



## PERFORMANCE OF BEAM-COLUMN CONNECTIONS IN STEEL STRUCTURES

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### SUMMARY

Many improved details, namely the post-Kobe details, were developed to prevent brittle fractures around the moment connection after the 1995 Kobe earthquake [1]. Some of these details are investigated not only from a view of structural behavior but also discussed from practical points of view, especially cost performance aspects. Details, including welding joints, are categorized for application in the performance-based seismic design.

### INTRODUCTION

Most of low to middle-rise steel buildings in Japan are constructed with moment frames using cold-formed rectangular hollow section columns and H-shaped beams with column-through-diaphragms to connect them. In a moment connection, the stiffness of a rectangular hollow section in the out-of-plane direction is not rigid enough to transfer the stresses occurred in the beam web. This causes the stress concentration in the end of the beam flange, where heavy full-penetrate welds are applied.

In the 1995 Kobe EQ (earthquake) brittle fractures around the welding metal were observed as shown in Fig.1. These were initiated from the hidden notch in the scallop or the crater end of the welding pass. Improved details to prevent these brittle fractures were developed under the consideration of material properties, welding procedures, composite actions with slab, or structural demands for deformation capacities. In these post-Kobe details, the improvement efforts are mainly focused to more careful fabrication or welding procedures, such as non-scallop details, quality control of inter-pass temperature and heat input, or a careful inspection with the records according to ISO system.

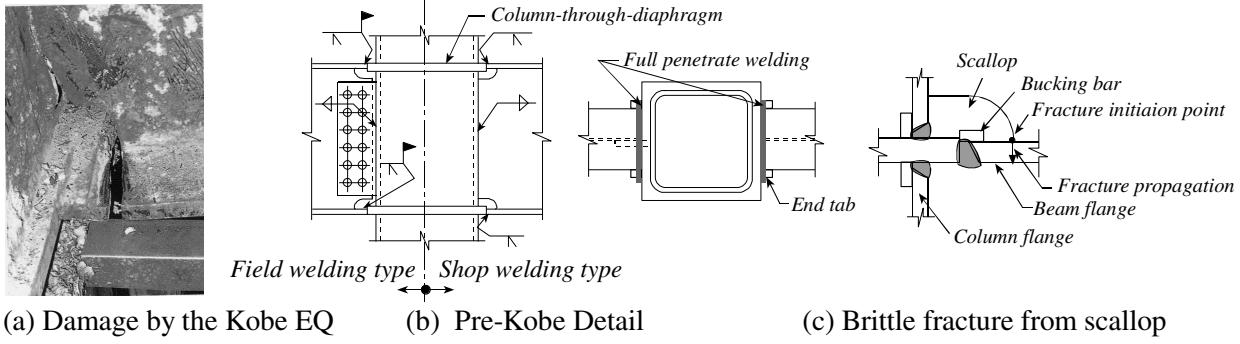
These show the fine contrast with the post-Northridge details in US (United States). The application of traditional beam-column connection details, named as the pre-Northridge type, for a new steel building was restricted after the 1994 Northridge EQ. And improved details such as Reduced Beam Sections (RBS) or some patented details were developed to have sufficient rotational capacities (the SAC criterion is to be experimentally recognized over 3 cyclic-loading-processes of  $\pm 0.03$  radians rotational capacity with sufficient strength [2]).

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**Fig.1 Brittle fractures around the beam-column connection**

These are aimed to shift the maximum stress point from the weld metal to the base metal, mainly because of that most market available wide-flange members are made of less-qualified (especially in weldability) recycled steel. However in Japan, most of steel members are made from blast-furnace process (virgin steel), which has rather better quality (in mechanical and chemical properties, such as notch toughness, less-scattered strength level, or weldability) than recycled steel in US. Therefore improvement efforts were shifted to welding/fabrication procedures, in other words, cost-up in fabrication process.

## PERFORMANCE AND DEMAND OF BEAM-COLUMN CONNECTIONS

### Behavior of beam-column connection

Figure 2 shows an example of analytical studies of a typical beam-column connection [3], when 1.2 times of ideal full plastic moment ( $=1.2 \cdot Z_p \cdot \sigma_y$ ) is applied in the beam. The column is box-350x350x12 (BCR295: cold-formed rectangular hollow section), the beam is H-500x200x12x19 (SN490B), the diaphragm is 22mm thickness steel plate (SN490C), and the conventional-scallop detail is used.

As shown in Fig.2, stresses occurred in the beam web do not transfer to the column, as the stiffness of a rectangular hollow section in the out-of-plane direction is not rigid enough. The maximum stress occurred at scallop is over the tensile strength  $\sigma_u$ . Two levels of seismic design are performed, namely the elastic design for moderate EQ (level 1) and the elasto-plastic design for severe EQ (level 2).

As shown in Eq.1, the contribution of the web part is, in normal Japanese design practice, neglected for the evaluation of  $M_y$  (the yield moment of a beam) used in level 1 due to the above-mentioned phenomenon.

And  $M_p$  (the plastic moment of a beam) in level 2 is, as described in Eq.2, conservatively evaluated.

It is commonly used in structural design that  $\beta=0$  for the field welding connection (as shown in Fig.1(b) left, high-strength bolts are applied for beam web connection),  $\beta=1$  for the shop welding connection with non-scallop detail, and,  $\beta=1/3$  to  $1/2$  for the shop welding connection with scallop detail.

$$M_y = Z_f \cdot \sigma_y \quad (1)$$

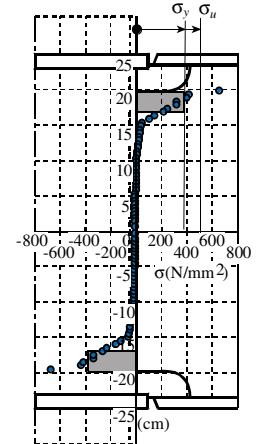
$$M_p = (Z_{fp} + \beta Z_{wp}) \cdot \sigma_y \quad (2)$$

where,

$Z (=Z_f + Z_w)$ ,  $Z_f$ ,  $Z_w$  : elastic sectional modulus of whole beam, flange part and web part, respectively

$Z_p (=Z_{fp} + Z_{wp})$ ,  $Z_{fp}$ ,  $Z_{wp}$ : plastic sectional modulus of whole beam, flange part and web part, respectively

$0 < \beta < 1$  : web contribution factor, which depends on width to thickness ratio of the box column, etc.



**Fig.2**  
**Stress distribution in the beam web**

$\sigma_y$ : yield stress of steel beam

### Performance of beam-column connection

Figure 3 shows a typical example of cyclic loading test of beam-column connection [4]. The column is box-350x350x12 (BCR295: cold-formed rectangular hollow section), the beam is H-500x200x12x19 (SN490B), the diaphragm is 22mm thickness steel plate (SN490C), and non-scallop detail is applied. Gradually increased cyclic load  $Q$  is applied at the end of the beam and then the final fracture, starting from hidden cracks in the end-tab (one side of beam flange tip), occurred after the 6<sup>th</sup> loading cycle. Rotational capacity is defined as Eq.3 or Eq. 4.

$$\theta_p = \sum \theta_{pi}^+ + \sum \theta_{pi}^- \quad (3)$$

$$\eta = \theta_p / \theta_y \quad (4)$$

where,

$\theta_{pi}^+$ ,  $\theta_{pi}^-$ : plastic rotational angle of the  $i^{\text{th}}$  cycle in positive and negative directions, respectively

$\theta_p$ : cumulative plastic rotational capacity,  $\eta$ : cumulative plastic rotational capacity ratio

$\theta_y = 0.005 \sim 0.01$  radian: yield rotational angle, depends on the frame stiffness

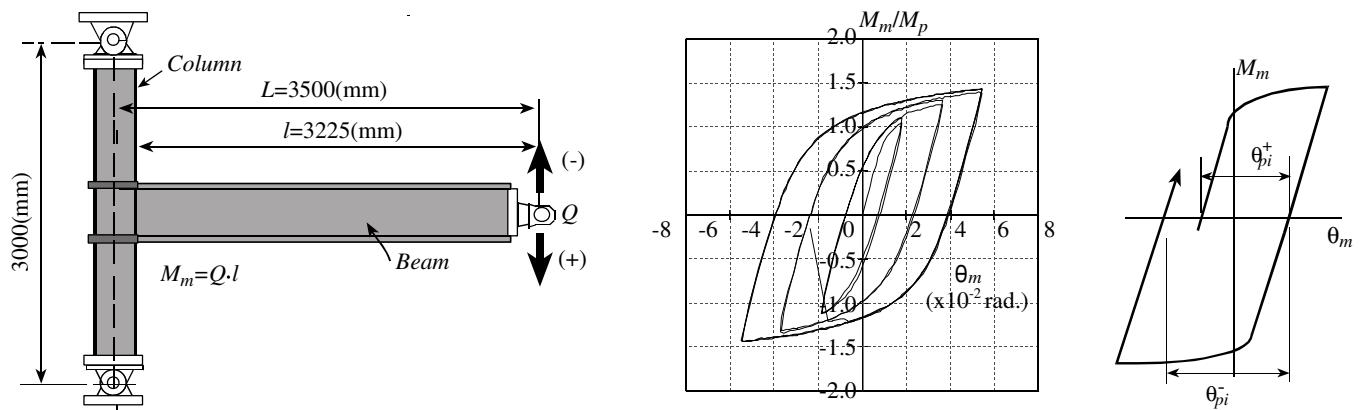


Fig.3 Deformation capacity of the beam-column connection

### Demand of beam-column connection

From many dynamic response studies using many types of EQ waves applied to many types of low to middle-rise moment resistant frames, it was reported that 0.1 to 0.2 radians of plastic cumulative rotational capacity ( $\theta_p$ ) are enough to resist against a severe EQ for steel buildings with the code minimum ultimate retained strength ( $D_s \geq 0.25$  is required in the Japanese seismic code). This level is normally realized for the post-Kobe detailed connections as conventional practice of welding/fabrication in Japan.

### Welding joint of beam-column connection

The cumulative rotational capacity ratio ( $\eta$ ) of the moment connection is affected by welding procedures, steel materials, and details.

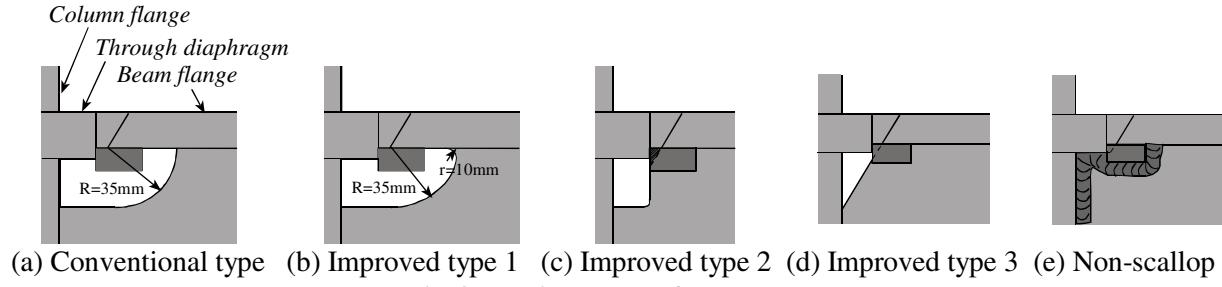
As for the steel materials, the SN specified steel (JIS G 3136) is commonly used after the Kobe EQ, therefore, the yield ratio YR, the through-thickness strengths, or the chemical composition Ceq are guaranteed, and the notch toughness vE exceeding 70J at 0°C is easily available.

The non-scallop details are widely used in shop welding connections, therefore, the welding procedures (the welding wire materials, heat-inputs, inter-pass temperatures, types of scallop, types of end-tub, tuck-welding positions for bucking-bar, or the offset tolerance) have an important role for ductility.

### *Non-scallop detail*

Non-scallop or improved-scallop type welding joint details, shown in Fig. 4, are widely used after the Kobe EQ. They can reduce the stress-concentration occurred in the K-area (the crossing point of beam flange to web), therefore brittle fracture after slightly exceeding the yield-stress level, starting from the bottom of scallop, can be avoided. These improved details can occur normally when exceeding 0.2 radians of cumulative rotational capacity  $\theta_p$ .

However, an additional cost is necessary to introduce new scallop-machines for steel fabricators.



**Fig.4 Various type of scallop shapes**

### *End tub (run-off tub)*

The final fracture occurs from the hidden crack in the end-tubs at the stress-concentrated tip of the beam-flange, when the brittle fracture starting from the K-area is prevented by non-scallop details. Steel or ceramics are used for the end-tub materials, however skilled techniques are needed for welders not to make welding defects around the end-tubs.

### *Tuck-welding position for bucking-bar*

A short bead welding, such as a tuck-welding, makes hardened HAZ (heat affected zone), therefore tuck-welding on the stress concentration points (such as K-area, tips of beam-flange, or underneath of the bucking-bar at the lower flange of beams) are prohibited in conventional recommendations to avoid the risk of brittle fracture [5]. Tuck-welding for bucking-bar is recommended at 1/4 position of the beam flange width with 50mm of minimum length.

### *Offset tolerance in full-penetrate welding joint*

Notification No.1464 of the Ministry of Land, Infrastructure and Transport, issued in 2001, described the offset limitations for full-penetrated welding joints. As an example, in the case of plate-thickness  $t > 15\text{mm}$ , the offset  $e$  should be  $e \leq t/10$  and  $e \leq 3\text{mm}$  without detailed examination.

In the practical fabrication process, these are too strict to satisfy for the full-penetrate welding joint of column-through-diaphragm and beam-flange, therefore the plate thickness of diaphragm is usually +6mm (+0~3mm was usually applied in the pre-Kobe detail) of beam-flange thickness [6].

### *Heat-input, inter-pass temperatures, and welding wire*

Semi-automatic CO<sub>2</sub> or Ar-CO<sub>2</sub> gas-shielded welding method is widely used for steel buildings, however, the welding wires (YGW-11,15: 490N/mm<sup>2</sup> class or YGW-18,19: 540N/mm<sup>2</sup> class in nominal strength) are guaranteed under the table 1 (JIS Z 3312) conditions.

It is reported that the welding time is 1600 seconds (including waiting time for cooling) and the arc-time ratio is under 30% when table 1 welding condition (YGW-11 by 25KJ/cm of heat-input and 250°C of the maximum inter-pass temperature) is applied to the SN490 steel plate of 22mm thickness and 300mm width.

It is also reported that the welding time is only 450 seconds, arc-time ratio is 100%, and the inter-pass temperature is over 500°C for 40KJ/cm of heat-input with no-control for inter-pass temperature.

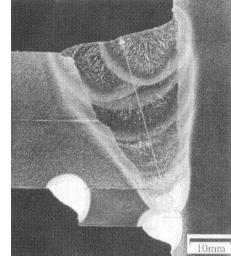
**Table 1. Recommended conditions for welding wire (JIS Z 3312)**

	Welding condition		Strength class of steel plate		
	Heat-input	Inter-pass temperature	400N/mm <sup>2</sup>	490N/mm <sup>2</sup>	540N/mm <sup>2</sup>
1	15~20kJ/cm	≤150°C	YGW-11,15,18,19	YGW-11,15,18,19	YGW-18,19
2	15~30kJ/cm	≤250°C	YGW-11,15,18,19	YGW-11,15,18,19	YGW-18,19
3	15~40kJ/cm	≤350°C	YGW-11,15,18,19	YGW-18,19	

Inter-pass temperature is measured at 25mm apart from weld metal center (also at steel plate center in longitudinal direction)

These examples show that the quality control of heat-input and inter-pass temperature is highly related to the fabrication efficiency, and in other words, also to the cost.

As shown in photo.1, a multi-layered welding is applied to a steel building, therefore the former bead is re-melted in the latter pass and mechanical properties of deposit are different in this narrow area. The mechanical property of low-alloy steel is roughly decided by the cooling time from 800°C to 500°C ( $\Delta T_{8/5}$ ) and chemical composition. It is quantitatively determined how the mechanical properties of welding metal (strength and notch-toughness) are deteriorated as heat-input and inter-pass temperature increased.



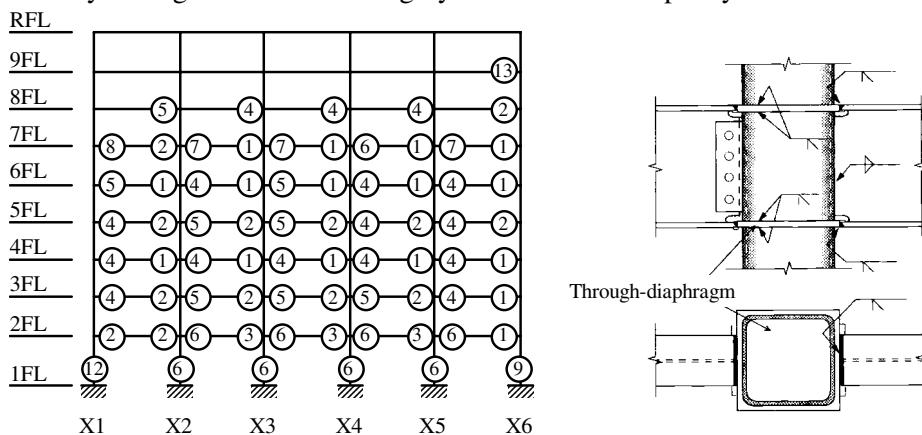
**Photo. 1  
Multi-layered welding**

## COST PERFORMANCE OF BEAM-COLUMN CONNECTIONS

### Classification of welding joints according to strength and deformation demand

Figure 5 shows the ultimate state of push over analysis for 9 stories typical steel moment frame (span=9m, story height=4m, constructed using cold-formed rectangular hollow section columns and H-shaped beams) in Japan. Equivalent static EQ load is applied from the left side, and ○ shows the plastic-hinges with sequential numbers of formation. The ultimate retained strength of this frame is more than 1.3 times of the required ( $D_s=0.25$  under 7<sup>th</sup> FL., and =0.3 over 8<sup>th</sup> FL.) by the Japanese seismic code, and more than 1.4 times of the strong-column & weak-beam type connection. Plastic-hinges did not develop in columns without the base of 1<sup>st</sup> floor, and the early hinges are located at the right side beam-ends.

The critical points (exceeding yield stresses and strain-hardening occurrence) in this frame are the full-penetrated welding joints of beam-flanges located at plastic-hinges, therefore the most careful welding/fabrications and inspections are required for these points. However, in other joints, such as the full-penetrated welding of column to diaphragm or panel-zone to diaphragm, these careful procedures are less required as they belong to a different category in deformation capacity.



**Fig. 5 Ultimate state expected from push over analysis of a typical steel building in Japan**

According to table 2, the full-penetrated welding joints of beam-flanges, where plastic-hinges are expected, categorize as A, and the full-penetrated welding of column to diaphragm or panel-zone to diaphragm are categorized as B. It is reasonable to change the quality control level or inspection level according to the welding joint category from a cost performance point of view.

**Table 2 Classification of the full-penetrate welding joint according to strength and deformation demand**

	Stress and strain level	Examples of joints
Category A	$\sigma_y < \sigma_{\max}$ , $\varepsilon_{st} < \varepsilon_{\max}$	Plastic-hinge position expected from analysis
Category B	$\sigma_y \approx \sigma_{\max}$ , $\varepsilon_{\max} < \varepsilon_{st}$	General moment connection except plastic-hinge position
Category C	$\sigma_{\max} < \sigma_y$ , or compression stress	Column to column joint where no-tensile stress occurs

where,  $\sigma_{\max}$  : maximum stress occurred in the weld metal,  $\sigma_y$ : yield stress of the weld metal,  
 $\varepsilon_{\max}$  : maximum strain occurred in the weld metal,  $\varepsilon_{st}$ : strain-hardening strain of the weld metal

There are some methods to change the category from A to B, as follows.

#### *Over-matching procedure*

There are many items however the most important property of the welding joint is the strength. Over-matching condition ( $F_w > F_b$ , where,  $F_w$ : strength of weld metal,  $F_b$ : strength of base metal) is required for full-penetrated welding joints. Therefore, one solution is to use high-strength welding wire, such as YGW-18 (540N/mm<sup>2</sup> class) instead of YGW-11 (490N/mm<sup>2</sup> class), for SN490 (490N/mm<sup>2</sup> class) steel (base-metal), even if the material-cost increases.

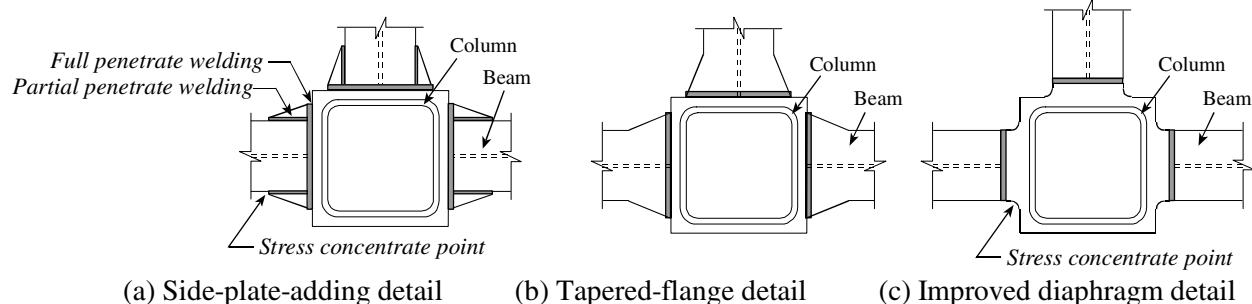
#### *New details to reduce the stress-concentration*

New details aimed to reduce the stress concentration occurred in the beam flange welding metal, such as side-plate-adding details (Fig.6 (a)) or tapered-flange details (Fig.6 (b)), are developed. These details are mainly used for the field welding type connection (the full-penetrate field welding is applied for the beam flange and high-strength bolts for the beam web), which aims to recover the bending strength of beams (the web contribution factor  $\beta=0$  as shown in Eq.2).

The tapered-flange details are used only for built-up H sections however the side-plate-adding details can be applied for roll-H sections, which are more economical steel members. In Fig.6(a), the partial penetrated welding for the side plate needs the most careful welding/fabrication procedure, because the weld-beads are located at the stress concentration points (shifted plastic-hinge location). There is the know-how to avoid the brittle fracture in side-plate welding procedure.

Fig.6 (c) is mainly used for the shop welding type connection.

This detail is aims to shift the stress concentration point from weld metal to base metal with corner R, where the hidden notches accompanied with welding process are eliminated.



**Fig. 6 Stress reduction details**

## CONCLUSIONS

### **Cost performance**

A building owner requests, as a business strategy, the highest performance levels in seismic design. The cost of steel fabrication tends to increase in order to meet these higher demands, however a price-down pressure was accelerated from the recent economic recession.

The seismic capacity of a steel building depends highly on the welding qualities of beam to column joints. Fabrication costs, especially welding costs, are affected by joint details or the required deformation capacities.

Most economical details shall be selected to clear the structural demands, which are different from joint types or joint positions even in the same structure.

There are two ways of cost saving for steel building, namely the material-cost saving and the process-cost saving.

When the weld metal located in the maximum stress point, the most careful welding/fabrication process is required. However, some improved details, in which the maximum stress point is shifted from weld metal to base metal, are enough to apply more economical process.

### **Detail categorizing**

Details, used for the most of low to middle rise steel buildings in Japan, are categorized as A, B, C in this paper according to the required structural performance for the purpose of the process cost balancing. Categories of details specified in the design document may be helpful for steel fabrications.

### **Performance-based seismic design**

"Life-safety against severe EQ" and "Serviceability after moderate EQ" are the standard seismic design criteria, however "Repairability against severe EQ" is the higher level of seismic design criteria. In the 1995 Kobe EQ and the 1994 Northridge EQ, steel moment frames were damaged, however "Life-safety" was basically attained except poor welding procedures, because the steel is essentially a ductile material.

Poor welding procedures were observed in the collapsed buildings. Like for example a fillet welding was applied even if a full-penetrate welding was specified in the design document causing the loss of human life. However, this could be avoided with an inspection process.

Careful welding procedures in the post-Kobe details are mainly aims for the performance of "Repairability".

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