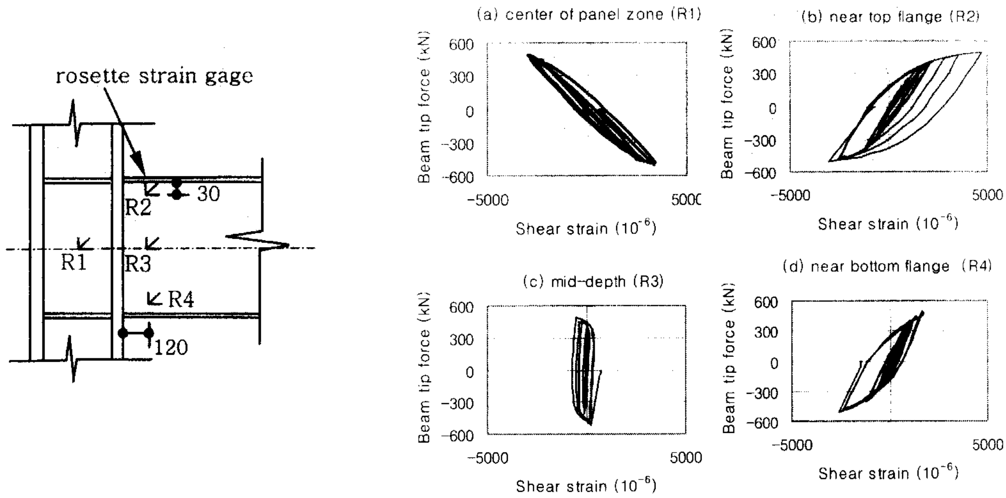


Fig 5 Measured cyclic shear strain responses (specimen DB700-SB)

3. NUMERICAL SIMULATION AND FORCE TRANSFER MECHANISM

To gain further insight into the force transfer mechanism in the connection, the test specimen DB700-MB was modeled and analyzed using the general purpose finite element analysis program ABAQUS.[14] Both the flanges and web of the beam and column were modeled with the 8-node continuum element (element type C3D8I in ABAQUS). A concentrated force of 499 kN was applied at the beam tip to simulate the expected maximum beam shear. Extensive parametric analyses were conducted by varying the length of weld access hole and the shear tap configuration. Fig. 6 shows two shear tap configurations studied. Numerical results showed that, contrary to the expectation from the beam theory, the shear tap transfers just 50% of the beam shear applied and reverse shear occurs around the mid-height of the beam web. This means that significant portion of the beam shear (about 50 % of the total) should be transferred through the beam flanges. Parametric analyses also showed that this trend is rather insensitive to the variation of the shear tap thickness. Thicker shear tap was not so efficient in attracting the force due to increased eccentricity between the beam web and the shear tap. Fig. 7 shows typical principal stress distribution obtained from the finite element analysis. Considering that the middle area of the beam web is virtually devoid of stress, it is believed that the middle portion of the beam web can be unconnected to the column flange without any loss of the connection seismic capacity.

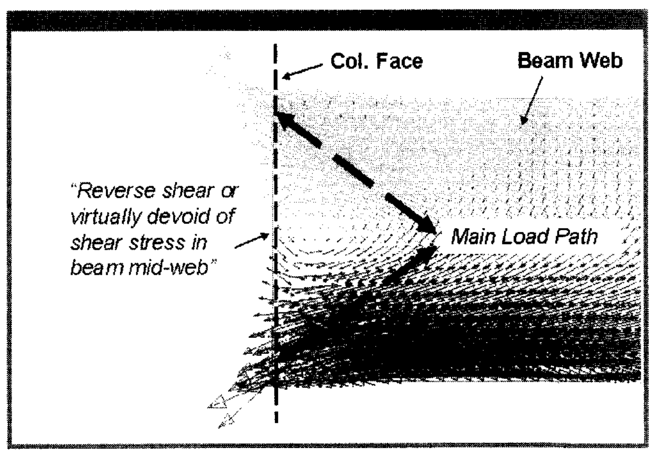
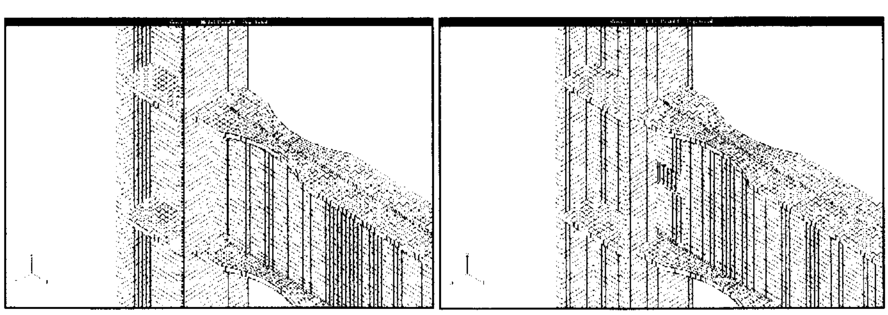


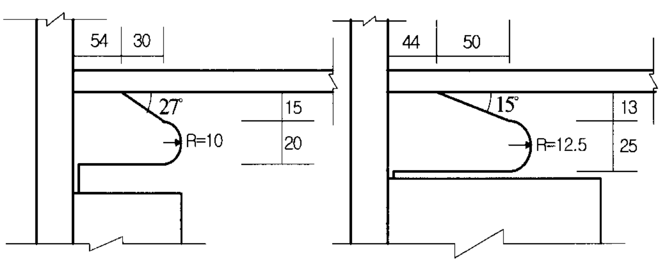
Fig 7 Typical principal stress distribution near the column face

Brittle fracture tended to initiate at the weld access hole, where the stress concentration is the highest and the CVN



(a)Conventional (b) Modified

Fig 6 Two shear tap configurations studied toughness of the base metal is the lowest. Thus, minimization of shear stress concentration around the weld access hole should be a crucial design consideration. The finite element analysis of this study showed that weld access hole geometry with a shallower transition slope can reduce the stress concentration at the weld access hole by 20 % as compared in Figs. 8 and 9. Under typical conditions, the shear tap transfers about 50 % of the beam shear and 10 % of the total moment, respectively. Finite element analysis results also showed that moment transfer through the shear tap can be computed with reasonable accuracy using the elementary mechanics as presented in the next section. Improved beam web attachment schemes as suggested by the experimental and analytical results of this study are presented in Fig. 10. First, beam web bolts are allocated on the principal load path and the middle portion of the beam web is not connected to the column flange. Second, weld access hole geometry with a shallower transition slope is recommended to reduce shear stress concentration. Third, rather heavy shear tap is used to provide secondary load path for moment transfer, thus reducing demand on the beam flange.



(a) Conventional (b) Modified

Fig 8 Weld access hole dimensions

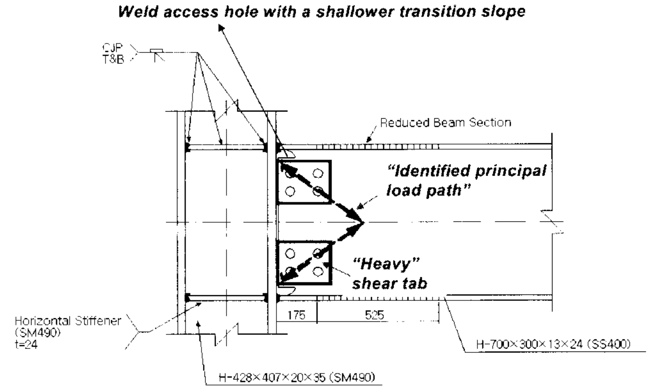


Fig 10 Improved features suggested by the previous experimental and analytical results

4. RECOMMENDED DESIGN PROCEDURE

All the foregoing observations were incorporated in the design procedure proposed in this study. The design procedure is presented in a step-by-step manner in the following.

Determination of Beam Design Shear Force

Based the seismic moment profile (see Fig. 11), the design beam shear can be computed using Eqs. (1) and (2).

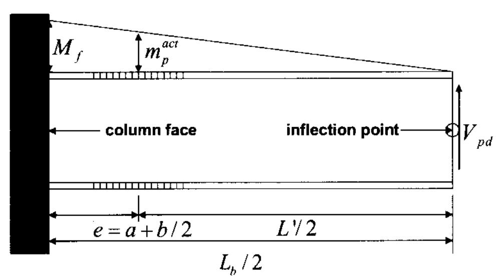
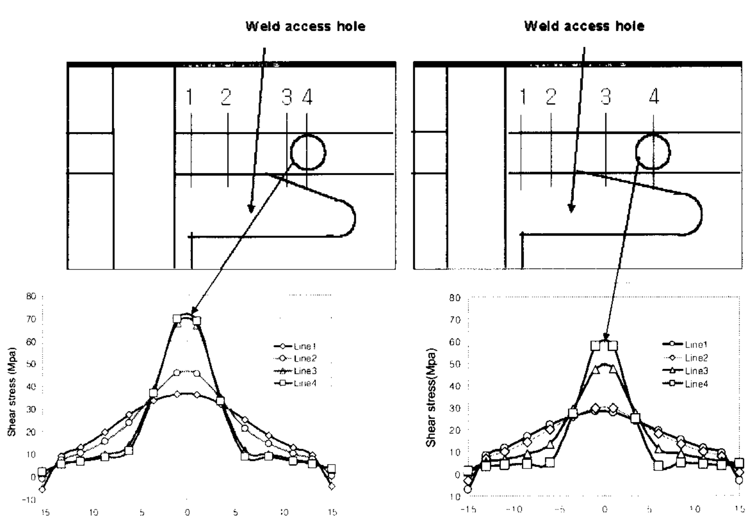


Fig 11 Seismic moment profile for RBS design



(a) Conventional (b) Modified

Fig 9 Comparison of shear stress distribution around weld access hole

$$m_p^{act} = \alpha \times Z_{RBS} \times F_{ye} \quad (1)$$

$$V_{pd} = \frac{m_p^{act}}{(L'/2)} \quad (2)$$

where m_p^{act}= the strain hardened plastic moment at the RBS center, Z_{RBS}= plastic section modulus at the narrowest reduced beam section, and F_{ye}= the expected yield strength of the beam. The strain hardened plastic moment at the RBS is calculated using the expected yield strength of the beam and strain hardening factor of α. Based on the test results conducted by Lee et al. (2003), an α value of 1.25 is recommended for SS400 steel beam.

Determination of Shear and Normal Forces Acting on Shear Tap

Fig. 12 shows the shear and normal force components, V and H, acting on the interface between the shear tap and the column flange. These force components can be computed with reasonable accuracy using Eqs. (3) through (6) in the following.

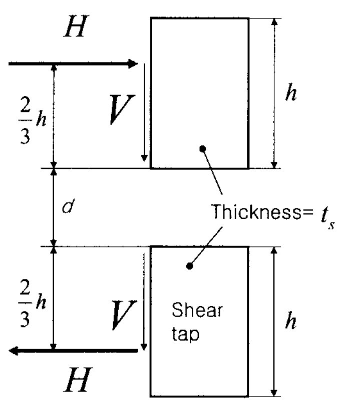


Fig 12 Shear and normal force components acting on the shear tap

$$V \approx \frac{1}{4} V_{pd} \quad (3)$$

$$M_f = V_{pd} \times (L_b/2) \quad (4)$$

$$M_w \approx \left(\frac{I_w}{I_w + I_f} \right) \times M_f \quad (5)$$

$$H \approx M_w / \left(\frac{4}{3} h + d \right) \quad (6)$$

where I_f= the second moment inertia of the beam flanges, I_w= the second moment inertia of the shear tap, M_f= total moment at the column face (see Fig. 11), and M_w= moment through the shear tap. For the remaining symbols, refer to Figs. 11 and 12.

Sizing Shear Tap

The thickness of shear tap, t_s, should be checked based on the Von Mises yield criterion as follows.

$$\sigma^2 + 3\tau^2 = \left(\frac{H}{t_s h} \right)^2 + 3\left(\frac{V}{t_s h} \right)^2 \leq (\phi F_y)^2 \quad (7)$$

$$t_s \geq \sqrt{\frac{\left(\frac{H}{h} \right)^2 + 3\left(\frac{V}{h} \right)^2}{\Phi F_y}} \qquad where \ \Phi = 0.9 \quad (8)$$

Design of Beam Web Bolts

The bolts in the beam web are subjected to both eccentric horizontal and vertical force components. To ensure the force transfer through the beam web in collaborative manner with the welded flanges, the slippage of the web bolts should be prevented against the expected maximum eccentric loading, that is, the bolt group in the beam web should be slip-critically designed against the eccentric loading as shown in Fig. 13. With the known force components (H and V) and bolt arrangement, the shear in the most heavily loaded bolt, F_{max}, can easily be calculated based on elementary statics. F_{max} should be limited to the slip critical load of a single bolt, F_{slip}, or Eq. (9) should be satisfied.

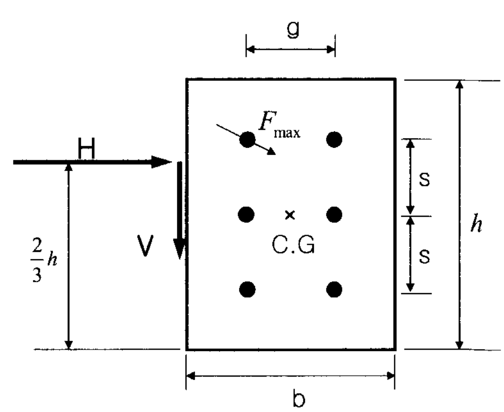


Fig 13 Free body diagram of the shear tap

$$F_{max} = \sqrt{\left(\frac{H}{n} + F_x \right)^2 + \left(\frac{V}{n} + F_y \right)^2} < F_{slip} = \mu N \quad (9)$$

where n= number of bolts, F_x, F_y= maximum horizontal and vertical force components due to eccentricity, μ = slip coefficient, and N= pretension force.

Fig. 14 shows an example of the connection details designed following the procedure described above. The connection in Fig. 14 was designed with assuming L_b= 7200 mm, F_{ye}= 313 Mpa (SS400 steel), and α= 1.25 (SS400 steel). Detailed calculations are omitted here due to space limitations. The required thickness of the shear tap was 16 mm (SM490 steel). With a slip coefficient of 0.40 (shot-blast surface treatment assumed), the slip-critical bolted web connection consisted of twelve fully-tensioned M27-F10T high strength bolts. It is noted that the bolt requirement is much higher than that from the conventional design which ignores the actual force transfer mechanism in the connection. With conventional design practice, eight fully-tensioned M22-F10T high strength bolts were sufficient to meet the slip-critical condition (see Fig. 1). As discussed in Section 3, weld access hole geometry with a shallower transition slope of 15 degrees was used to reduce the stress concentration. To check the adequacy of the slip-critical design procedure proposed in this study, simple finite element simulation of the slip behavior was conducted using ABAQUS. Three-dimensional finite element models for the beam and column subassemblies in Figs. 1 (Conventional) and 14 (Modified) were prepared using eight-node continuum element (C3D8I in ABAQUS). The surface interaction between the shear tap and the beam web was formulated by the Coulomb friction model with elastic slip with 0.0015 % of the characteristic element length. Elastic slip of this minimal amount was required to circumvent numerical convergency problem. Pretension force was simulated by applying a pair of concentrated forces to the front side of the shear tap and the back side of the beam web (see Fig. 15). The main purpose of this analysis was to check whether or not the slip-critical condition is satisfied by either of the two designs. A beam tip force of 499 kN was applied to simulate the beam shear corresponding to the strain hardened plastic moment at the RBS center. Fig. 16 compares the slip response of the two analytical models. It is evident that the conventional design, contrary to the design assumption, can not prevent the web bolt slippage . The modified model designed per the procedure recommended in this study exhibits superior slip resistance. The full-scale pilot test with the connection details shown in Fig. 14 will be conducted to verify the recommended design procedure experimentally.

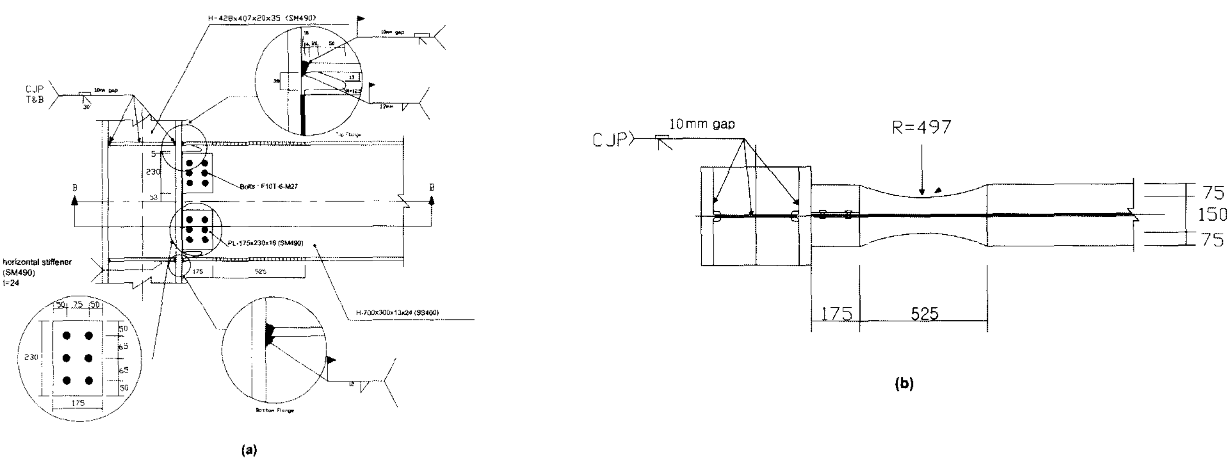
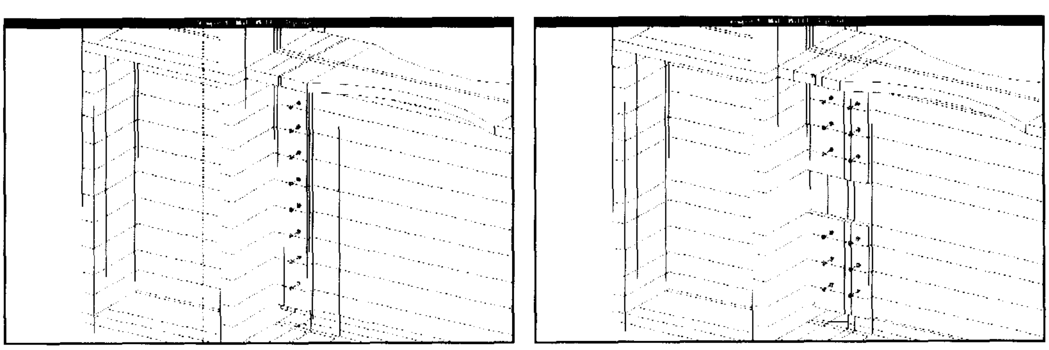


Fig 14 Details of example design: (a) side view (b) top view



(a) Conventional (b) Modified

Fig 15 Simple finite element modeling of slip behavior

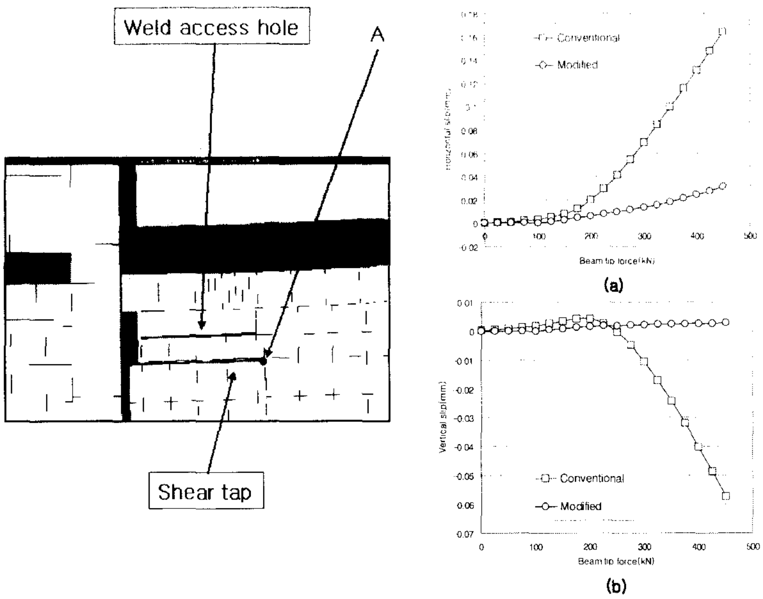


Fig 16 Comparison of slip response at the upper right corner of shear tap (point A): (a) horizontal slippage, (b) vertical slippage

5. SUMMARY AND CONCLUSION

Main conclusions on the seismic design of RBS steel moment connections with bolted web attachment are summarized as follows.

(1) Both the experimental data and numerical results of this study confirm that the actual load transfer mechanism in the connection is completely different from that universally assumed in the connection design. The results of this study suggests that the practice of providing web bolts uniformly along the beam depth based on the beam shear only needs to be reconsidered.

(2) A rational seismic design procedure for RBS steel moment connections with bolted web attachment, which is more consistent with the actual load path identified from the experimental and numerical results, is proposed together with improved details. In the proposed procedure, the slip-critical design is conducted based on the eccentric horizontal and vertical force components at the interface between the shear tap and the column flange. The slip-critical bolt requirement per the proposed design procedure is much higher as compared to that from the conventional design method which ignores the actual force transfer in the connection.

(3) The finite element simulation of the slip behavior in this study showed that the conventional design can not prevent the web bolt slippage, whereas the modified connection following the proposed procedure exhibits superior slip resistance. The full-scale testing will soon be conducted to experimentally verify the design procedure proposed in this study.

ACKNOWLEDGEMENT

Funding for this research was provided by the Korea Earthquake Research Center (KEERC Project No. R11-1997-045-11004-0).

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